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			Document checker Mark Pniewski
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			Document Author Gayan Wijesundara
			Document checker Mark Pniewski

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1 Introduction

This report describes the verification of the design for Upper Brook Street (MAUBS) VMU panel. Details specific to this VMU are included here but aspects that are more generic to Ultra High-Performance Fibre Reinforced Concrete (UHPFRC) panel verifications on this project are detailed in the UHPFRC Design and FE Analysis Specifications (1).

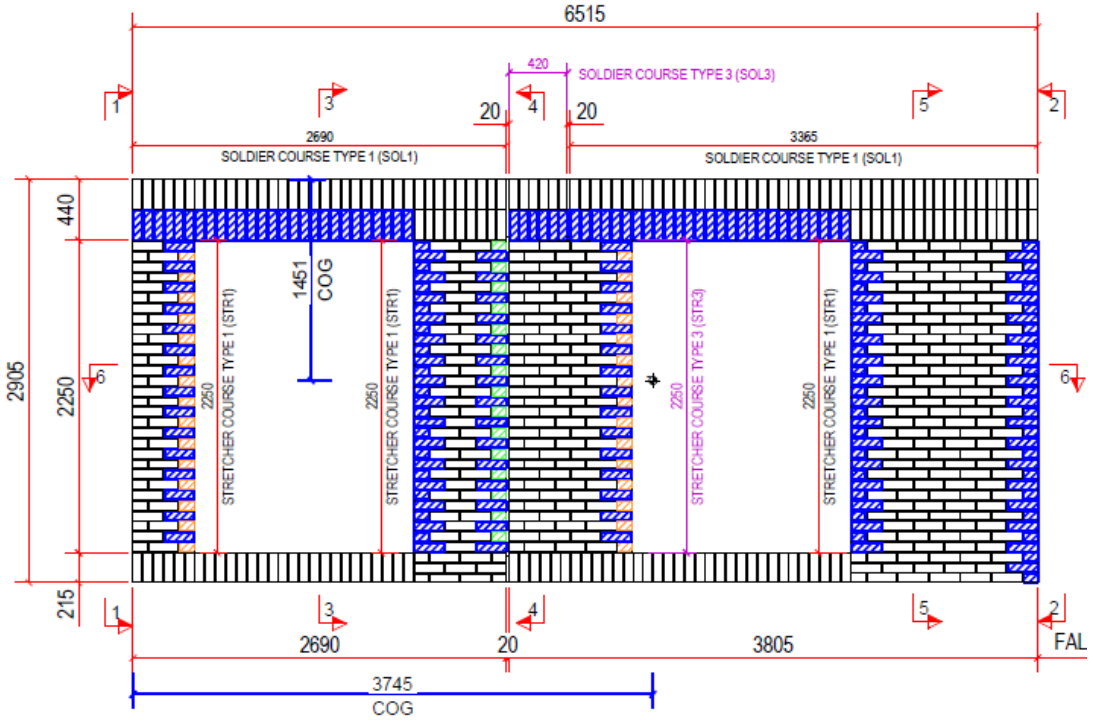
2 References

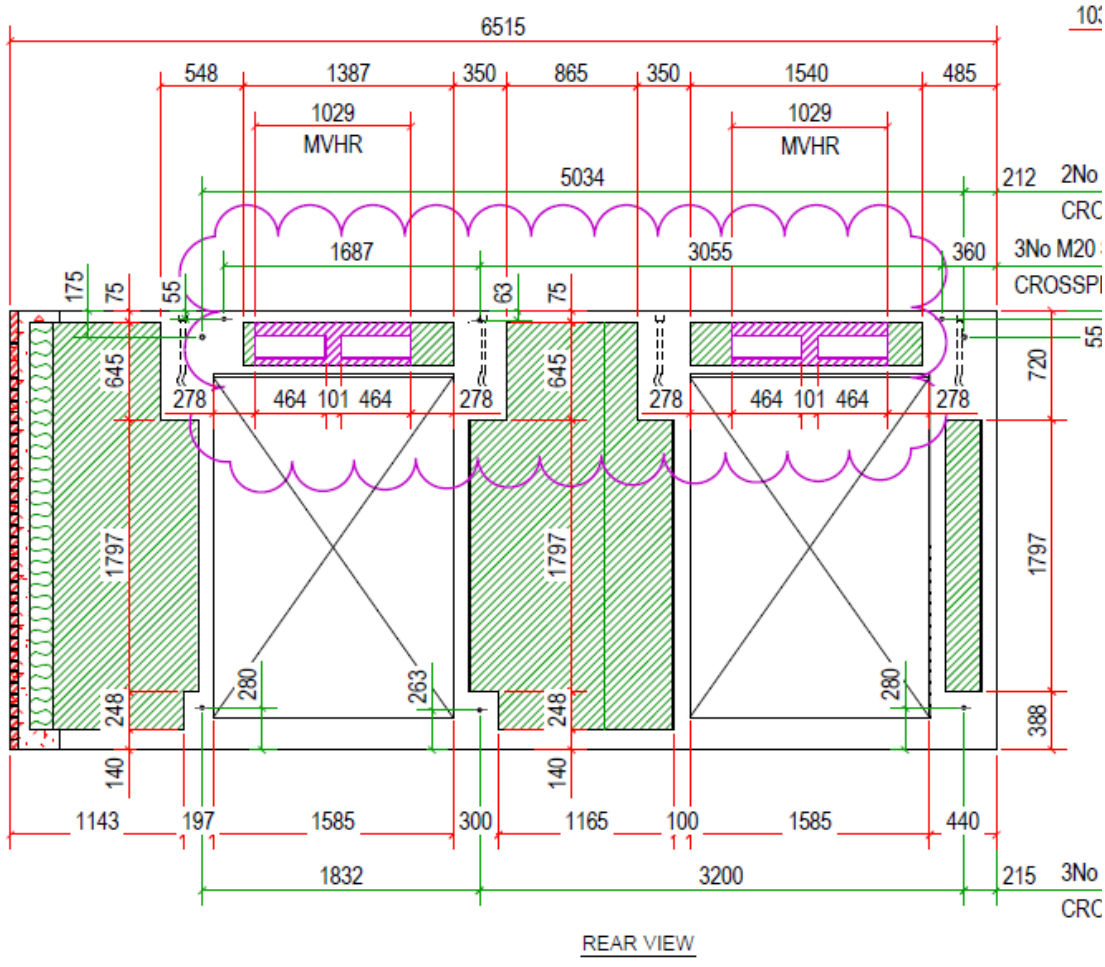
References specific to this VMU design and analysis are listed below.

1) Appendix A: 24112 – Upper Brook Street - UHPFRC Design and FE Analysis Specifications
2) MAUBS-TPL-XX-ZZ-CA-X-000001 – Design Assumptions and Wind Calculations
3) 24112-(04)-SA8-C02 Brick faced UHPFRC panel VMU proposal – Upper Brook Street block

3 Geometry

The VMU has an approximate weight of 6625.0kg. It is 2.9m tall and 6.52m wide. It is clad in 50mm bricks. The panel arrangement including key dimensions and fixing is shown on drawing 24112-(04)-SA8-C02 (3). The geometry of the panel is shown in Figure 3-1 and 3-2.


6515
420
SOLDIER COURSE TYPE 3 (SOL3)
20
20
2690
SOLDIER COURSE TYPE 1 (SOL1)
3365
SOLDIER COURSE TYPE 1 (SOL1)
440
2250
2905
215
1451
COG
STRETCHER COURSE TYPE 1 (STR1)
STRETCHER COURSE TYPE 3 (STR3)
2250
2690
20
3805
FAL
3745
COG


6515
548
1387
350
865
350
1540
485
1029
MVHR
1029
MVHR
5034
212
2No
CRC
1687
3055
360
3No M20
CROSSPI
175
75
55
63
75
55
645
645
720
278
464
101
464
278
278
464
101
464
278
1797
1797
1797
280
263
280
248
248
388
140
140
1143
197
1585
300
1165
100
1585
440
1832
3200
215
3No
CRC
REAR VIEW

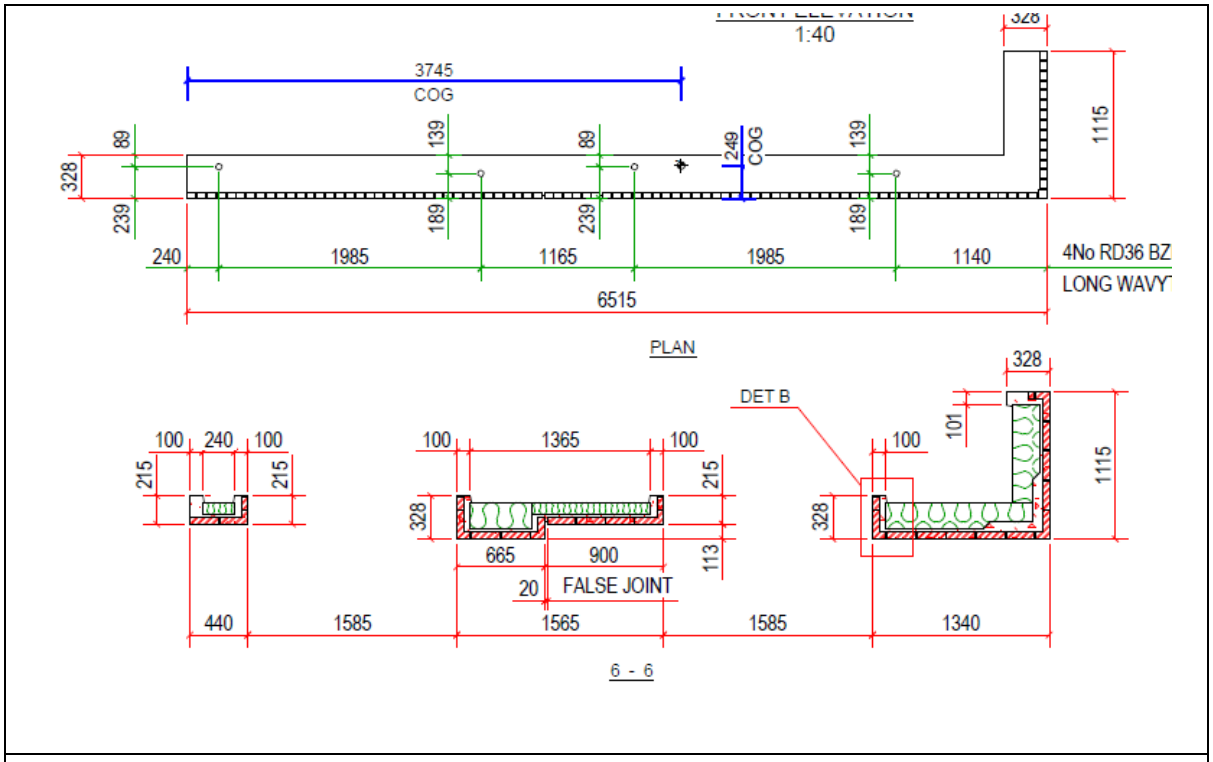


Figure 3-1 Panel geometry

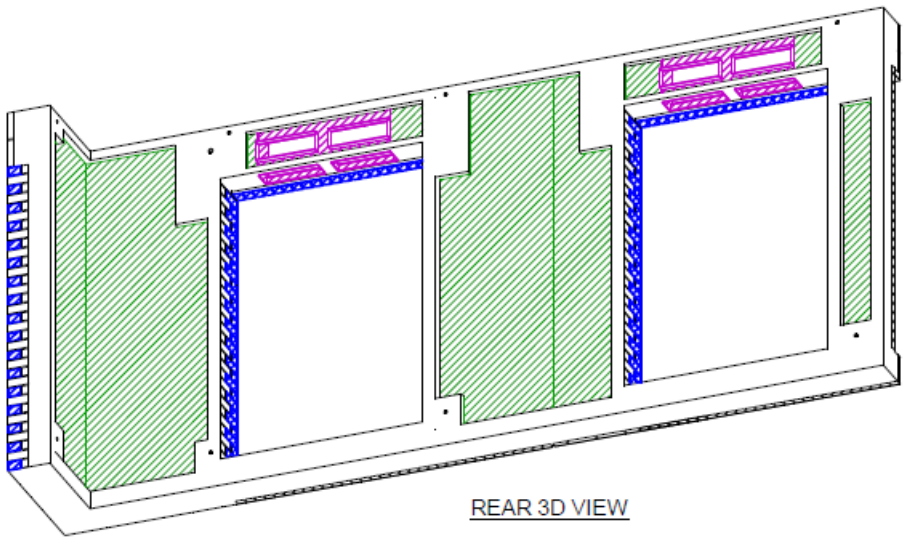


Figure 3-2 VMU panel 3D geometry.

4 Material Properties

Mechanical Properties of UHPFRC including identity cards for early age and 28-day parameters, material partial safety factors, fibre orientation factors and design values used in analysis are included in the UHPFRC Design and FE Analysis Specifications (1).

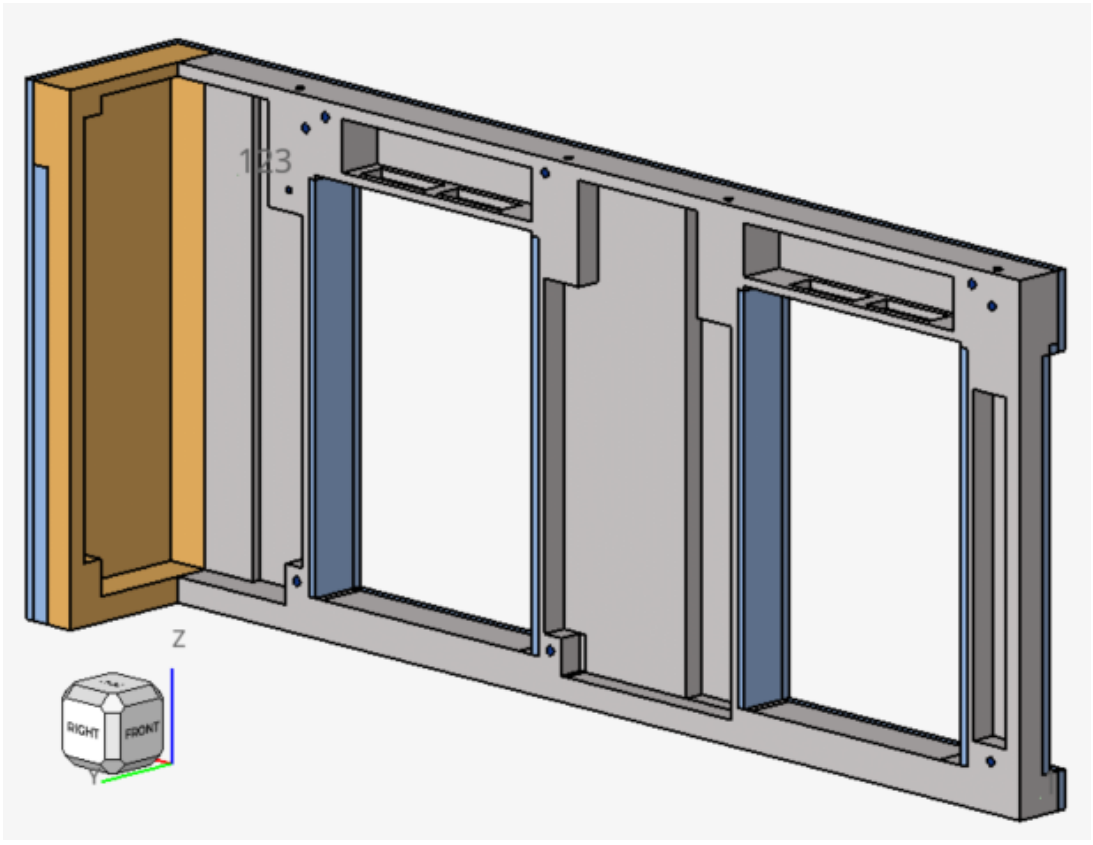
5 Finite Element Model and Mesh

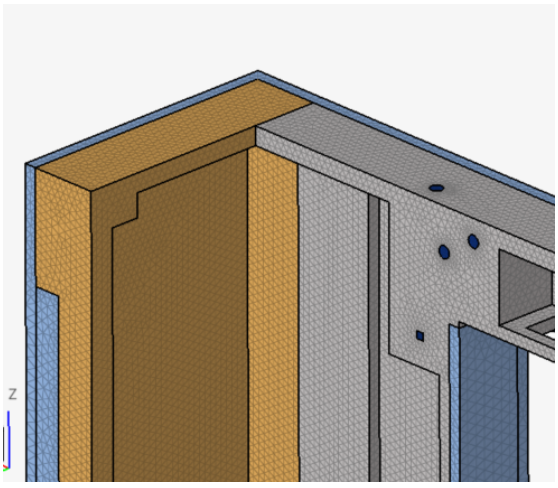
The FE modelling methodology including element types used to represent UHPFRC and any necessary reinforcement is detailed in the UHPFRC Design and FE Analysis Specifications (1). A

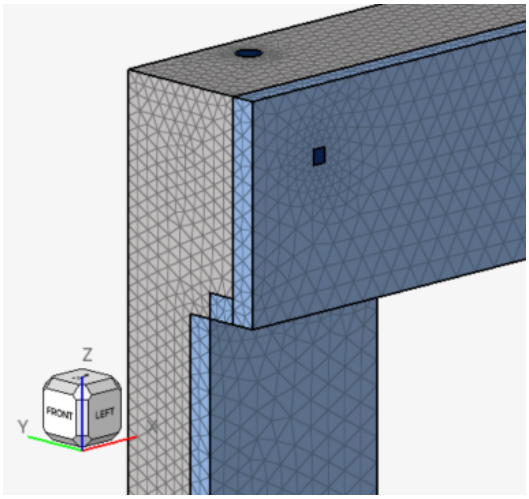
description of the software used for geometry modelling, structural analysis and post-processing is also included (1).

The arrangement of the panel VMU FE model is shown in Figure 5-1.

All brick units are modelled as a 50mm thick uniform 3D solid with reduced stiffness (E_{UHPFRC} / 1000) but with full density to ensure that UHPFRC is conservatively checked against the total weight of the panel.



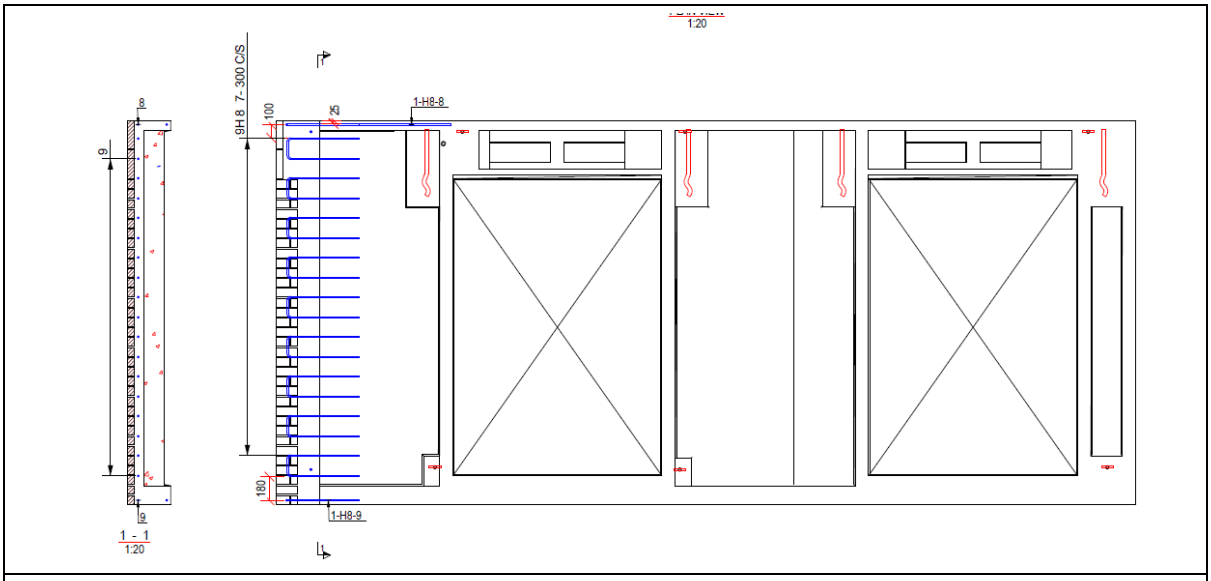




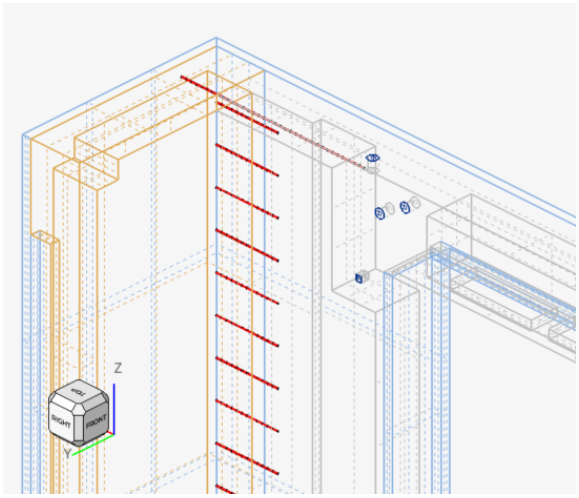
Closeup view: top left corner	Brick facing in light blue

Figure 5-1 3D FE mesh for VMU panel

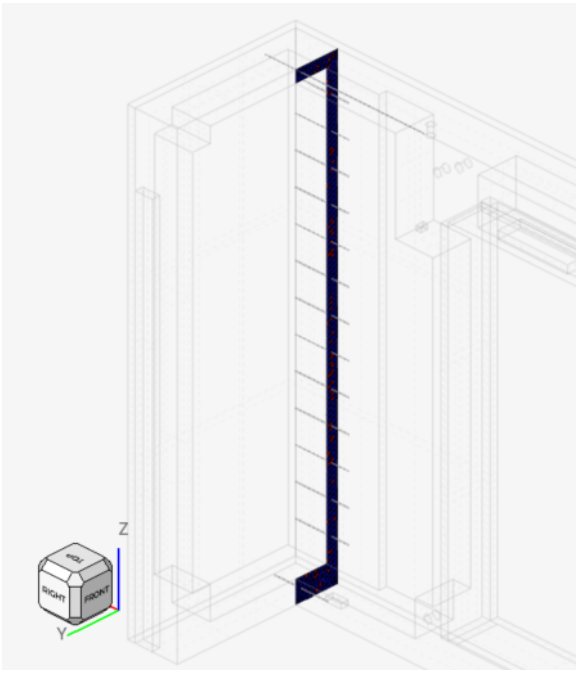
This VMU is a twice-cast panel with a temporary construction joint between the two UHPFRC segments shown in yellow and grey in Figure 5-1. The joint interface was modelled as a frictionless contact. Additional reinforcement across the joint was modelled as 1D beam elements connected to the surrounding concrete with no bond failure (see Figure 5-2 for rebar arrangement).



Rebar layout



1D beam elements in red



Frictionless contact in blue

Figure 5-2 Interface at the temporary construction joint

6 Boundary Conditions

6.1 Demoulding (DM)

The panel will be cast with the stone tile face that will form the outside of the façade facing down. Once the UHPFRC has gained sufficient strength, it will be demoulded by using demoulding Thorp lift

type A6, referenced in the UHPFRC Design and FE Analysis Specifications document (1). These boundary conditions and A6 lifting arrangement are illustrated in Figure 6-1. For demoulding analysis load combination, gravity acts in the negative Y direction during demoulding.

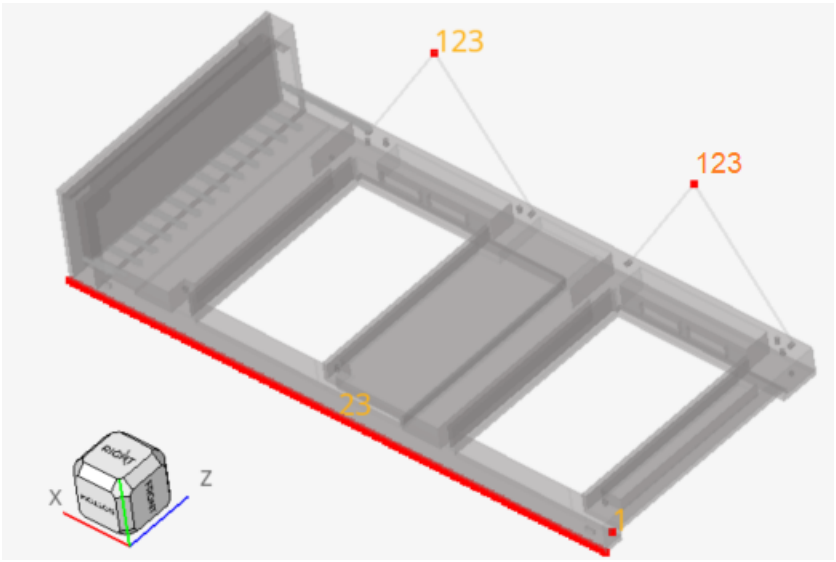


Figure 6-1 Supports during demoulding: 1 = global X fixity; 2 = global Y fixity; 3 = global Z fixity

6.2 Vertical Lifting (FL)

The panel is lifted using Thorp lift type B2. For vertical lifting analyses load combination FL gravity acts in the negative Z direction as shown in Figure 6-2.

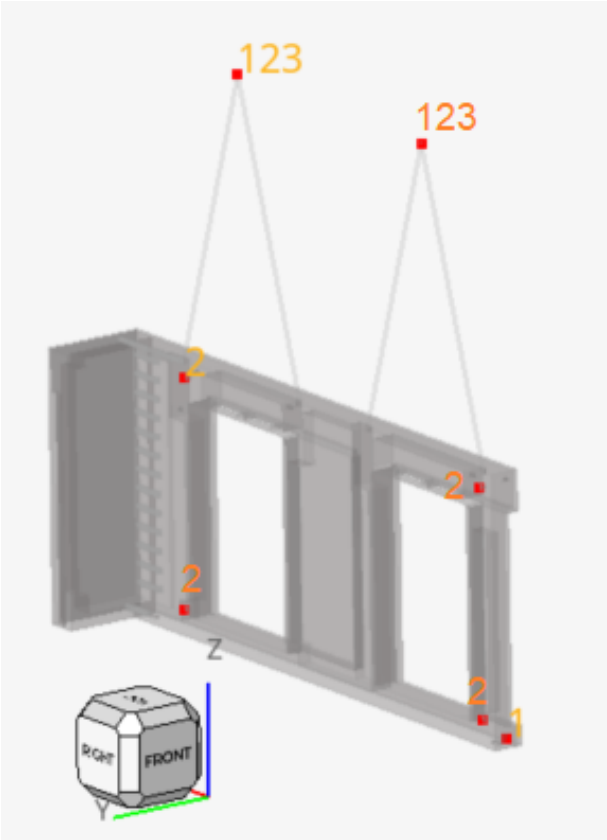
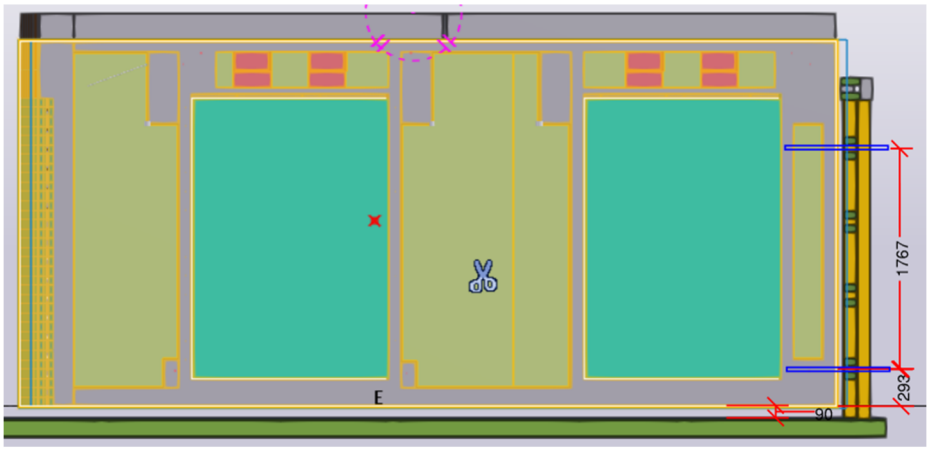


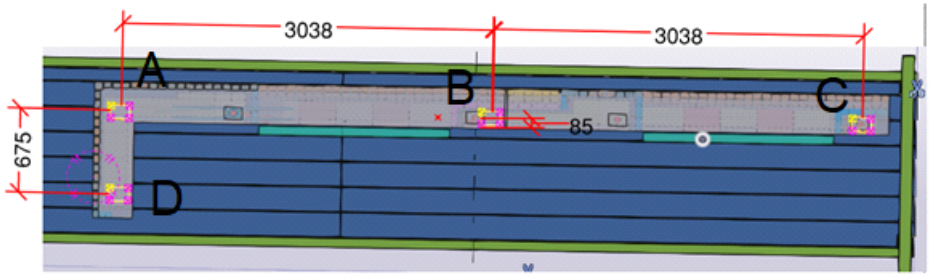
Figure 6-2 Supports during vertical lifting: 1 = global X fixity; 2 = global Y fixity; 3 = global Z fixity

6.3 VMU on the Skid (PD)

When the panel is in its final position, it will be on the skid and supported vertically off by four shims and restrained to the side frame of the skid against lateral movements. These boundary conditions are illustrated in Figure 6-3.



Panel lateral fixings in blue



Plan view showing shim locations and labels (A, B, C and D)

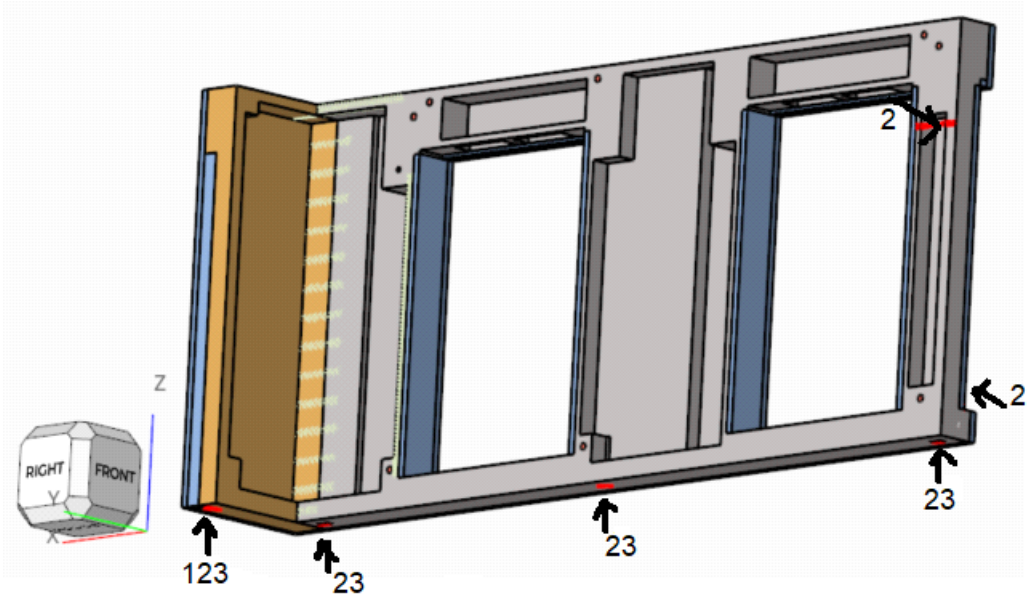


Figure 6-3 Supports for final conditions: 1 = global X fixity; 2 = global Y fixity; 3 = global Z fixity

7 Loading

7.1 Load Cases

The type of loads that have been considered are listed below (see Section 8 for load combinations).

Table 7-1 Analysis load cases

Reference	Description	FE application method
D	UHPFC self-weight	Modelled volumes, mass density, gravity
B	Brick/tile self-weight	Modelled as volumes with appropriate mass density
G	Window self-weight	Point load per window (window area x area density of window (75.0kg/m^2))
P	+Y wind pressure*	Modelled surface area, pressure. Based on 0.97 kPa.
N	-Y wind pressure	Modelled surface area, pressure. Based on 0.97 kPa.

* See Appendix B for wind pressure calculations

7.1.1 Self-weight of panel

Self-weight of the panel was defined using the gravity loading option in Hypermesh and automatically accounted for by the software. For the demoulding stage, this is applied in the global -Y direction, see Figure 6-1. For vertical lifting self-weight is applied in the global -Z direction, see Figure 6-2.

7.1.2 Self-weight of brick facing

Additional 50mm brick facing has been modelled as 3D solids with appropriate mass density (see Figure 7-1). Therefore, self-weight of these units was automatically accounted for in the model.

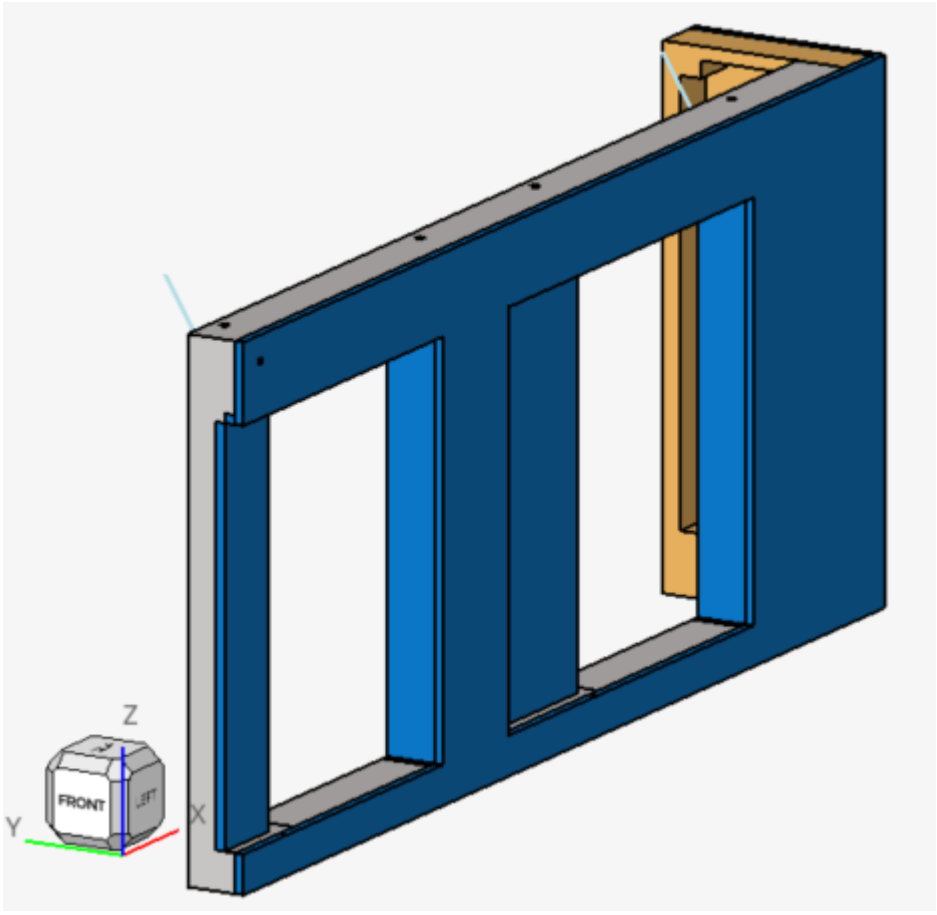


Figure 7-1 3D solid stone segments in blue

7.1.3 Wind Load on Panel

Maximum net wind pressure acting on the VMU was calculated to be 0.97kPa in accordance with BS EN 1991-1-4: 2005 (Appendix B). This was applied in both directions (global +Y and -Y directions) using two separate load cases (referred as P and N in Table 7-1).

Figure 7-2 shows the area where wind pressure is applied.

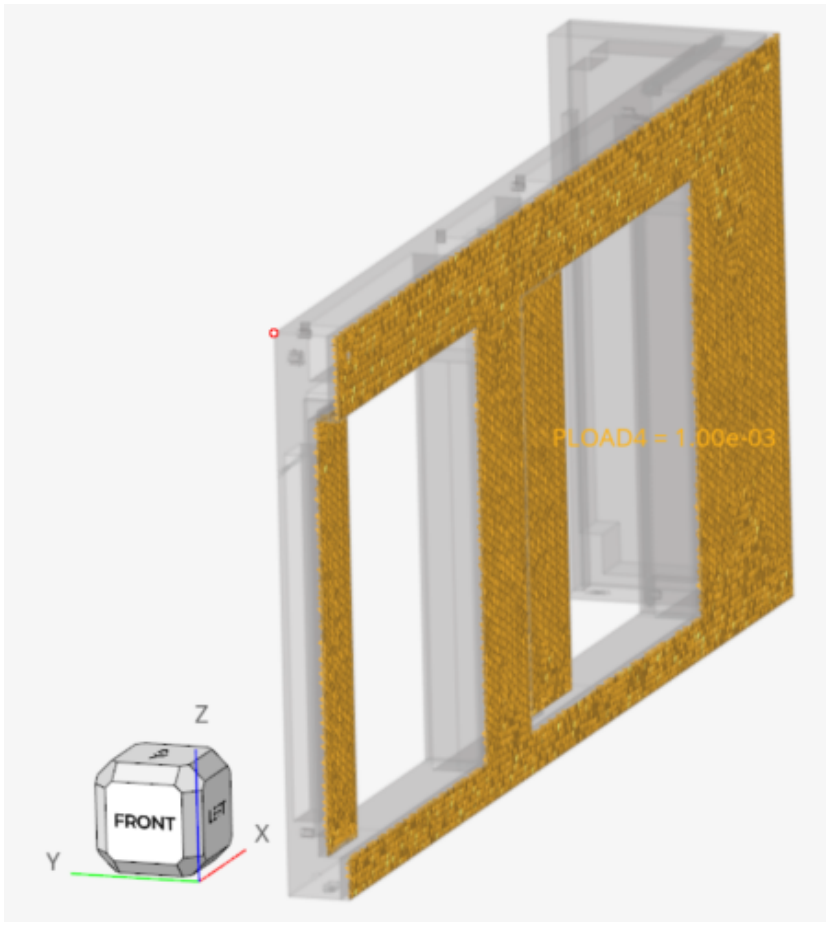


Figure 7-2 Wind pressure on panel

7.1.4 Wind load on windows

The total wind loads on each window has been applied as a point load at the centre of the window as shown in Figure 7-3. The magnitude of each load is equal to the product of wind pressure (0.97 kPa) and window area (3.57m^2).

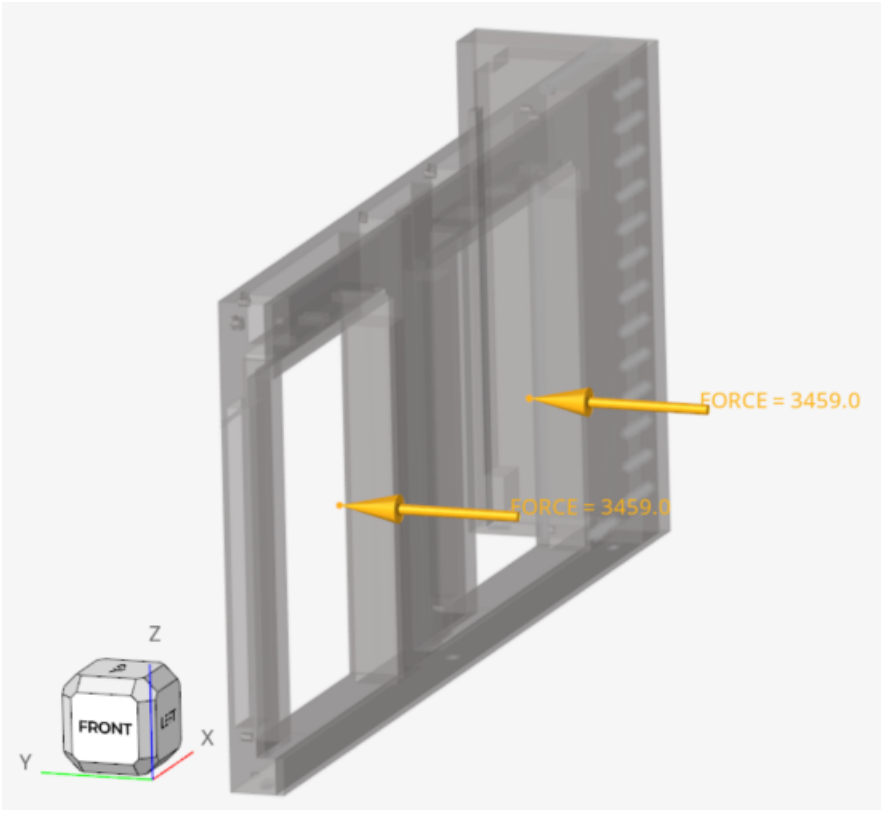


Figure 7-3 Wind load on window

7.1.5 Self-weight of windows

Self-weight of each window has been applied as a point load as shown in Figure 7-4. The magnitude of each load is equal to the product of the area density of window (75.0kg/m^2) and window area (3.57m^2).

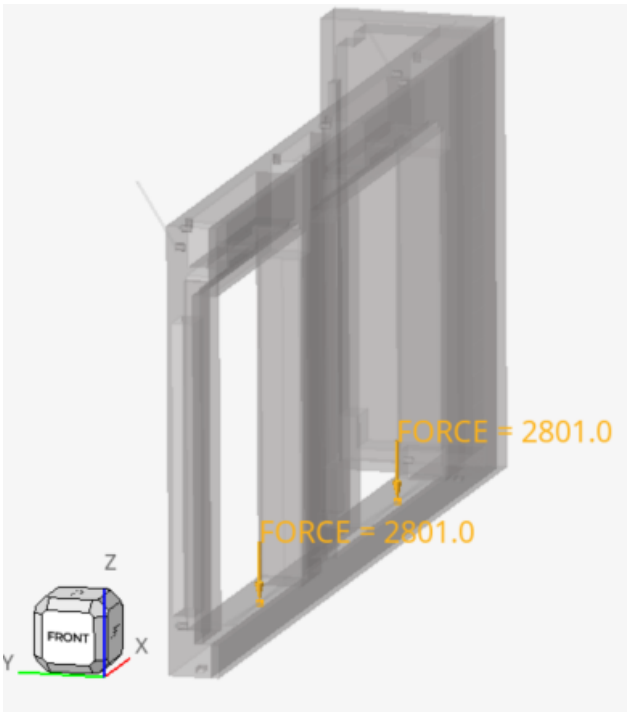


Figure 7-4 Self-weight of window

7.2 Combinations

SLS and ULS load combinations are listed in Table 7-2 and Table 7-3 respectively. Partial load factors and dynamic load factor definitions are detailed in ref. 1.

Table 7-2 SLS load combinations

Combination and model IDs	Age	Description	Load case IDs, load participation factors, and dynamic load factors
SLS DM	Early stage	Demould lift (PL the same)	D 1.0 x 1.1 B 1.0 x 1.1
SLS FL		Factory lift	D 1.0 x 1.1 B 1.0 x 1.1
VMU +Y or -Y Wind	28-day	Site lift	D 1.0 B 1.0 G 1.0 P or N 1.0
Load case IDs are given in Table 7-1			

Table 7-3 ULS load combinations

Combination and model IDs	Age	Description	Load case IDs, load participation factors, and dynamic load factors
ULS DM	Early stage	Demould lift (PL the same)	D 1.35 x 1.1 B 1.35 x 1.1
ULS FL		Factory lift	D 1.35 x 1.1 B 1.35 x 1.1
VMU +Y or -Y Wind	28-day	Site lift	D 1.35 B 1.35 G 1.35 P or N 1.5
Load case IDs are given in Table 7-1			

8 Results

The results for each of the demoulding, factory lifting, and panel's final position on the skid are given in this section. Results of predicted crack widths have been summarised by enveloping results for all conditions and presenting values as contoured utilisations. Crack width utilisation is defined as follows:

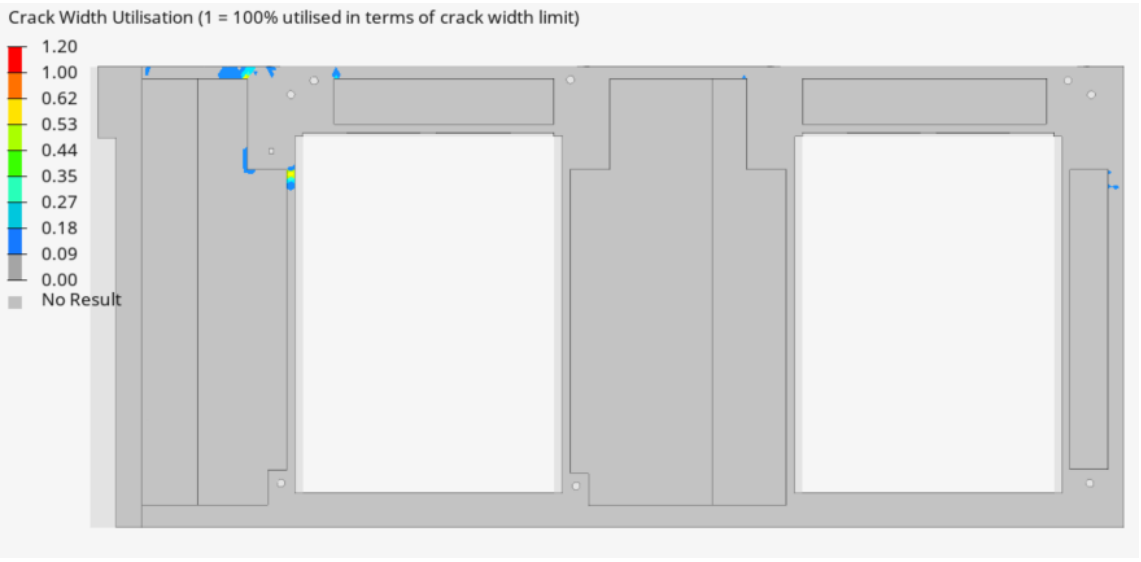
Crack width utilisation envelope = maximum predicted crack width / crack width limit

The crack width limit is 0.1mm and 0.3mm for SLS and ULS limit states respectively. If the crack width utilisation is less than 1.0 then the design verification check passes.

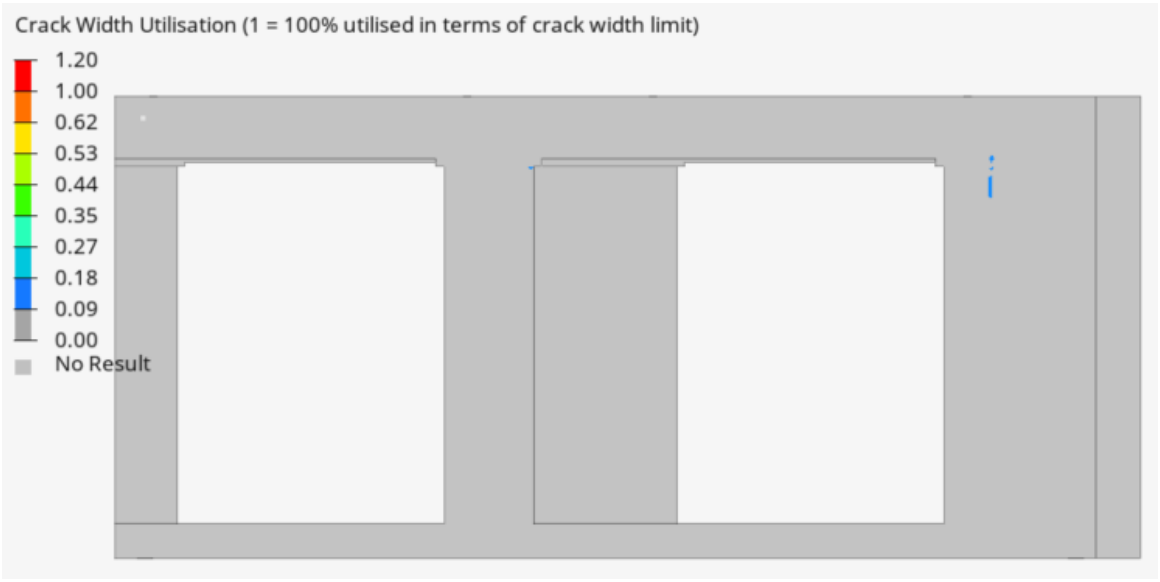
All additional results such as stress and strain envelopes for different load combinations are provided in Appendix C.

8.1 Crack width utilisation

Contour plots of enveloped maximum crack width utilisations are shown in Figure 8-1. Results are plotted for panel inside and outside faces.



Inside face

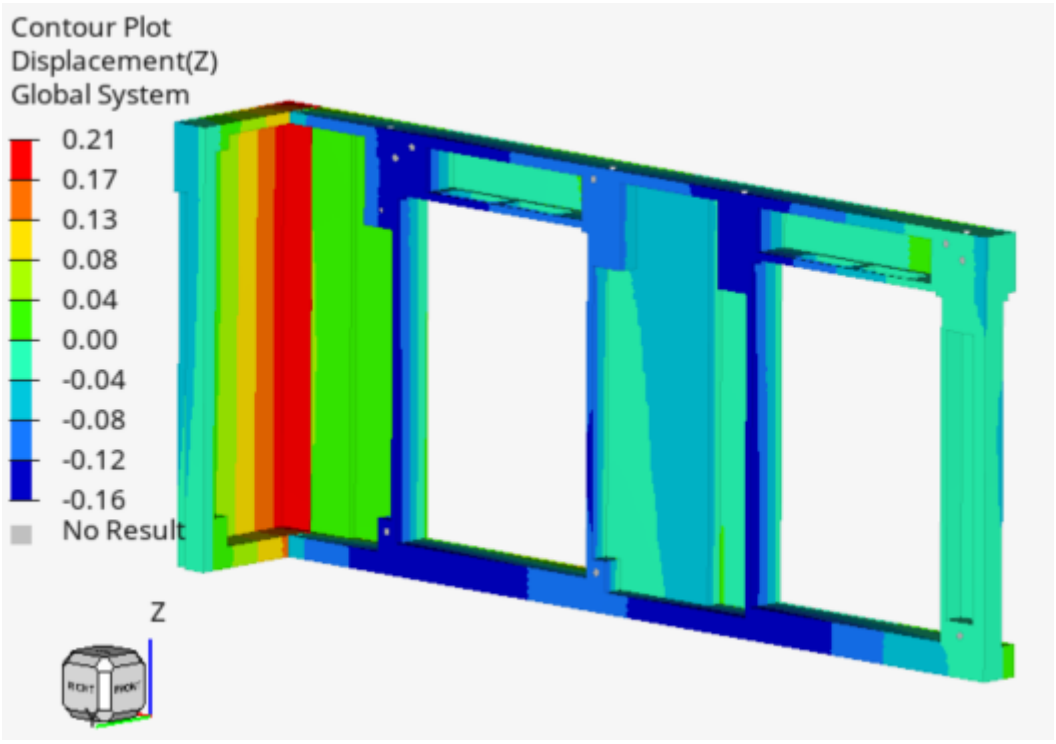


Outside face

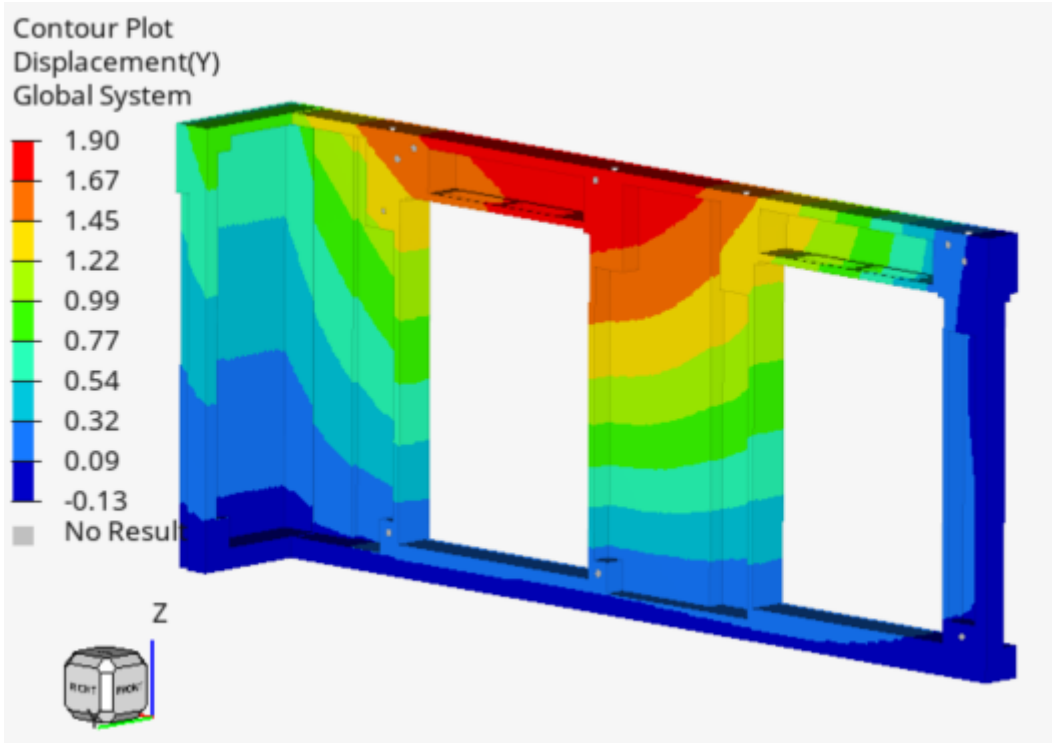
Figure 8-1 Crack width utilisation envelope for panel

8.2 Deformation Plots

Maximum predicted deformation in mm when the panel is positioned on the skid is shown in Figure 8-2.



In-plane



Out-of-plane

Figure 8-2 Displacement envelope [mm] under SLS +Y and -Y wind loads

The predicted maximum in-plane deformation at SLS is 0.21mm and less than $L_{in_plane}/360$ = 2905/360 = 8.1mm. Therefore, the predicted maximum deformations at SLS are acceptable, with a utilisation of 2.6%.

The predicted maximum out-of-plane deformation at SLS is 1.9mm and less than $L_{out_of_plane}/200$ = 2905/200 = 14.5 mm. Therefore, the predicted maximum deformations at SLS are acceptable, with a utilisation of 13.1%.

8.3 Stress in reinforcement

Envelope of maximum predicted stress in MPa in the reinforcement shown in Figure 8-3.

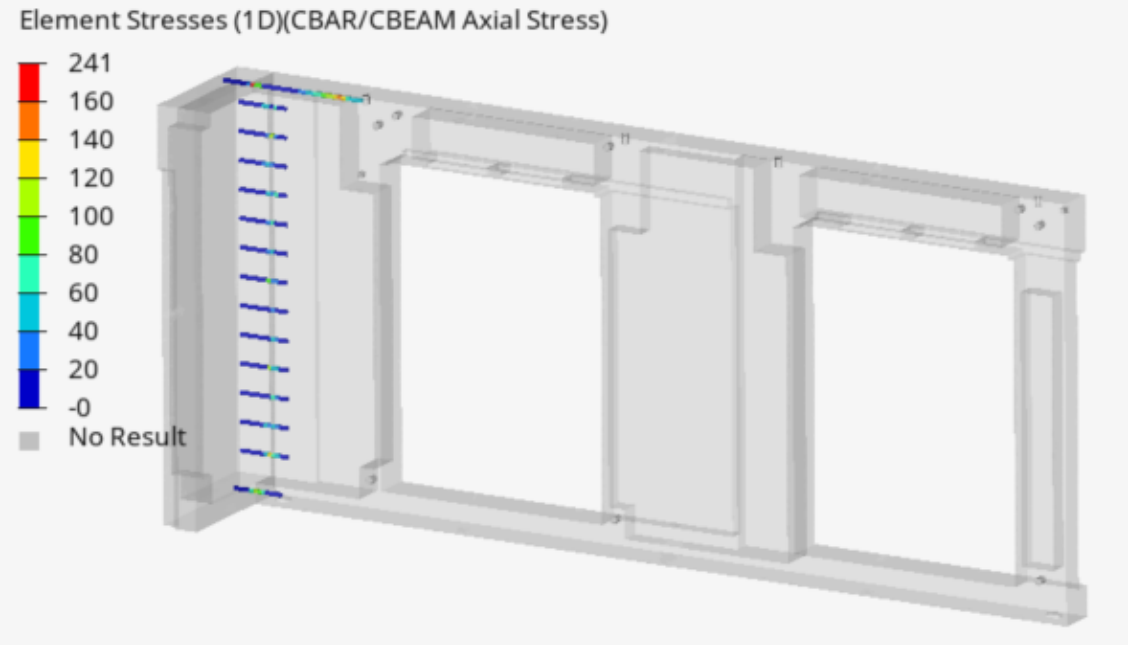


Figure 8-3 Envelope of maximum stress in the reinforcement (MPa)

8.4 Stability of VMU on Skid

The VMU is laterally restrained to the skid. Therefore, it cannot overturn itself.

The predicted total vertical and horizontal reactions on shims A, B and C (see Figure 8-4 for shim IDs and locations) due to wind loads are provided in Table 8-1 together with percentage utilisation against sliding due to panel bending. The results confirm that the net vertical force at each shim location is adequate to prevent panel sliding on the skid.

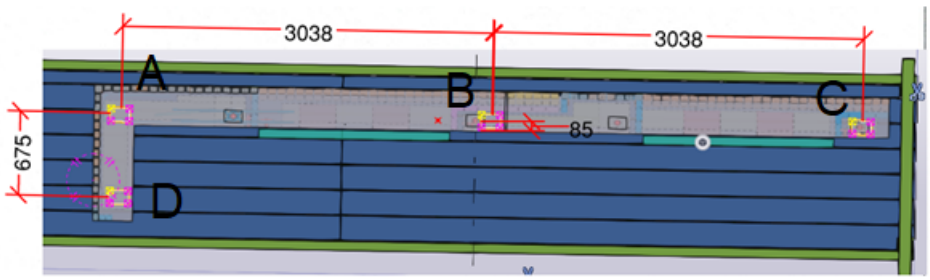


Figure 8-4 Shim IDs and their locations

Table 8-1 Vertical and horizontal reactions on shims A and B

Shim ID	Reaction Force in kN		Friction Capacity (kN)*	% Utilisation
	Vertical	Horizontal		
A	36.0	2.9	14.4	20.1
B	21.1	6.4	8.44	75.8
C	13.5	1.6	5.4	29.6

*Friction capacity = vertical force x frictional coefficient (0.4).

8.5 Stability of the Entire Assembly (VMU + skid) under Wind Loads

Stability of the skid and VMU against overturning and sliding due to wind-induced lateral loads is checked for multiple skid inclinations. See Figure 8-5 for an illustration of forces considered in stability calculations.

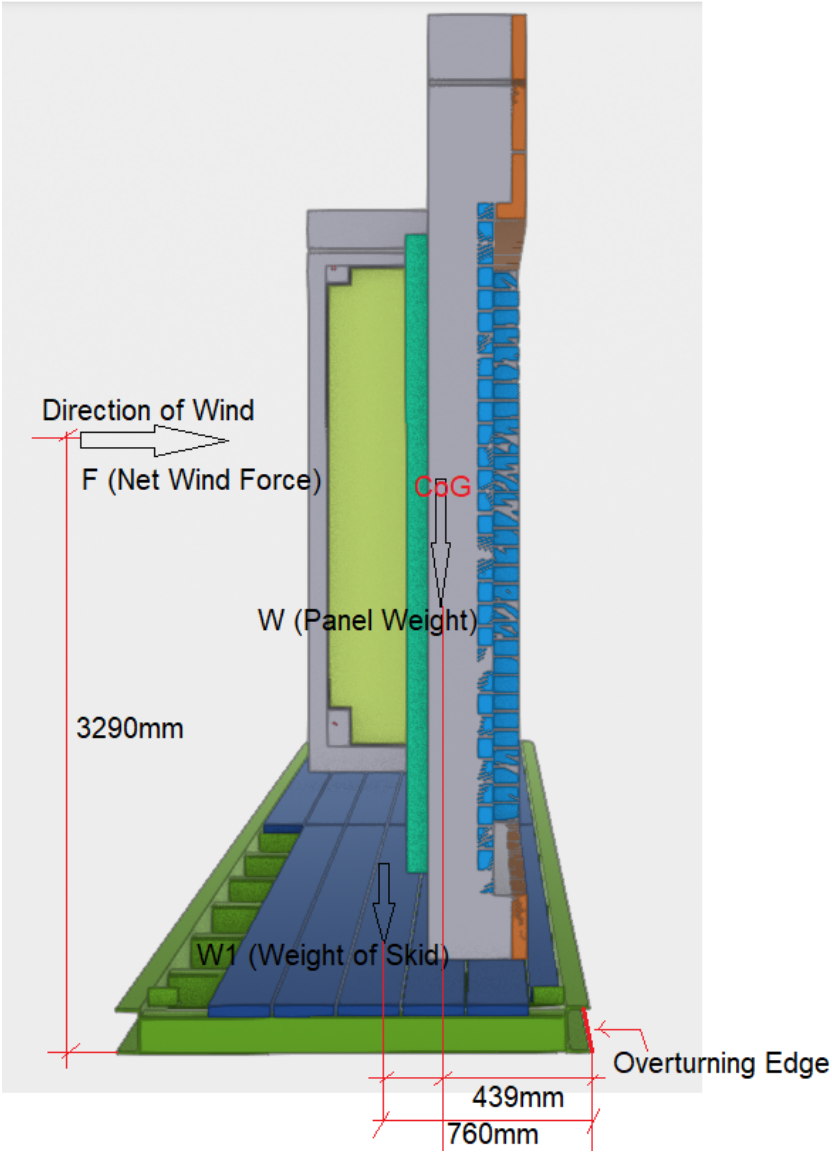


Figure 8-5 Forces for VMU and skid stability calculations

A summary of calculated percentage utilisations against overturning and sliding are provided in Table 8-2. Appendix D provides detailed stability calculations. The results indicate that the VMU is stable up to a skid inclination of 5 degrees against overturning. The VMU is safe against skid sliding.

Table 8-2 Summary of panel stability calculations

Skid Inclination	% Utilisation in terms of Overturning	% Utilisation in terms of Sliding
0	77	33
2.5	86	
5.0	96	
7.5	109	

9 Conclusions

This report details a series of nonlinear FE analyses, design verification and stability checks carried out to verify that the proposed MAUBS VMU have been designed to meet the structural performance required by design code requirements as detailed in the UHPFRC design and analysis document (1).

The results indicate that the VMU has adequate structural capacity to resist the loads (see Table 9-1).

The VMU is stable on the skid against sliding and overturning due to wind.

Table 9-1 Design verification conclusion

Verification check	Utilisation [%]	Result
Maximum crack width utilisation	<60	Pass
Maximum SLS deformation in the permanent condition	13.1	Pass
Reinforcement utilisation in terms of yielding	42	Pass

APPENDIX A: 24112 – Upper Brook Street - UHPFRC Design and FE Analysis Specifications

THORP PRECAST LTD

Project Name	**Upper Brook Street (UBS)**
Thorp Project No.	**24112**
Client Name	**McClaren**

Document Title	**UHPFRC Design and FE Analysis Specifications**
Document Author	**Gayan Wijesundara**
Document Checker	**Mark Pniewski**

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1 Introduction

This document details the methodology used to analyse different Ultra High-Performance Fibre Reinforced Concrete (UHPFRC) panel types designed to meet the structural performance criteria, in accordance with the codes and standards listed in Section 4.6 - References. Separate reports are produced for the work carried out on individual panels covering input and including geometry, temporary lifting and permanent conditions, and design verification results. Details and parameters common to all panels are described in this report.

The structural performance of UHPFRC panels is checked using a series of static structural nonlinear finite element (FE) analyses using Altair Hyperworks 2024.1 FE software.

During manufacture and installation panels undergo a sequence of operations and those considered are listed in Table 1-1. During each of these operations, a different set of material properties, loads, and boundary conditions are used that relate to different conditions.

Table 1-1 Sequence of manufacturing and handling operations

Operation ID	Operation	UHFRPC age
1	Demoulding, Pitching & Vertical lifting & Storage	Early-stage
2	Vertical lifting on site	28-day
3	Final position	28-day

2 UHPFRC Material Properties

2.1 Mechanical Properties of UHPFRC

UHPFRC is a material with high compressive strength compared to traditional concrete, typically achieving more than 150 N/mm² (MPa). Under tension, the material exhibits a monolithic and semi-ductile behaviour due to the presence of fibres bonded within the binder matrix.

The characteristic material properties used in all FE analyses, including material partial safety factors and fibre orientation factors, are listed in Table 2-1 and Table 2-2 for 28-day aged and early-stage aged UHPFRC. Mechanical properties at 28-days are based on Thorp test data given in the Thorp UHPFRC Technical Datasheet G200MAF V2 (ref. 1). The early-stage mechanical properties are based on Thorp test data listed in the 18-hour UHPFRC Mechanical Properties Report (ref. 2).

Table 2-1 Mechanical properties of UHPFRC at 28 days

Property	Value	Unit	Description
K_G	1.9*	-	Fibre orientation factor for global effects
K_L	2.2*	-	Fibre orientation factor for local effects
L_f	14	[mm]	Steel fibre length
ρ	25	kN/m^3	Density
f_{ck}	150	MPa	Characteristic compressive strength at 28 days
$f_{ctk,el}$	8.0	MPa	Characteristic value of limit of elasticity under tension
f_{ctfk}	7.5	MPa	Characteristic value of post-cracking strength at 0.3mm
f_{ctfm}	8.5	MPa	Mean value of post-cracking strength
f_{ctf1}	6.0	MPa	Characteristic value of post-cracking strength at 1%

E_{cm}	50	GPa	Mean value of Young's modulus
μ	0.2	-	Poisson's ratio
ϕ	1.0	-	Creep coefficient
γ	1.3 (ULS); 1.0 (SLS)	-	Partial safety factor of material

Table 2-2 Mechanical properties of UHPFRC for early stages

Property	Value	Unit	Description
KG	1.9*	-	Fibre orientation factor for global effects
KL	2.2*	-	Fibre orientation factor for local effects
L_f	14	[mm]	Steel fibre length
ρ	2500	kg/m^3	Density
f_{ck}	50	MPa	Characteristic compressive strength at 18 hours
$f_{ctk,el}$	3.0	MPa	Characteristic value of limit of elasticity under tension
f_{ctfk}	4.5	MPa	Characteristic value of post-cracking strength at 0.3mm
f_{ctfm}	6.0	MPa	Mean value of post-cracking strength
f_{ctf1}	4.0	MPa	Characteristic value of post-cracking strength at 1%
E_{cm}	30	GPa	Mean value of Young's modulus
μ	0.2	-	Poisson's ratio
ϕ	1.0	-	Creep coefficient
γ	1.3 (ULS); 1.0 (SLS)	-	Partial safety factor of material

**Initial values assumed in design calculations, verification & further changes if required, once the results of suitability tests are available.*

Material properties of steel reinforcement where used is given in Table 2-3.

Table 2-3 Mechanical properties of steel reinforcement

Property	Value	Unit	Description
ρ	7850	kg/m^3	Density
f_y	500	MPa	Yield strength
E_s	200	GPa	Young's modulus
μ	0.3	-	Poisson's ratio
γ	1.15(ULS)	-	Partial safety factor of material

2.2 Material Time Dependency of UHPFRC

Time dependency of material properties is based on UHPFRC maturity and considered at two ages: 28 days and 18 hours referred to as early-stage.

UHPFRC creep is included for the calculation of panel long term deflections, where relevant, and is based on the approximation given by using half Young's modulus (50/2 = 25 GPa).

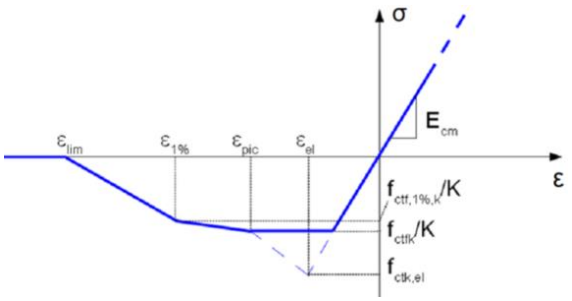
2.3 Material Behaviour Curves of UHPFRC

As the orientation of fibres within UHPFRC is affected by the casting process, UHPFRC is an anisotropic material; it is stronger in the directions where higher proportions of fibre have become aligned. However, as it is difficult to assess the final orientation of fibres within a precast component, UHPFRC is conservatively modelled as an isotropic material, using the properties of the material in the weakest orientation.

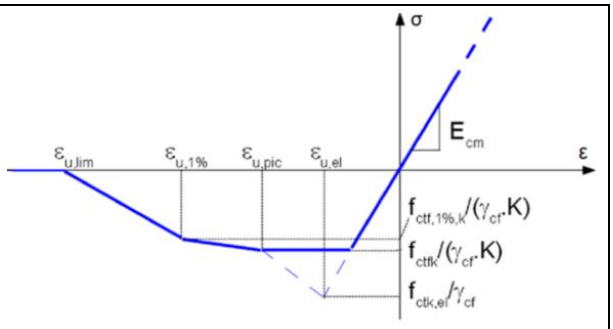
As no UK design code practice currently exists for the use structural use of UHPFRC, the rules set out in the French National annex NF P18-710:2016 (ref. 3) to EN 1992 are used.

2.3.1 Tensile behaviour

The method used to define UHPFRC tensile behaviour is based on NF P18-710:2016 (ref. 3) para. 3.1.7.3.1 (2) and the conventional stress-strain method which provides the relationship form with key material parameters determined from testing and the material datasheets (ref. 1 & ref. 2). Separate relationships for design are provided for SLS and ULS conditions and these are reproduced (Figure 3.205 (ref. 3)) in Figure 2-1.



SLS



ULS

Figure 2-1 UHPFRC constitutive relationship in tension

These relationships are for thick members where thickness e > 3 Lf. Lf is the fibre length.

Conservatively, and once fibre orientation factors are introduced, the relationship for tension for design is strain softening (tension class T1*).

The magnitudes of the strains ε_{pic} and $\varepsilon_{u,pic}$ shown in Figure 2-1 depend on the spacing of cracks, often referred to as the crack band width, the permissible crack width limit and elastic strain. A relationship exists for simple beam behaviour that relates crack band width to beam thickness. The thicker the beam the less ductile the material is and the greater the distance between cracks. For simplicity and conservatism, the thickness of the beam has been taken as 75 mm (if required this value will be adjusted on a panel-by-panel basis).

2.3.2 Compressive behaviour

The method used to define UHPFRC compressive behaviour at ULS is based on NF P18-710:2016 (ref. 3) para.3.1.7.2. The simplified stress-strain relationship is reproduced in Figure 2-2.

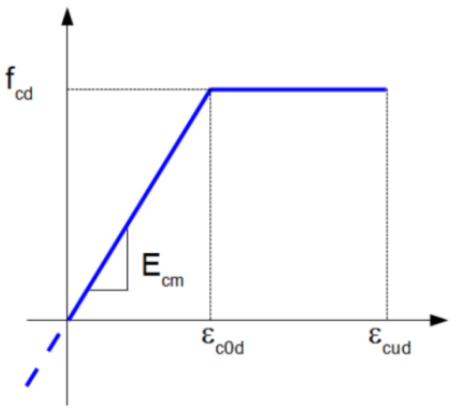


Figure 2-2 UHPFRC constitutive relationship in compression

3 Finite Element Models

3.1 Software

The Finite Element (FE) software used for UHPFRC panel design validation is Altair Hyperworks. Altair's implicit solver Optistruct is used to carry out materially non-linear static analyses.

3.2 Material models

3.2.1 UHPFRC

A non-linear material model based on Rankine plasticity is used to simulate the tensile behaviour of UHPFRC with nominal hardening to aid solution convergence. This is equivalent to a three-dimensional principal tensile stress cut-off. Compressive behaviour is limited to the design strength although because of the very high compressive strength UHPFRC generally remains linear under all conditions in compression. The actual material model being used within Altair software giving the required characteristics is based on Cast Iron. A diagram reproduced form Altair's documentation showing the yield surface is illustrated in Figure 3-1.

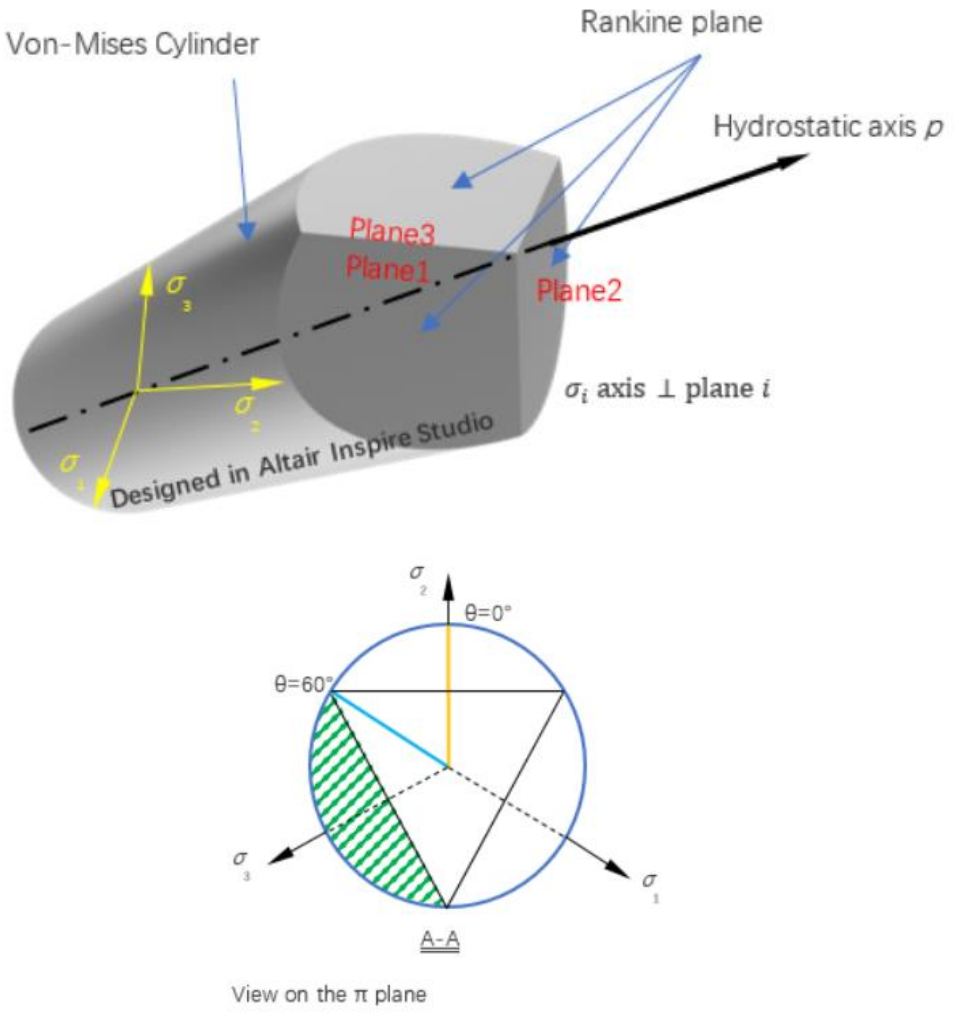


Figure 3-1 UHPFRC material model and yield surface

3.2.2 Steel reinforcement

When required, steel reinforcement uses a linear elastic material model and stress results are checked to ensure that linear conditions are satisfied. The reinforcement is fully bonded to the UHPFRC for all limit states.

3.3 Finite Elements and Meshing

FE models are created using first order TETRA 3D solid elements available in Altair Hyperworks. Although the model consists of 3D solid elements, stress states within relatively thin vertical and horizontal members within each panel, away from local stress concentrations around the fixings, are mostly either unidirectional or bidirectional, therefore allowing the model to be analysed with the material laws discussed in Section 2.3.

Representation of support fixings such as cast-in sockets is made with a zone of linear material to prevent stress concentrations that would be difficult to resolve with the non-linear material model.

1D beam elements are used to model the reinforcement when additional strengthening is required. These are fully bonded with the 3D solid UHPFRC elements using rigid constraint equations. Reinforcement results are checked to ensure modelling assumptions are satisfied.

As the TETRA 3D solid elements are first order high mesh densities must be used to be able to reliably predict structural behaviour and to satisfy solution equilibrium conditions. This is particularly important when UHPFRC detailing involves small dimensions and where parts of panels are very thin. Average element edge lengths used vary between 10 mm and 40 mm and ideally in layers in thin parts of at least three elements.

4 Design Validation

4.1 Material Properties

Design validation of UHPFRC panels is based generally on tensile behaviour as compressive stresses seldom exceed elastic limits. The tensile behaviour is characterised by tensile strains and maximum crack widths.

Material properties used in FE analysis to validate UHPFRC designs are based conservatively on the post-cracking design tensile strengths at strains ε_{pic} and $\varepsilon_{u,pic}$ as shown in Figure 2-1. The design strengths include material partial safety factors and fibre orientation factors. These factors are included in Table 2-1 and Table 2-2. The resulting design tensile strengths used in FE analyses and corresponding limiting crack widths are shown in Table 4-1.

The design tensile strengths at SLS and ULS were determined using the mechanical properties of UHPFRC provided in Section 2.1 and Equations 4-1 and 4-2 respectively.

At SLS:

$$Design\ tensile\ strength\ = \frac{f_{ctfk}}{K_G} \quad (4-1)$$

At ULS:

$$Design\ tensile\ strength\ = \frac{f_{ctfk}}{\gamma \times K_G} \quad (4-2)$$

Where, f_{ctfk}, K_G and γ are characteristic value of post-cracking strength, fibre orientation factor for global effects and partial safety factor for materials, respectively.

Table 4-1 Limiting values for design validation.

Limit state	UHPFRC age	Limiting factor	Value	Unit
SLS	Early stage	Design tensile strength	2.37	MPa
	28-day		3.95	MPa
	Any stage	Crack width	0.1	mm
		In-plane deflection E = 50/2 = 25 GPa	≤ Span / 360	
		Out-of-plane-deflection E = 50/2 = 25 GPa	H/200; for H≤ 3000mm H/300 + 5mm; for > 3000mm	
ULS	Early stage	Design tensile strength	1.82	MPa
	28-day		3.04	MPa
	Any stage	Crack width	0.3	mm

For deflection checks under long-term loading a reduced Young's modulus value is used as an approximation for UHPFRC creep behaviour.

Design stress strain curves based on the UHPFRC constitutive relationship in tension and those used for FE analysis and design validation are plotted in Figure 4-1 and Figure 4-2. Here the relationships have substituted values for the material parameters listed in Table 2-1 and Table 2-2.

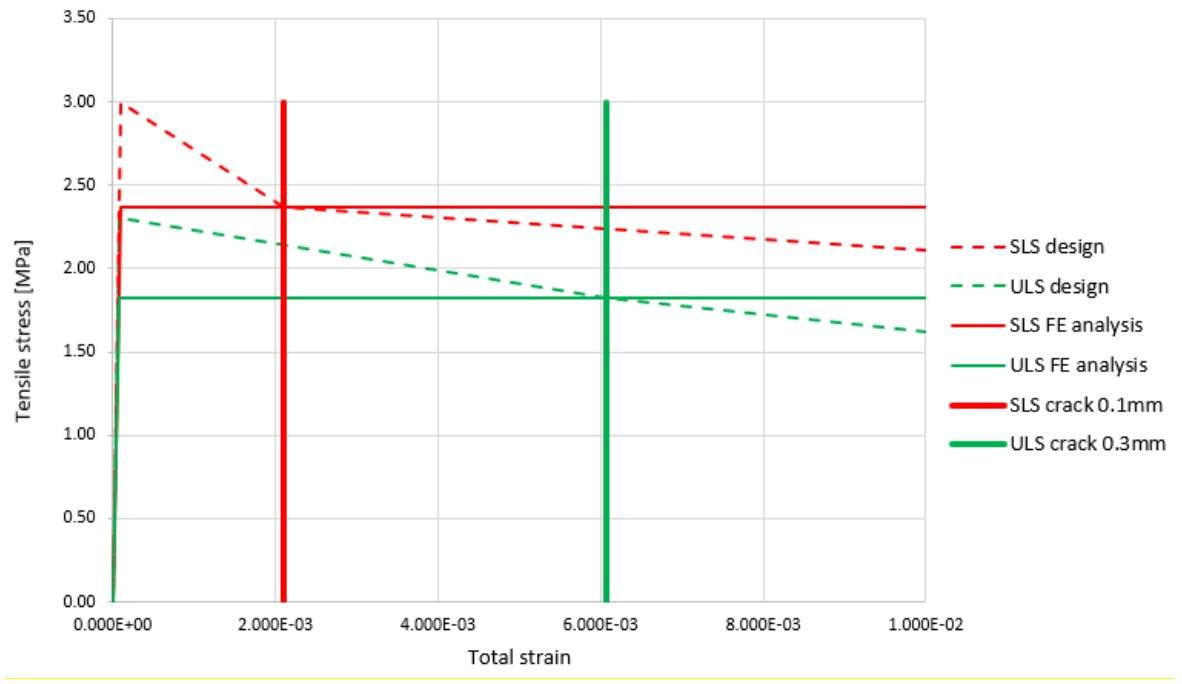


Figure 4-1 Early-stage characteristic and design stress strain curves

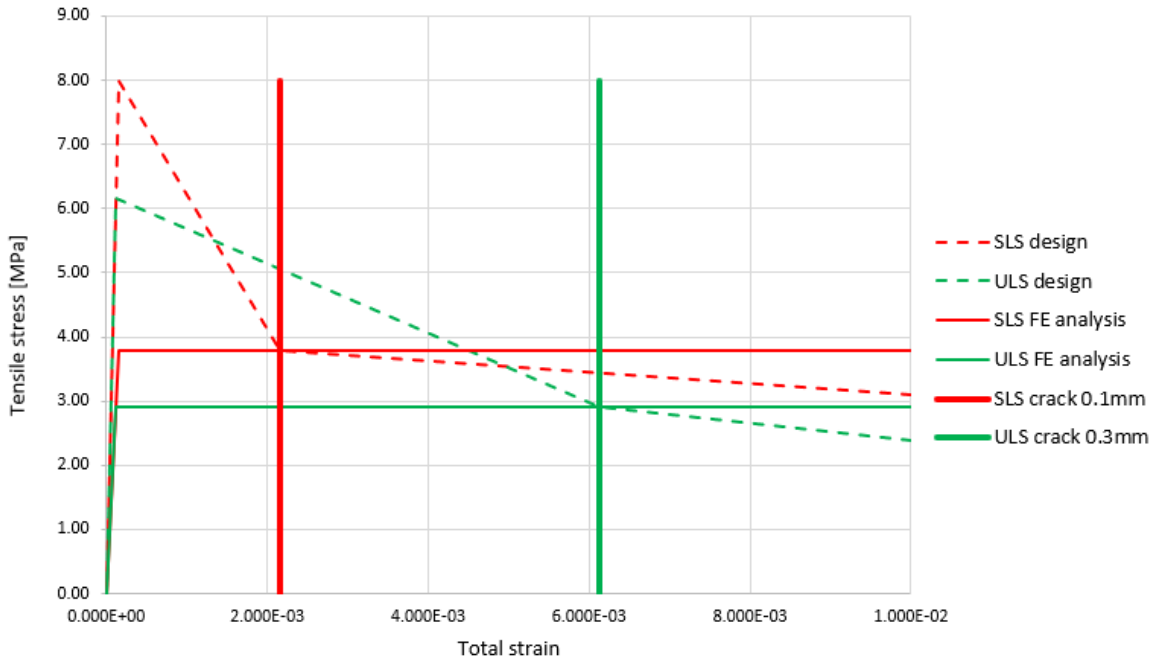


Figure 4-2 28 Day characteristic and design stress strain curves

4.2 Design loads

Partial load safety factors for different types of loads and load combinations are listed in Section 3.0 of the Design assumptions report. Each panel report lists the load types and load combinations used for the design verification of that panel.

The type of loads that are considered for each panel design verification and naming convention are selected from the list in Table 4-2 along with a description for that panel.

Table 4-2 Analysis load cases

Reference	Description	FE application method
D	UHPFC self-weight	Modelled volume, mass density, gravity.
B	Brick/tile self-weight	Modelled surface area, pressure. Based on brick/tile thickness. Thicknesses are typically 25mm and 50mm.
W	Wind pressure in storage	Modelled surface area, pressure. Applied to the panel faces in storage, with and without glazing and in both directions.
S	Superimposed dead load	Additional dead load from other panels. For example, if panels are stacked.
G	Self-weight of glazing	Modelled surface area, pressure. This is typically based on 75 kg/m^2.
L	Live load	Normally required where balconies and balustrades are used. Forces/pressures are defined in the design assumptions and wind calculations document.

P	Positive wind pressure	Modelled surface area, pressure. This may vary across different building facades.
N	Negative (suction) wind pressure	Modelled surface area, pressure.
A	Accidental load	If required, this represents an external force/pressure or altered support conditions as part additional checks.

4.3 Design load combinations

Table 4-3 and Table 4-4 list all SLS and ULS load combinations respectively with naming conventions, that are considered for each panel verification. The different age of UHPFRC is also included. For each panel those judged to be critical and to be analysed are selected from this list. Generally, each combination is associated with different support conditions, and these are detailed in each panel report. A subset of this list of loads and combinations are reproduced in each report, along with any additional combinations and adjustments required for specific checks.

Table 4-3 SLS Typical design load combinations

Combination and model IDs	Age	Description	Load case details and factors			
			ID	SLS	Dynamic Lift factor	Inclination factor
SLS DM	Early stage	Demould lift. Mould suction pressure may also be added, see (10).	D B	1.0 1.0	1.0 1.0	1.16 1.16
SLS PL		Pitching lift. In some cases, this may be the same as demoulding and, therefore, is not required.	D B	1.0 1.0	1.1 1.1	1.16 1.16
SLS FL		Factory lift	D B	1.0 1.0	1.1 1.1	1.16 1.16
SLS YS		Yard storage prior to installing windows with UHPFRC not fully hardened.	D B W	1.0 1.0 1.0		
SLS YSW	28 day	Yard storage after installing windows with fully hardened UHPFRC.	D B G W	1.0 1.0 1.0 1.0		
SLS SL		Site lift	D B G	1.0 1.0 1.0	1.3 1.3 1.3	1.16 1.16 1.16
SLS PD		Permanent dead load plus any superimposed dead load. This may be applied in stages to reflect the construction sequence in which case PD1, PD2 etc. references are used.	D B G S	1.0 1.0 1.0 1.0		
SLS PDN		Total permanent load with negative (suction) wind. Additional load combinations may be required if live load (L) is significant.	D B G S N	1.0 1.0 1.0 1.0 1.0		
SLS PDP		Total permanent load with positive (pressure) wind. Additional load combinations may be required if live load (L) is significant.	D B G S P	1.0 1.0 1.0 1.0 1.0		

Notes:

i. Load factor IDs are given in Table 4-2

ii. A subset of this list plus any additional load combinations are included in each panel report.

iii. The inclination factor is only required if lifters are not explicitly modelled.

Table 4-4 ULS Typical design load combinations

Combination and model IDs	Age	Description	Load case details and factors			
			ID	ULS	Dynamic Lift factor	Inclination factor
ULS DM	Early stage	Demould lift. Mould suction pressure may also be added, see (10).	D B	1.35 1.35	1.0 1.0	1.16 1.16
ULS PL		Pitching lift. In some cases, this may be the same as demoulding and, therefore, is not required.	D B	1.35 1.35	1.1 1.1	1.16 1.16
ULS FL		Factory lift	D B	1.35 1.35	1.1 1.1	1.16 1.16
ULS YS		Yard storage prior to installing windows with UHPFRC not fully hardened.	D B W	1.35 1.35 1.5		
ULS YSW	28 day	Yard storage after installing windows with fully hardened UHPFRC.	D B G W	1.35 1.35 1.35 1.5		
ULS SL		Site lift	D B G	1.35 1.35 1.35	1.3 1.3 1.3	1.16 1.16 1.16
ULS PD		Permanent dead load plus any superimposed dead load. This may be applied in stages to reflect the construction sequence in which case PD1, PD2 etc. references are used.	D B G S	1.35 1.35 1.35 1.35		
ULS PDN		Total permanent load with negative (suction) wind. Additional load combinations may be required if live load (L) is significant.	D B G S N	1.35 1.35 1.35 1.35 1.5		
ULS PDP		Total permanent load with positive (pressure) wind. Additional load combinations may be required if live load (L) is significant.	D B G S P	1.35 1.35 1.35 1.35 1.5		
ULS A		As PDN or PDP but with reduced wind loads. The accident may be an external load or the result of altered boundary conditions. Additional load combinations may be required if live load (L) is significant.	D B G S N A	1.0 1.0 1.0 1.0 0.2 1.0		

Notes:

i. Load factor IDs are given in Table 4-2
ii. A subset of this list plus any additional load combinations are included in each panel report.
iii. The inclination lift factor is only required if lifters are not explicitly modelled.

4.4 Utilisation checks

Utilisation checks are focused on UHPFRC tensile behaviour. The results for demoulding, factory and site lifting, and permanent conditions are summarised in separate reports for each panel type using contoured and enveloped utilisation plots. Appendices are also produced as detailed in Section 4.5 that provide all supporting result sets including enveloped crack widths, principal plastic strains and principal stresses. given for each panel and reported separately using contour diagrams and summary tables.

Crack width envelopes are compared with limit state limits. The following methodology is used to determine crack widths from the analysis results.

Total strain = Elastic strain + Smeared cracking strain

As a plasticity material model is used plastic strain is used to be equivalent to smeared cracking strain, therefore:

Smeared cracking strain = plastic strain = crack width/crack band width = w/L_c

Where L_c is the crack band width and distance between cracks.

Hence, w = plastic strain x L_c

NF-P18-470 (5) defines Lc as follows. Plastic strain results, in all principal directions, are readily available from the analyses.

strain into a crack opening, to satisfy the inequality ensuring a sufficiently strain hardening behaviour. For this transformation, it is conventionally accepted that the crack opening is equal to the strain multiplied by the minimum of 2/3 the thickness e of the member and twice the length L_f of the longest fibres contributing to non-brittleness.

Relating Lc to section depth appears to be based on simple beam bending and not detailed geometry and 3D stress analysis. Also, it means different values are required where depths (thicknesses) are different. Basing Lc on L_f is more realistic when plastic strain is being calculated directly.

Crack band width must principally relate to average plastic strain (for plasticity material modelling to represent smeared cracking), material tensile strength, and fibre orientation and length.

From G200MAF (6,7) L_f = 14 mm

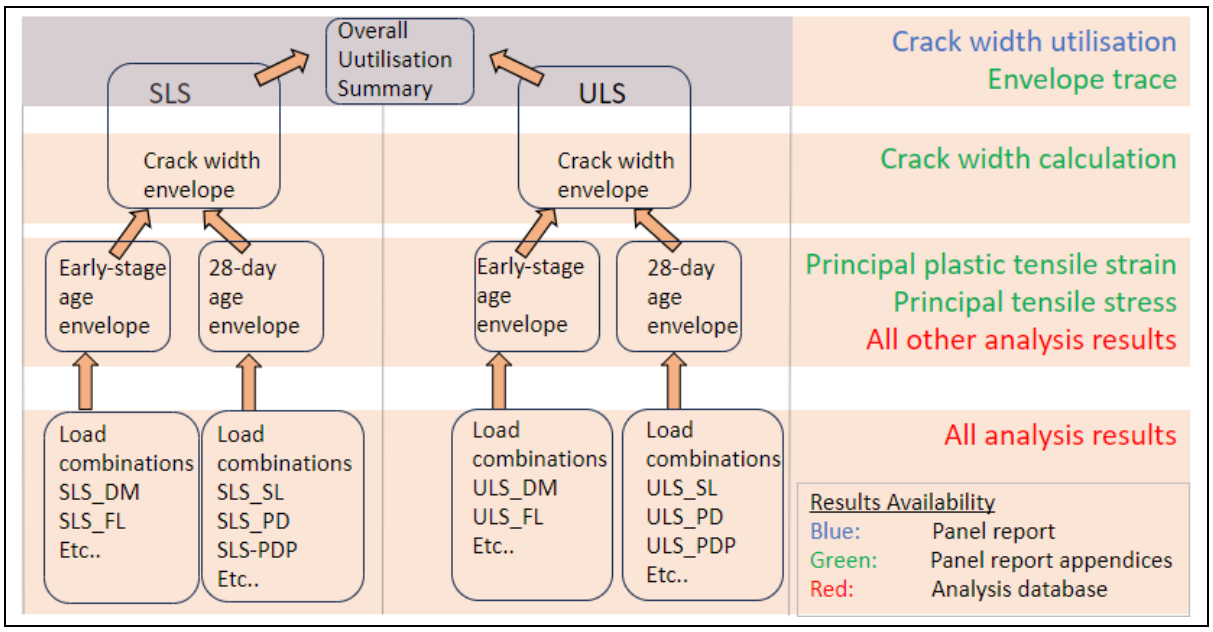
Hence, Lc = 2 x L_f = 2 x 14 = 28mm.

As this is also very dependent on environmental and manufacturing factors Lc = 75mm would seem to be a reasonable value to use. Anything larger would be more conservative.

4.5 Results

Results from the analysis of panels are included in the panel reports and their appendices as shown in Table 4-5. Additional results for all load combinations are held in the analysis database.

Table 4-5 Reported results



4.6 References

1. Thorp - UHPFRC Technical Datasheet G200MAF V2 for 28-day mechanical properties of UHPFRC, June 2022

2. Thorp – Thorp UHPFRC Technical Datasheet G200MAF 18-hour V2 July 2022

3. French Standard NF P18-710: National Addition to Eurocode 2 – Design of Concrete Structures – Specific Rules for Ultra High-Performance Fibre Reinforced Concrete. (April 2016)

APPENDIX B: Wind load calculations in accordance with BS EN 1991-1-4:2005

The following VMU wind load calculation covers storage at Thorp HQ and on the UBS site (if required).

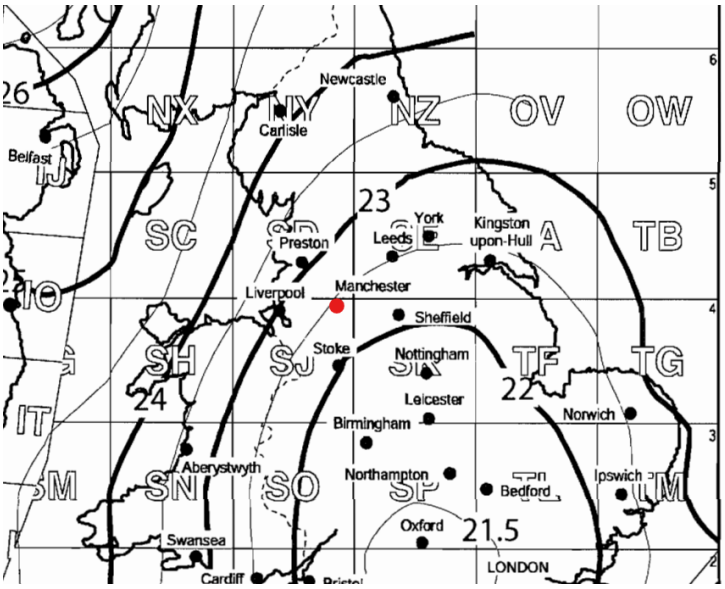


Figure 1 - Site location from Figure NA 1(worst-case between Stoke and Manchester)

The uncorrected basic wind velocity, $v_{b_map} = 22.5\ ms^{-1}$

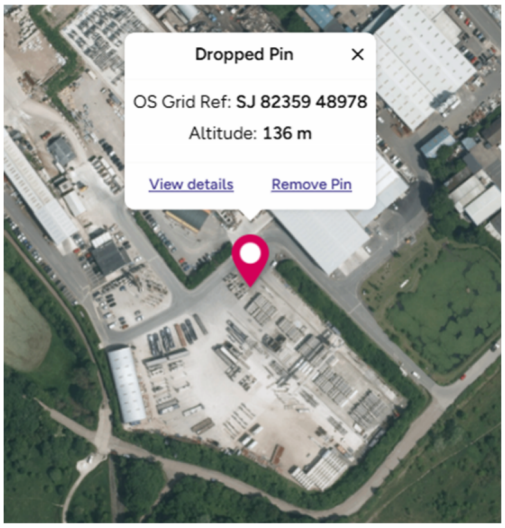


Figure 2 – Thorp HQ Site location from OS map (worst-case altitude for UBS or Thorp sites)

Examining the local OS map, the site (dropped pin) is at an altitude, $A = 136\ m$

The height of the panel with frame, $h = 3.39\ m$

The length of the panel = 6.514m

As defined in BS EN 1991-1-4:2005 clause 7.4.1 (2), $z = h = 3.39\ m$ (free-standing wall)

From Equation NA.2b, the altitude factor, $C_{alt} = 1 + 0.001 \cdot A \cdot \left(\frac{10}{z}\right)^{0.2}$ =1.169

And from NA.2.18, the air density will be taken as, $\rho = 1.226\ kgm^{-3}$

From Equation NA.1, the fundamental basic wind velocity, $v_{b_0} = v_{b_map} \cdot C_{alt} = 26.3\ ms^{-1}$

For temporary structure, $C_{season} = 1$ (Can not prove the VMU will remain for less than a whole year)

The specified life of the structure is 5 years, so the recurrence period will be taken as, $R = 5\ yr$

This gives an annual risk of exceedance, $p = 1 - e^{-\frac{1\ yr}{R}} = 0.181$

From Equation 4.2, this gives a probability factor, $C_{prob} = \sqrt{\frac{(1-0.2\cdot\ln(-\ln(1-p)))}{(1-0.2\cdot\ln(-\ln(0.98)))}} = 0.862$

From Table NA.1:

Table NA.1 **Directional factor c_{dir}**

Direction	0°	30°	60°	90°	120°	150°	180°	210°	240°	270°	300°	330°
c_{dir}	0,78	0,73	0,73	0,74	0,73	0,80	0,85	0,93	1,00	0,99	0,91	0,82

NOTE 1 *Interpolation may be used within Table NA.1.*

NOTE 2 *The directions are defined by angles from due North in a clockwise direction.*

NOTE 3 *Where the wind loading on a building is assessed only for orthogonal load cases, the maximum value of the factor for the directions that lie ± 45° either side of the normal to the face of the building is to be used.*

NOTE 4 *Conservatively, c_{dir} may be taken as 1,0 for all directions.*

Figure 3 - Table NA.1

The direction factor, $C_{dir} = 1.0$

From equation 4.1, the basic wind velocity, $V_b = C_{dir} \cdot C_{season} \cdot V_{b_o} \cdot C_{prob} = 22.67\ ms^{-1}$

From equation 4.10, the basic velocity pressure, $q_b = \frac{1}{2} \cdot \rho \cdot V_b^2 = 315.0\ Pa$

Determination of peak velocity pressure

The height of the highest panel on the building above the ground, $z = 3.39\ m$

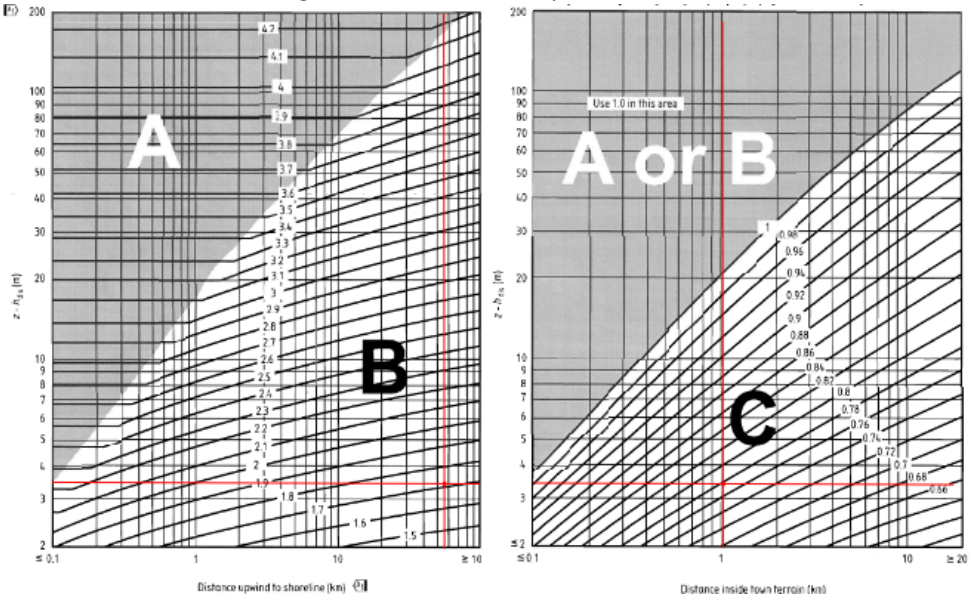
Conservatively assuming that the building around the site provide little shelter, the reference height will be, $h_{ref} = z = 3.39\ m$

For sites in town terrains, NA.3b allows the peak pressure to be calculated using,

$$q_p(z) = C_e(z) \cdot C_{e.T} \cdot q_b$$

Either site will be at least 1km to the nearest town edge and 57km from the nearest shoreline from the west.

Figures NA.7 and NA.8 then gives the factor for equation (NA.3b),



From the figures $C_e(z) = 1.72$ and $C_{e.T} = 0.78$

So that the peak velocity pressure, $q_p(z) = C_e(z) \cdot C_{e.T} \cdot q_b = 422.6\ Pa$

Even though the panel geometry is with a return corner, the return corner is less than the height and thus conservatively without return corners will be used.

The zones will be A, B and C with l < 4h but the structural design will assume Zone A loads throughout

Table 7.9 — Recommended pressure coefficients $c_{p,net}$ for free-standing walls and parapets

Solidity	Zone		A	B	C	D
$\varphi = 1$	Without return corners	$\ell/h \le 3$	2,3	1,4	1,2	1,2
		$\ell/h = 5$	2,9	1,8	1,4	1,2
		$\ell/h \ge 10$	3,4	2,1	1,7	1,2
	with return corners of length $\ge h$ [a]		2,1	1,8	1,4	1,2
$\varphi = 0,8$			1,2	1,2	1,2	1,2

[a] Linear interpolation may be used for return corner lengths between 0,0 and h

So that the max net pressure, $W = 2.3 \cdot q_p(z) \cdot 10^{-3} = 0.971\ kPa$

Appendix C: Supplementary FE analysis results and plots

Project Name	Upper Brook Street
Thorp Project ID	24115
Design Unit ID	VMU Panel
Document Author	Gayan Wijesundara
Date	06/02/2025

This document contains the supplementary plots (displacements, principal tensile stresses/strains, etc) produced using FE analysis for VMU. These also include the plastic strain plots used to derive the crack utilisation envelope plots that are provided in Section 8 of the design report.

Figure 1 shows a summary of different result sets produced as part of this design verification. For clarity, Figure 1 also shows where each type of results has been presented. In summary, this will help to understand how the final crack width utilisation envelopes have been derived using the results obtained for different load cases.

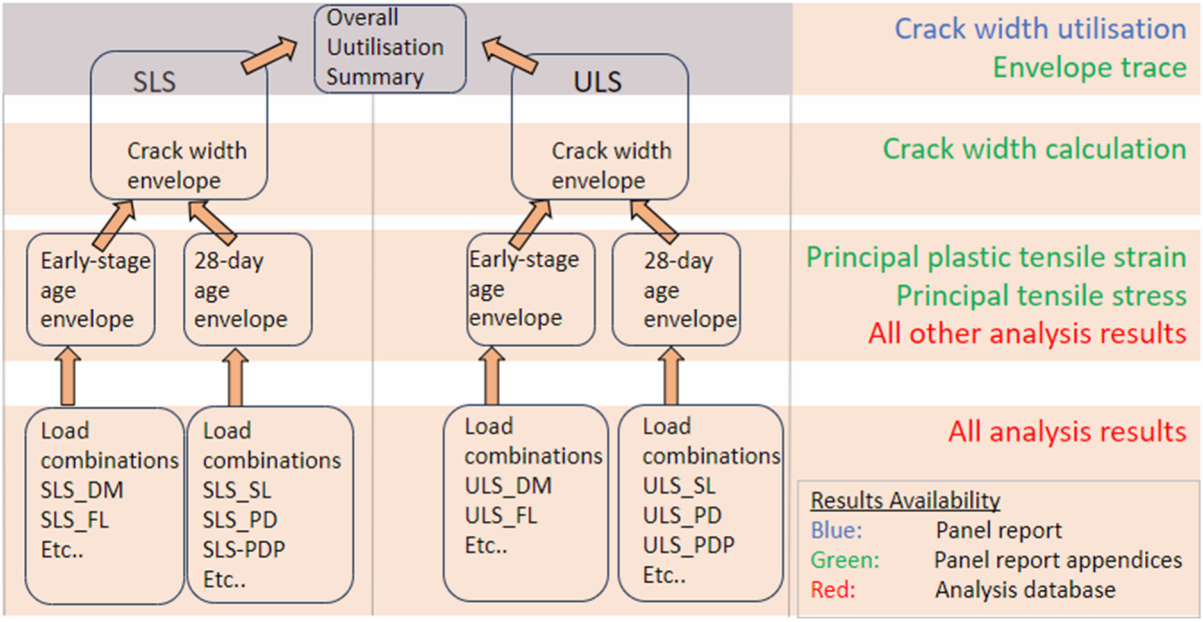


Figure 1 : Envelope_trace_plot

Figure 2 & Figure 3 show crack width envelope trace plots for front and rear faces of the panel respectively.

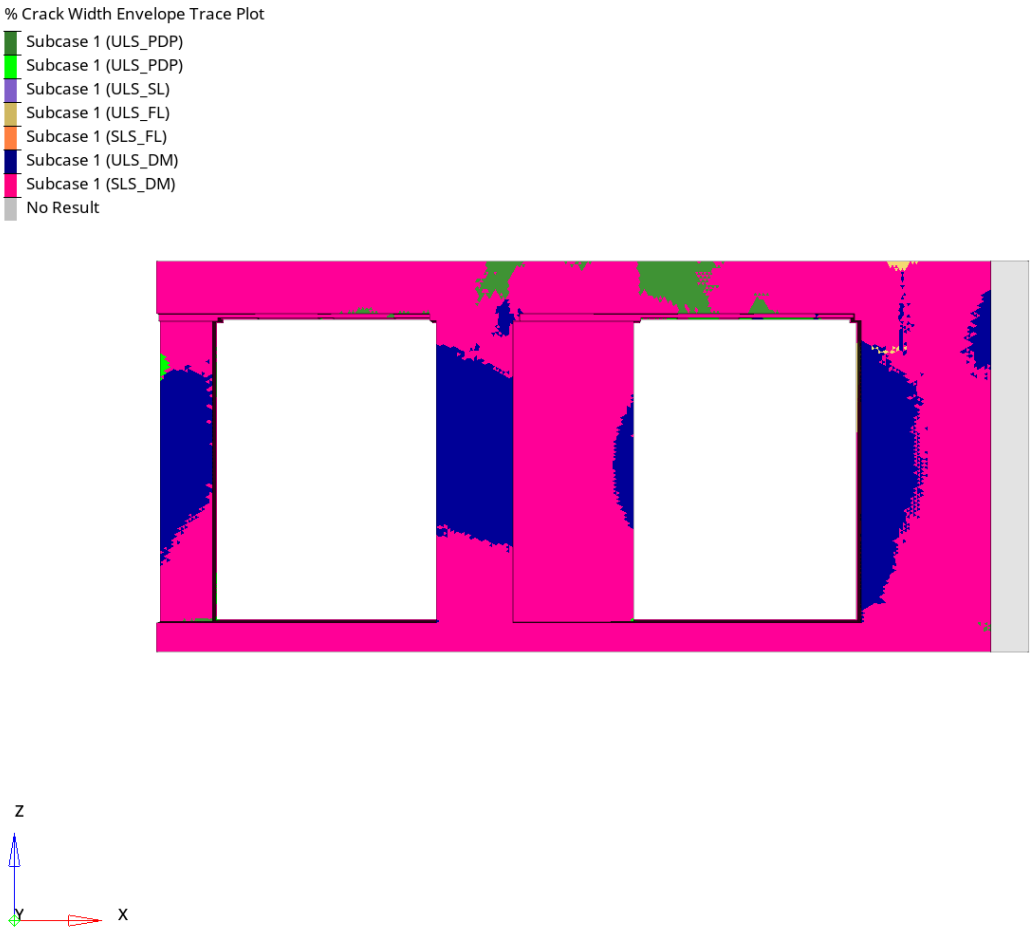


Figure 2 : Crack_width_envelope_trace_plot_front_view

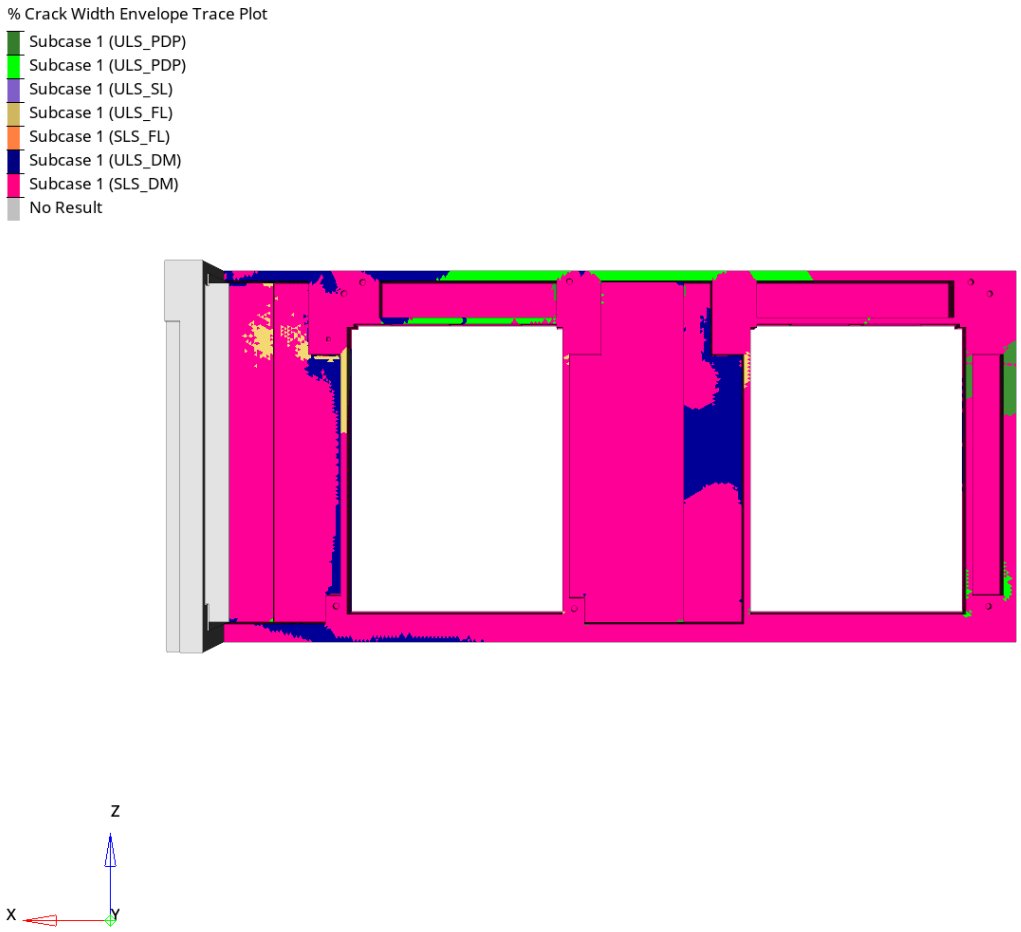


Figure 3 : Crack_width_envelope_trace_plot_rear_view

Figure 4 & Figure 5 show equivalent plastic strain envelopes under ULS storage loads for front and rear faces of the panel respectively.

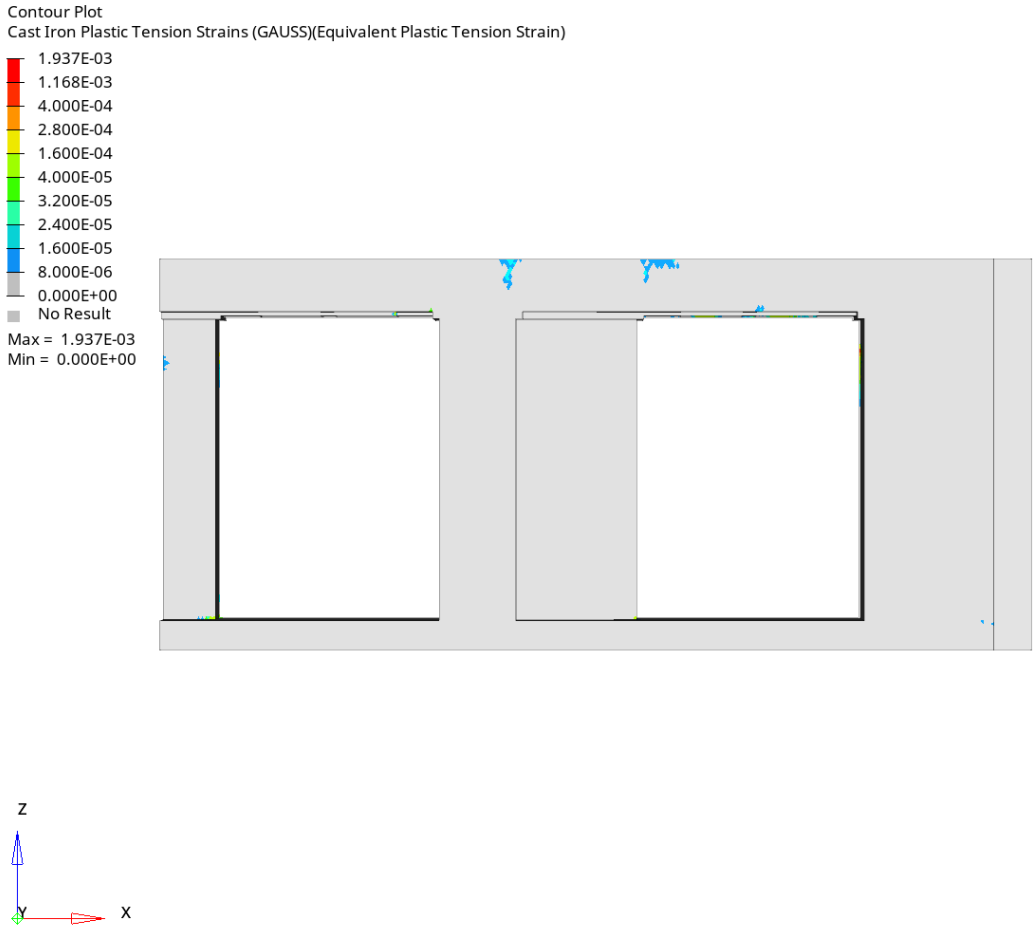


Figure 4 : Equi_Plas_strn_envelope_ULS_28_day_front_view

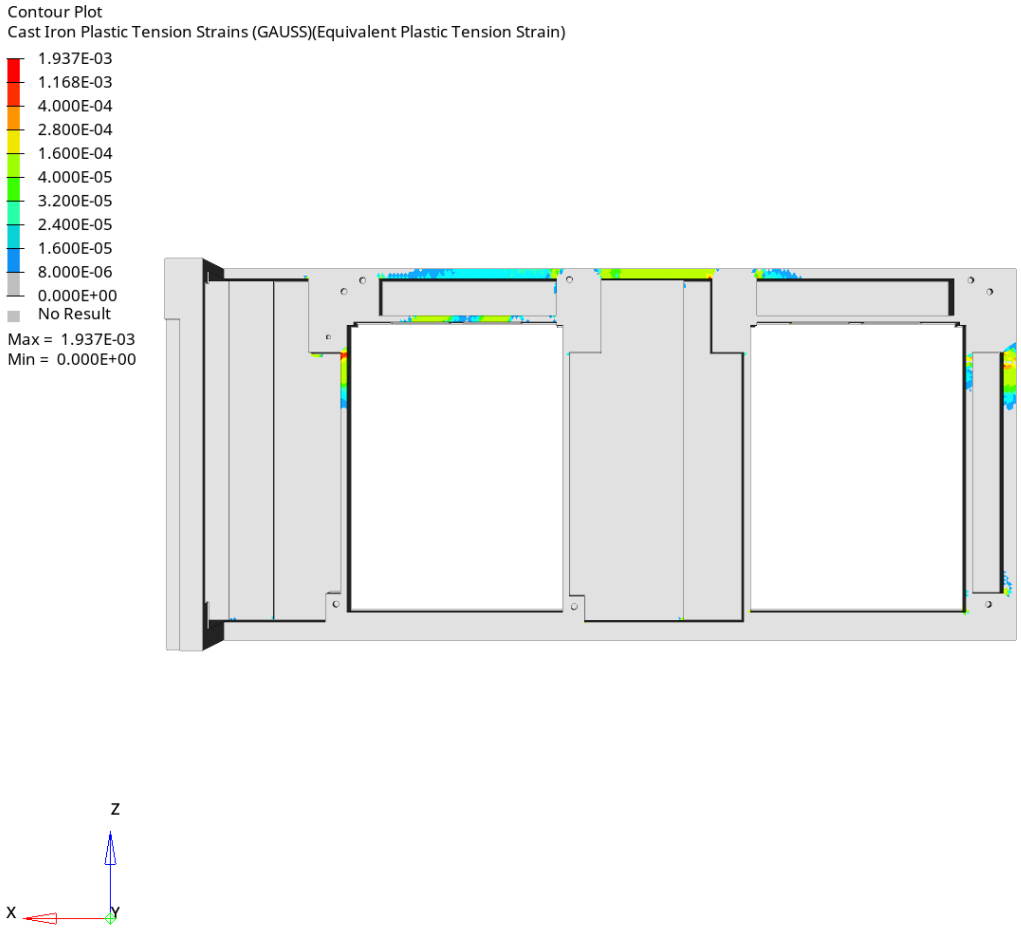


Figure 5 : Equi_Plas_strn_envelope_ULS_28_day_rear_view

Figure 6 & Figure 7 show equivalent plastic strain envelopes under ULS demoulding and factory lifting loads for front and rear faces of the panel respectively.

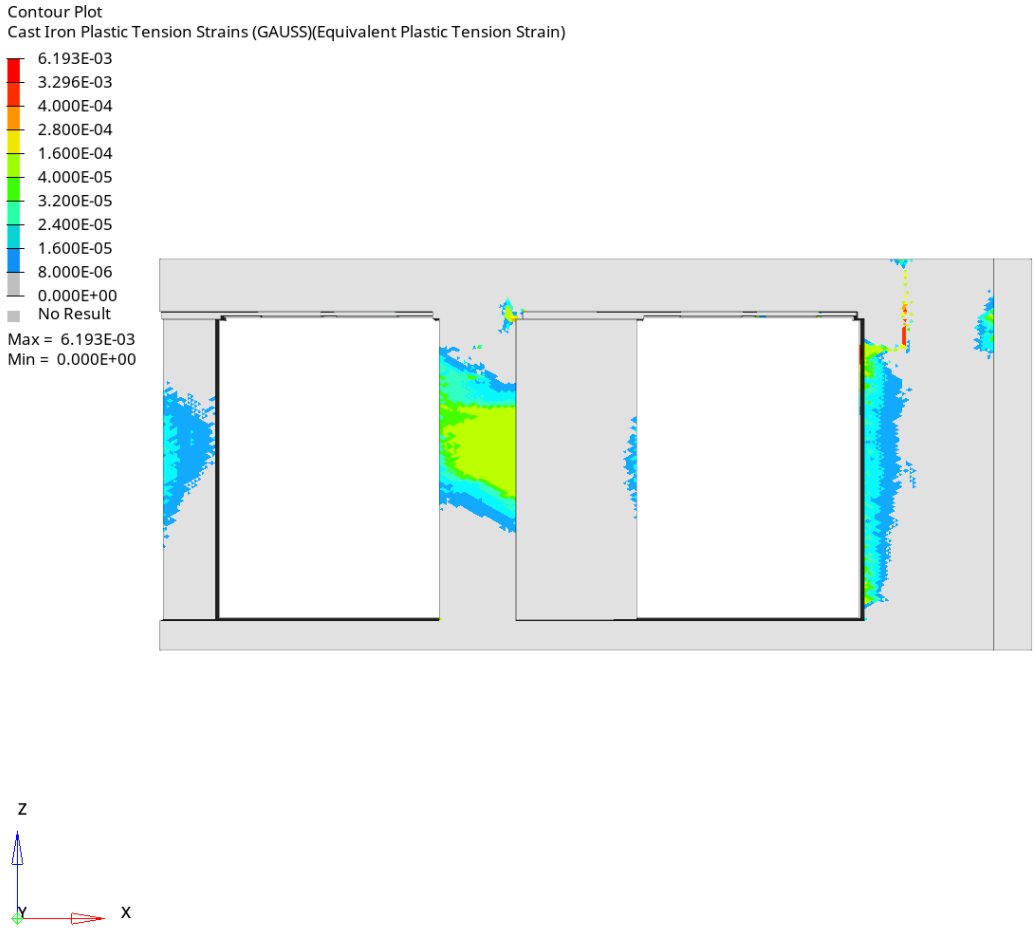


Figure 6 : Equi_Plas_strn_envelope_ULS_early_stage_front_view

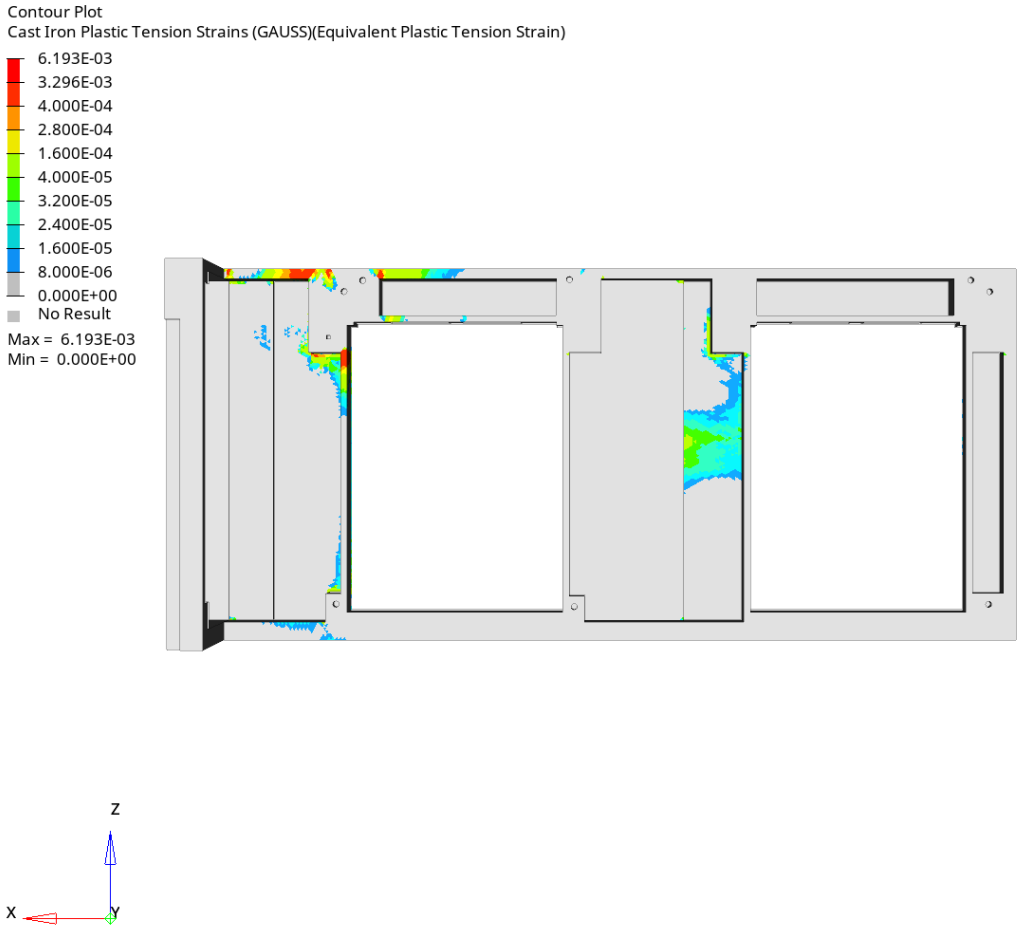


Figure 7 : Equi_Plas_strn_envelope_ULS_early_stage_rear_view

Figure 8 & Figure 9 show equivalent plastic strain envelopes under ULS storage for front and rear faces of the panel respectively.

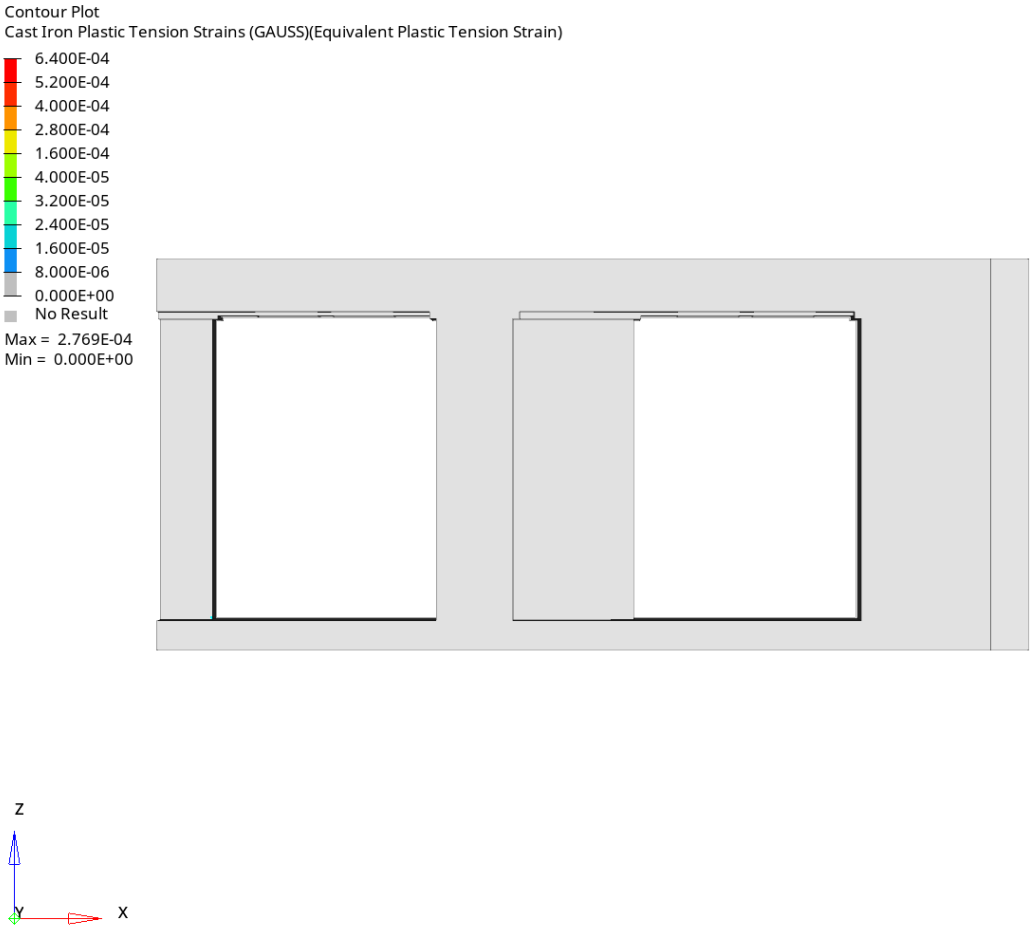


Figure 8 : Equi_Plas_strn_envelope_SLS_28_day_front_view

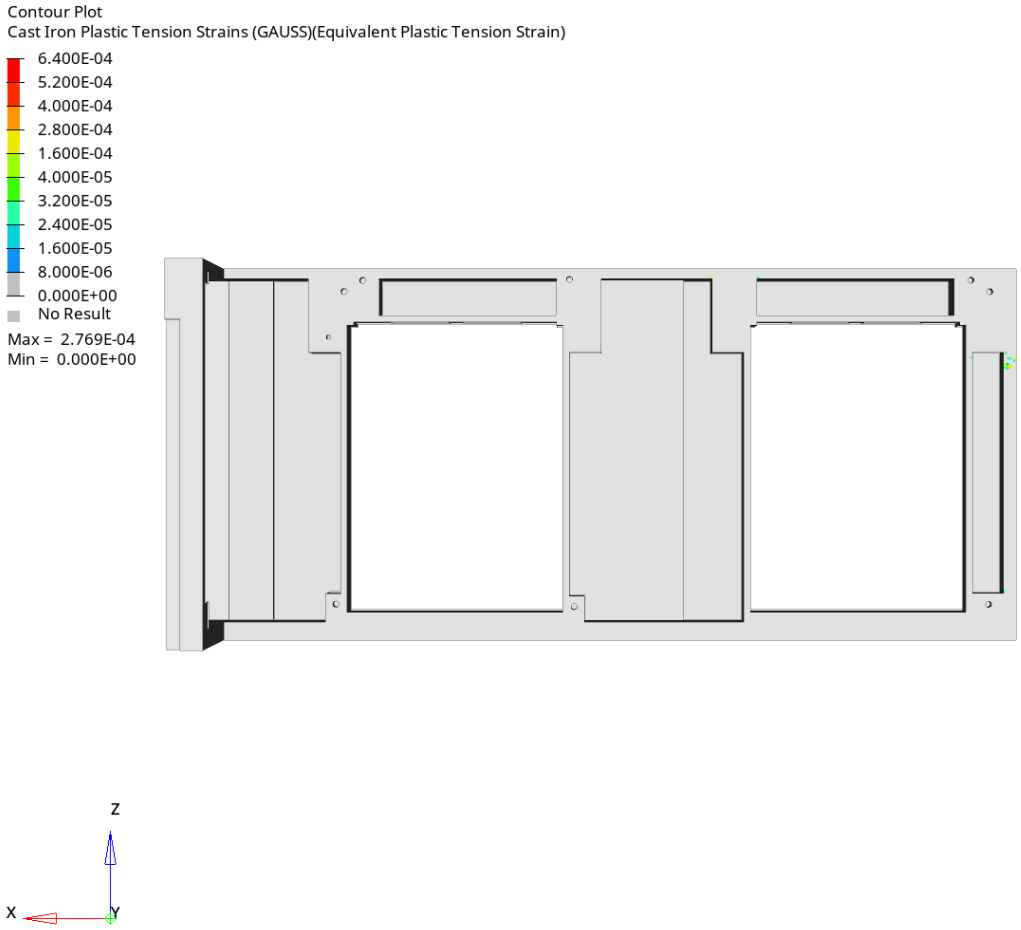


Figure 9 : Equi_Plas_strn_envelope_SLS_28_day_rear_view

Figure 10 & Figure 11 show equivalent plastic strain envelopes under ULS demoulding and factory lifting loads for front and rear faces of the panel respectively.

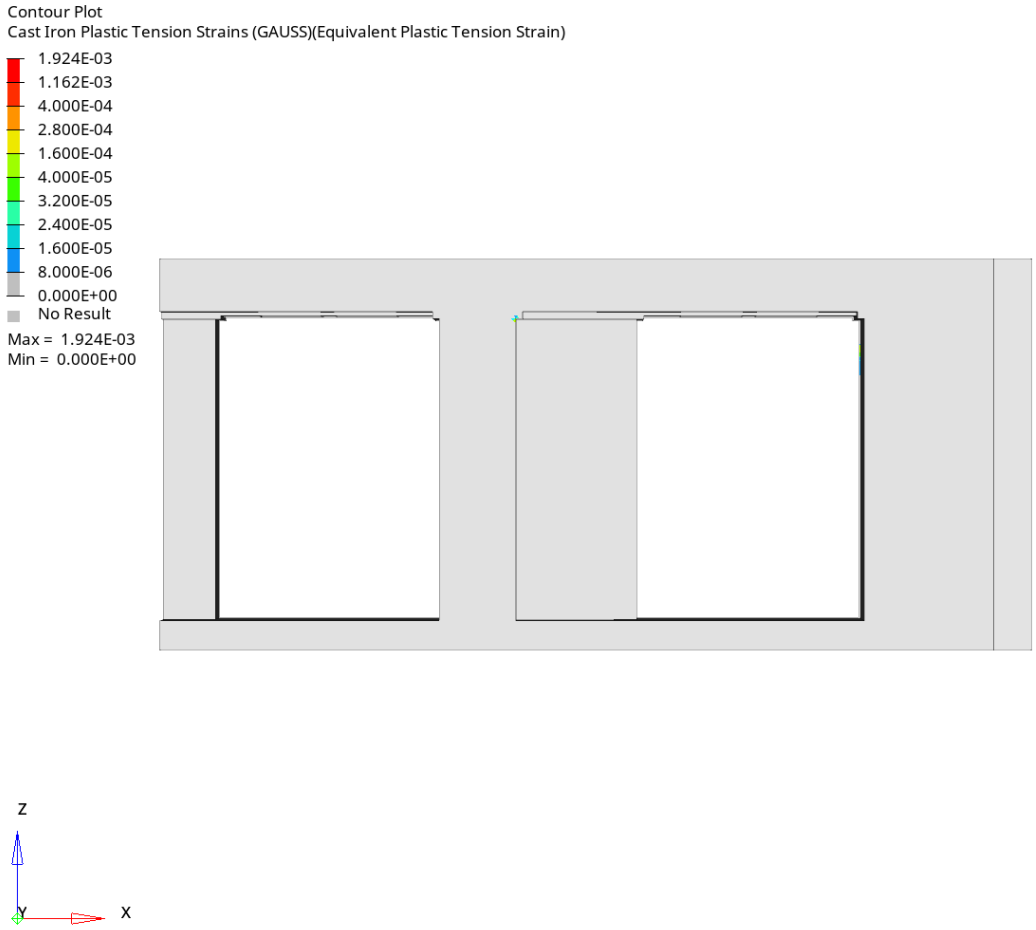


Figure 10 : Equi_Plas_strn_envelope_SLS_early_stage_front_view

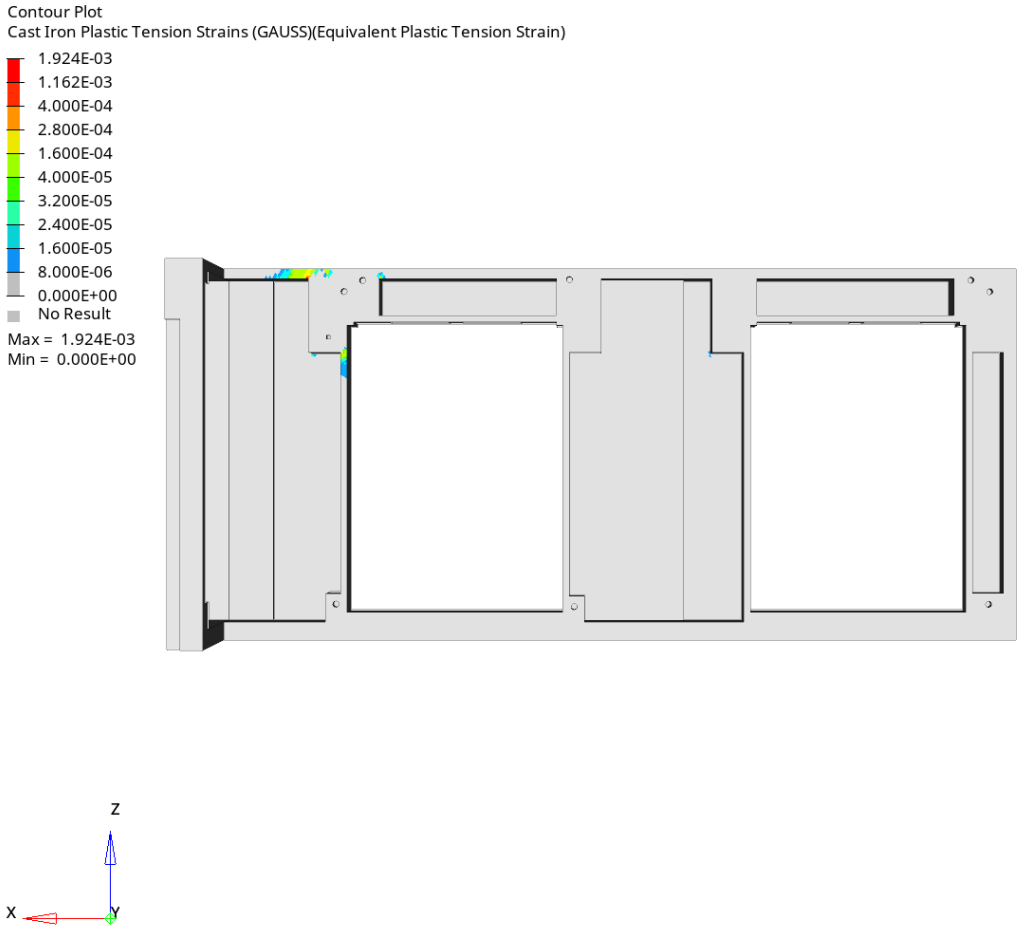


Figure 11 : Equi_Plas_strn_envelope_SLS_early_stage_rear_view

Figure 12 & Figure 13 show principal tensile stress envelopes under ULS storage loads for front and rear faces of the panel respectively.

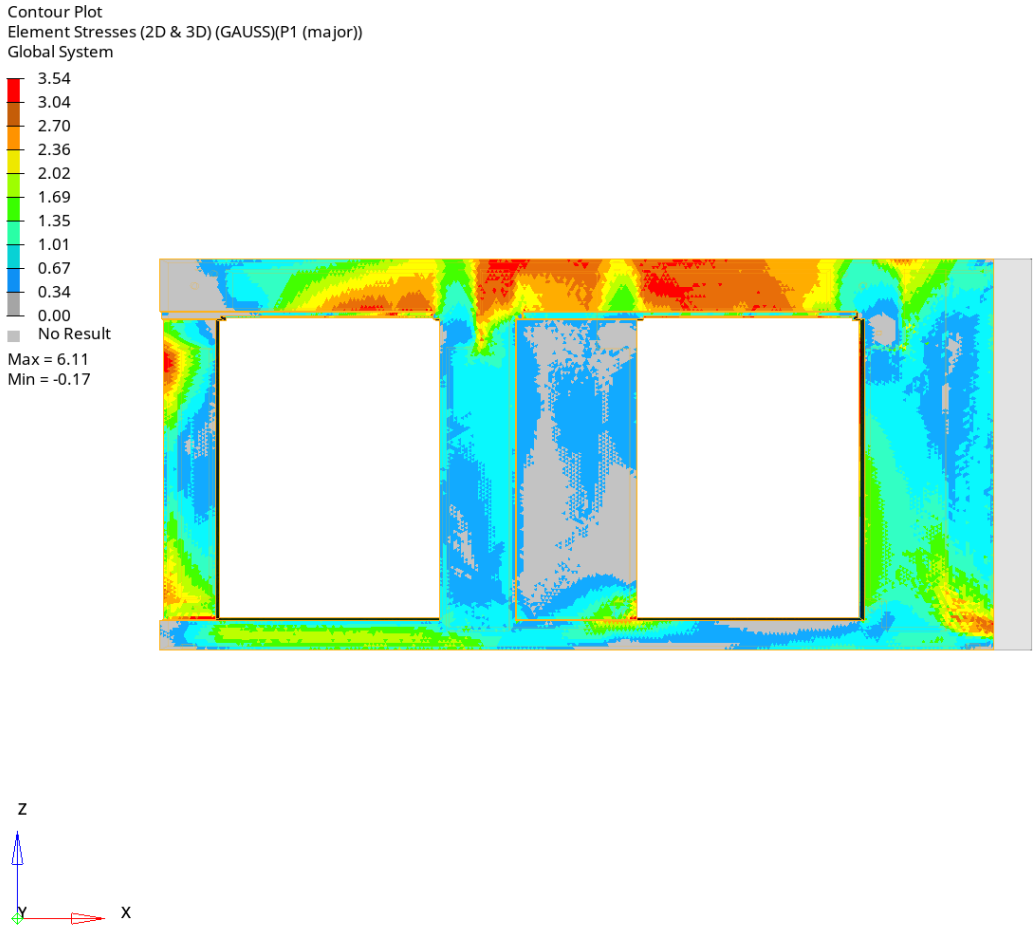


Figure 12 : P1_strs_envelope_ULS_28_day_front_view

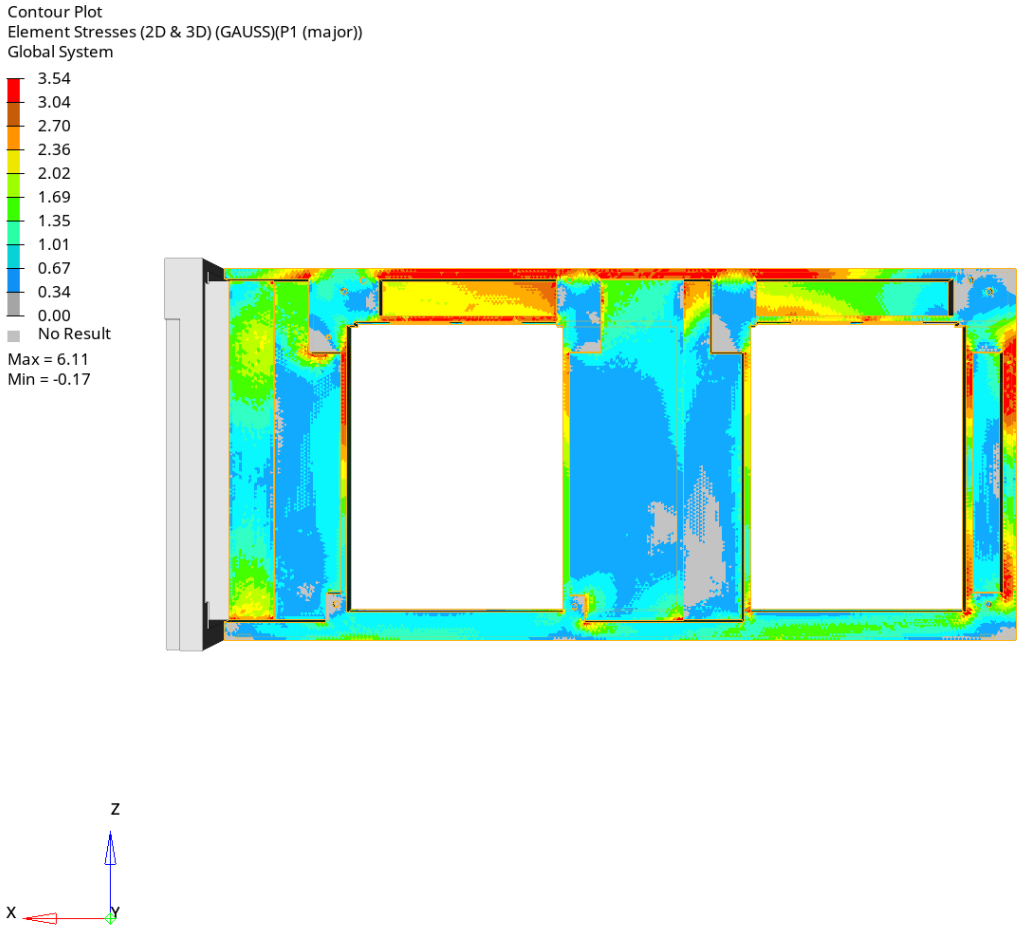


Figure 13 : P1_strs_envelope_ULS_28_day_rear_view

Figure 14 & Figure 15 show principal tensile stress envelopes under ULS demoulding and factory lifting loads for front and rear faces of the panel respectively.

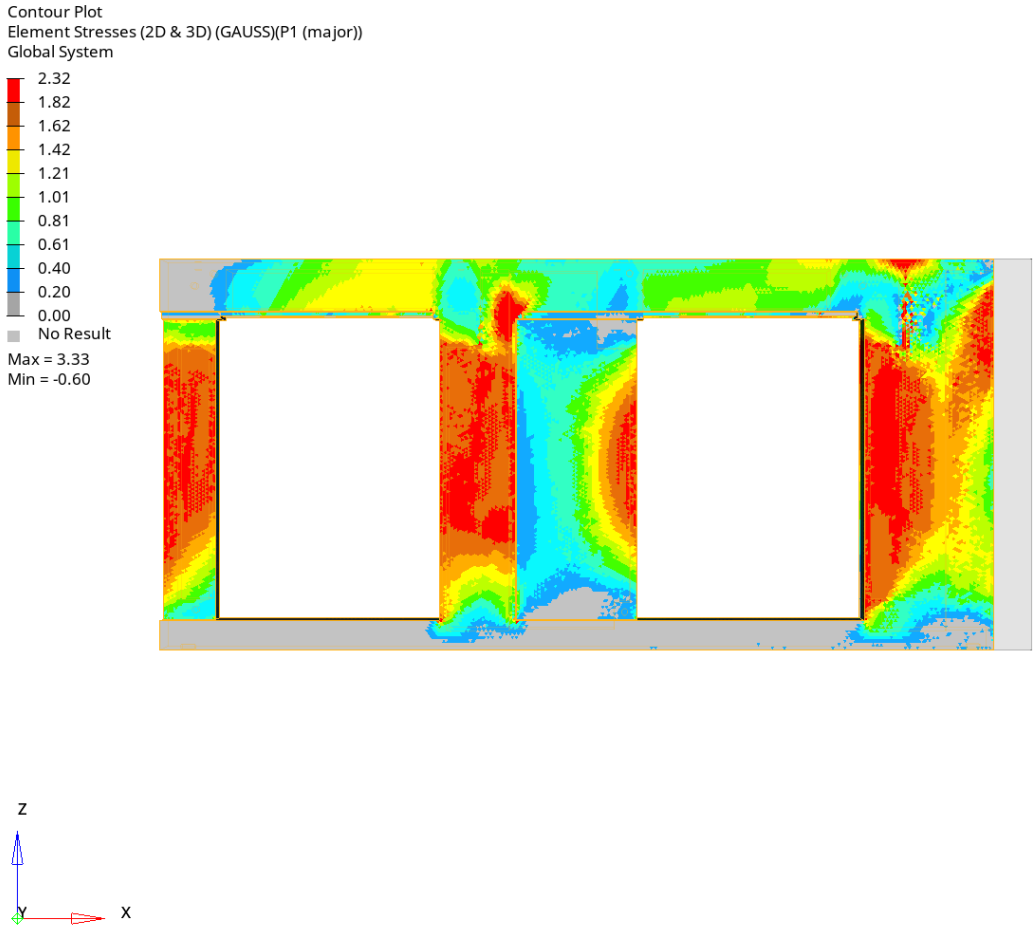


Figure 14 : P1_strs_envelope_ULS_early_stage_front_view

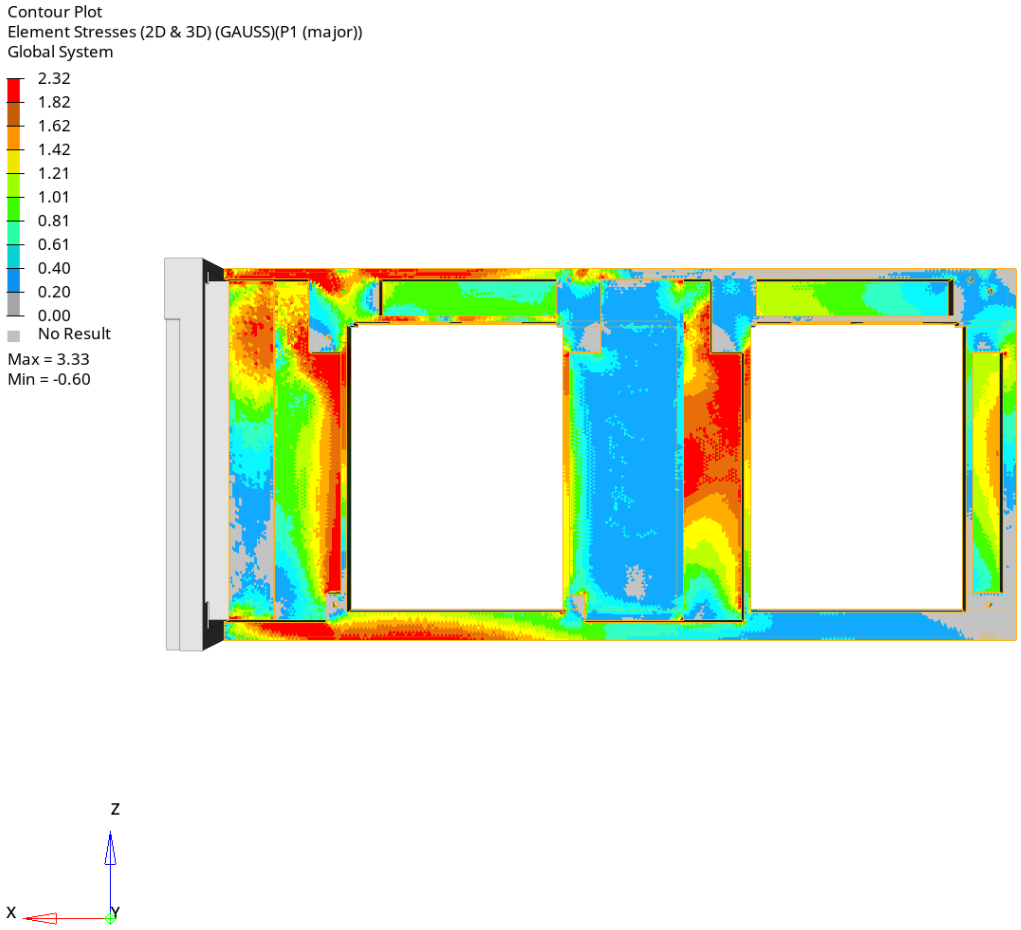


Figure 15 : P1_strs_envelope_ULS_early_stage_rear_view

Figure 16 & Figure 17 show principal tensile stress envelopes under SLS storage loads for front and rear faces of the panel respectively.

Project Name	Upper Brook Street
Thorp Project ID	24115
Design Unit ID	VMU Panel
Document Author	Gayan Wijesundara
Date	06/02/2025

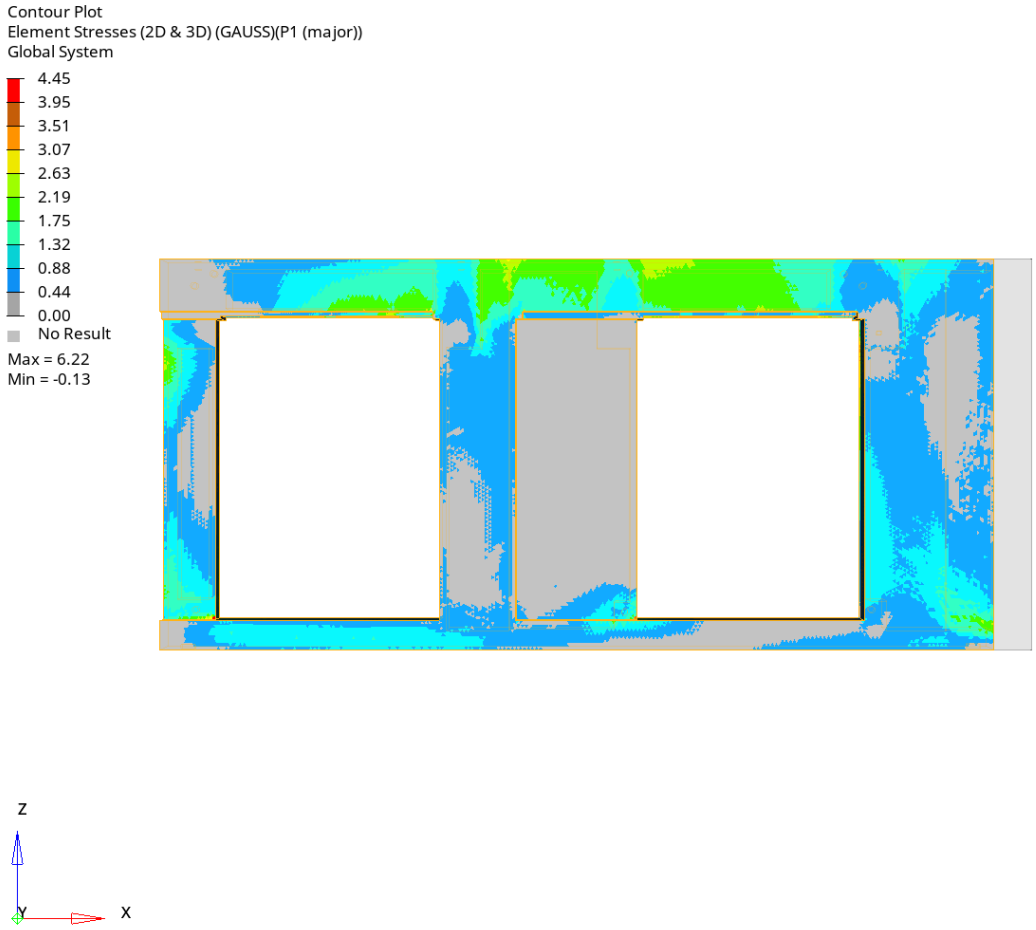


Figure 16 : P1_strs_envelope_SLS_28_day_front_view

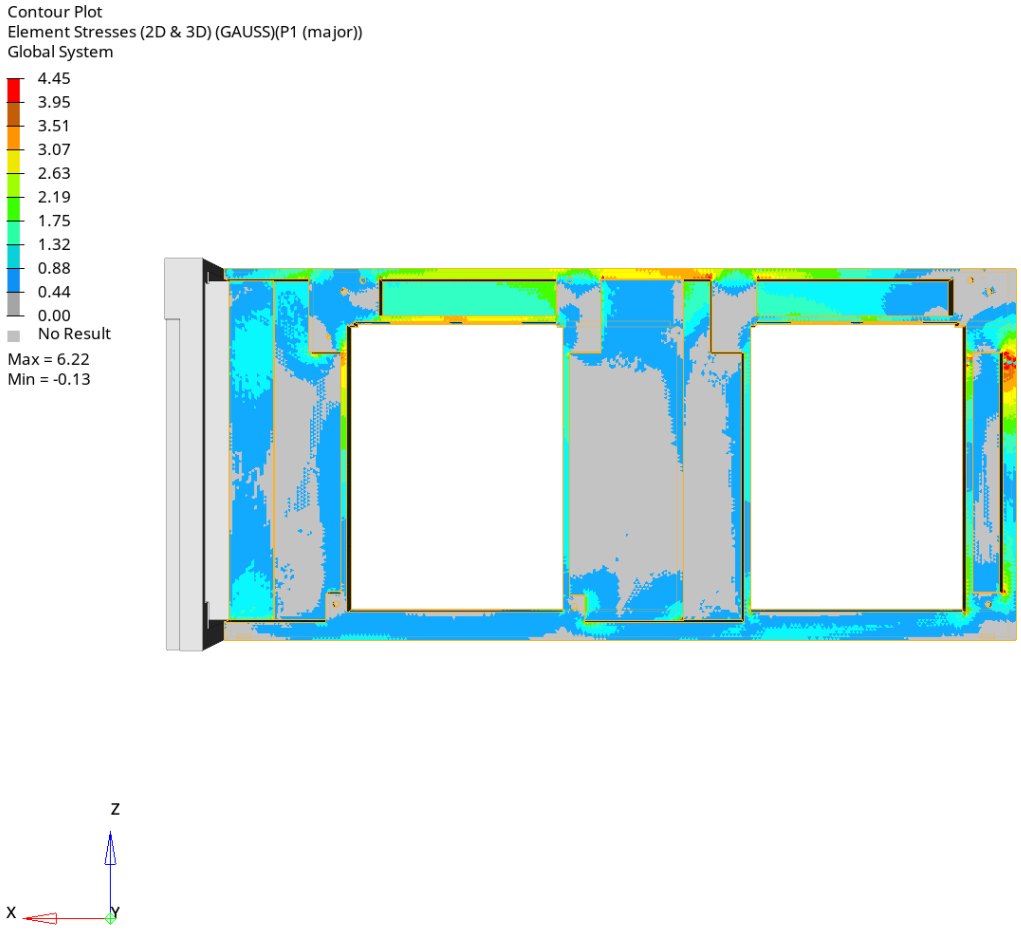


Figure 17 : P1_strs_envelope_SLS_28_day_rear_view

Figure 18 & Figure 19 show principal tensile stress envelopes under SLS demoulding and factory lifting loads for front and rear faces of the panel respectively.

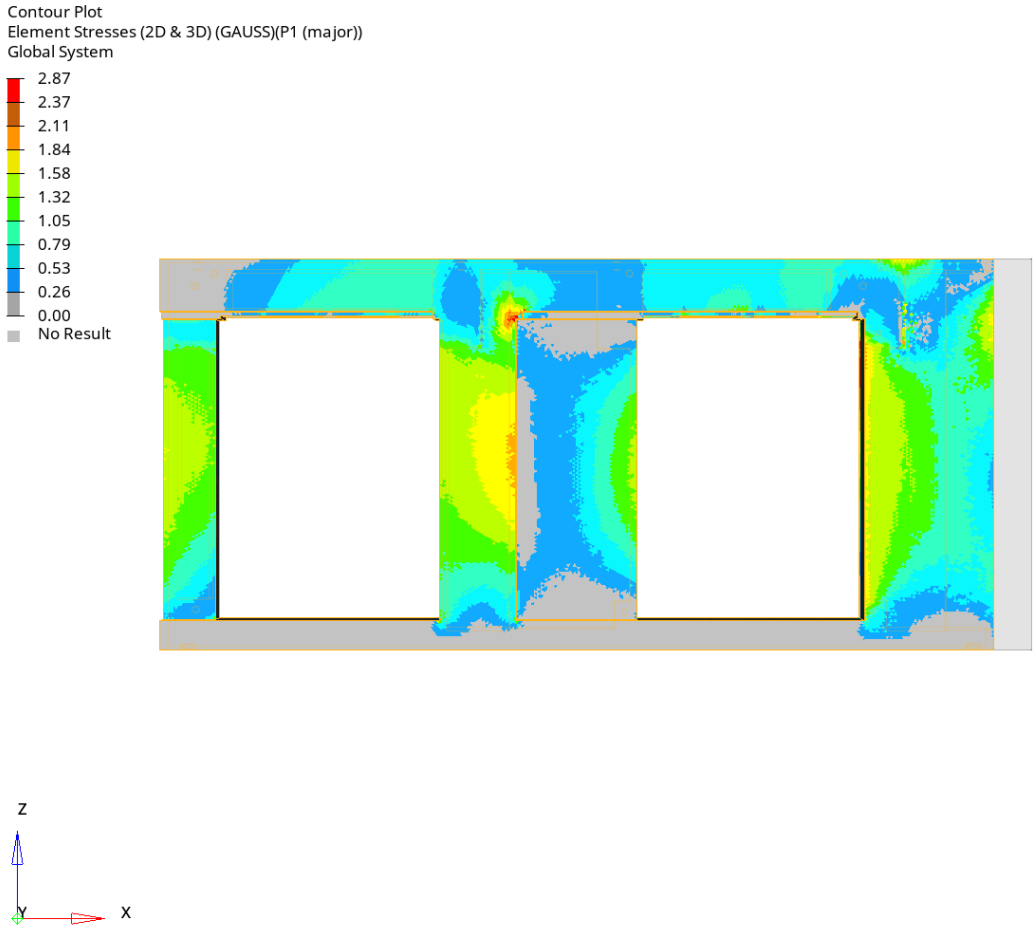


Figure 18 : P1_strs_envelope_SLS_early_stage_front_view

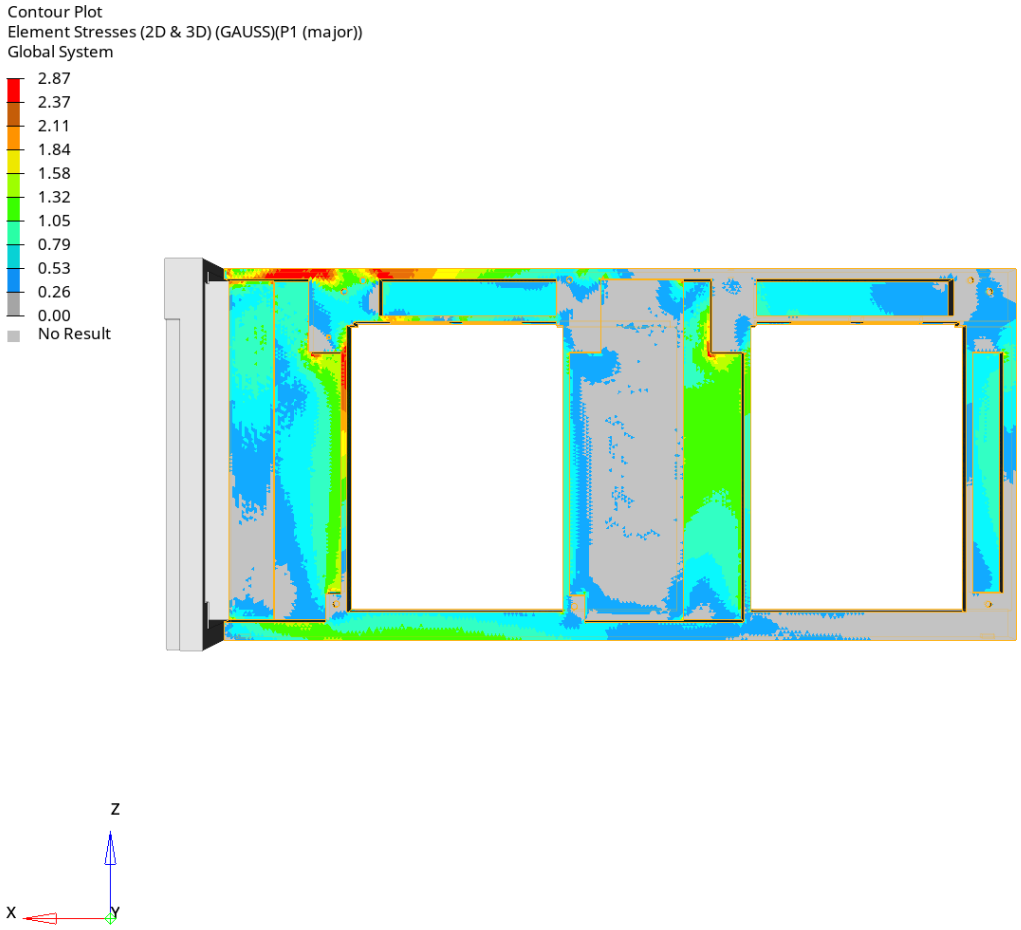


Figure 19 : P1_strs_envelope_SLS_early_stage_rear_view

Figure 20 & Figure 21 show envelope crack width plots under ULS loads for front and rear faces of the panel respectively.

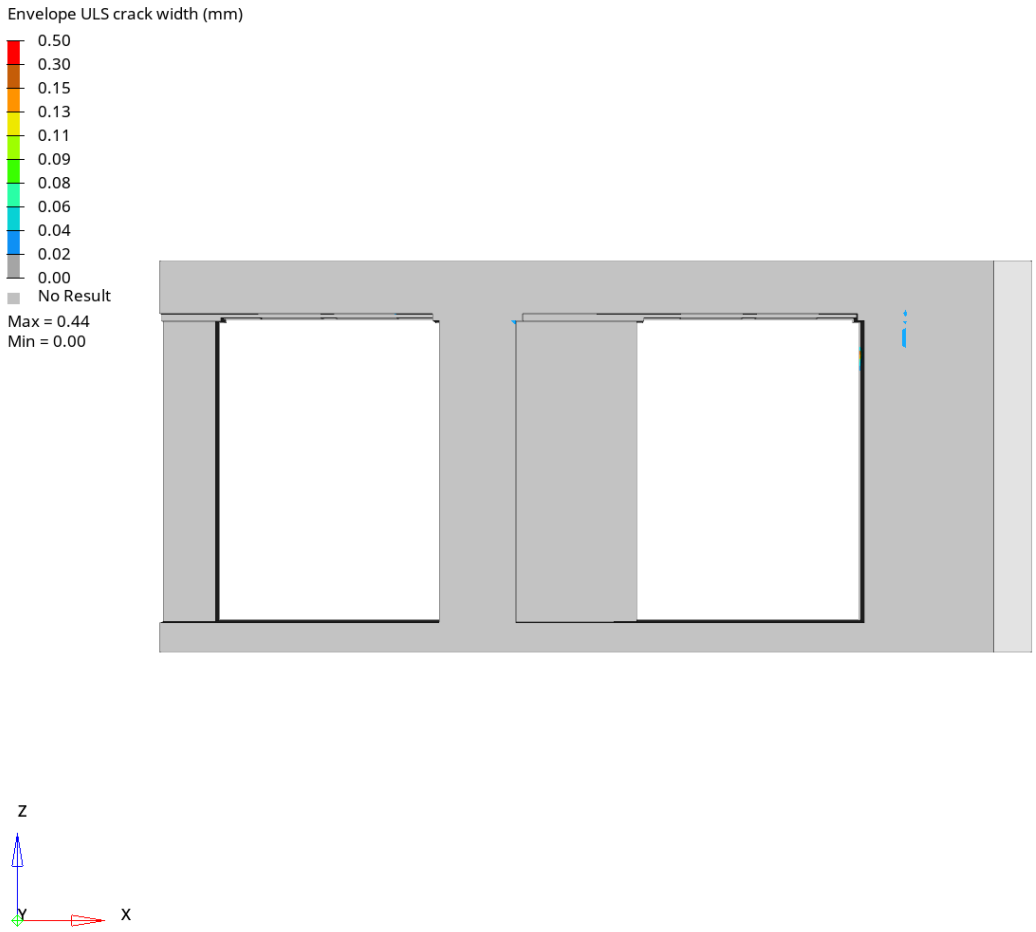


Figure 20 : ULS_envelope_crack_width_front_view

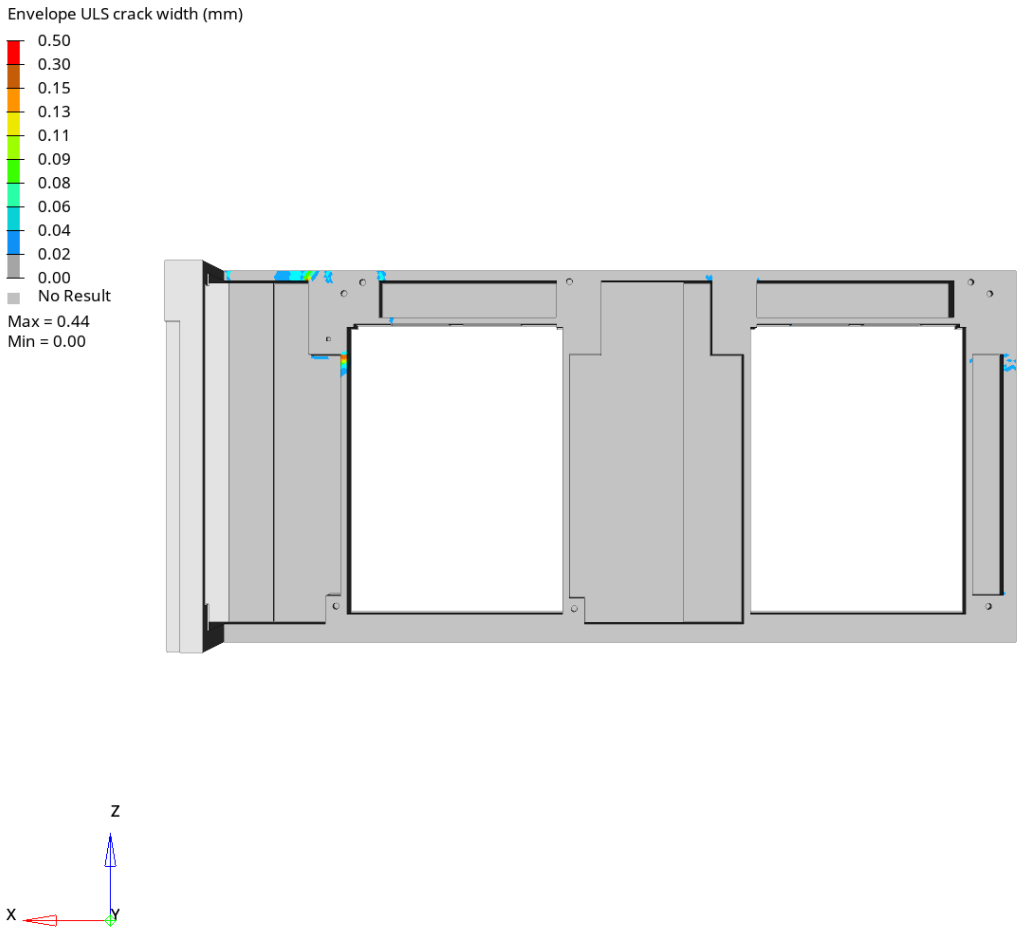


Figure 21 : ULS_envelope_crack_width_rear_view

Figure 22 & Figure 23 show envelope crack width plots under SLS loads for front and rear faces of the panel respectively.

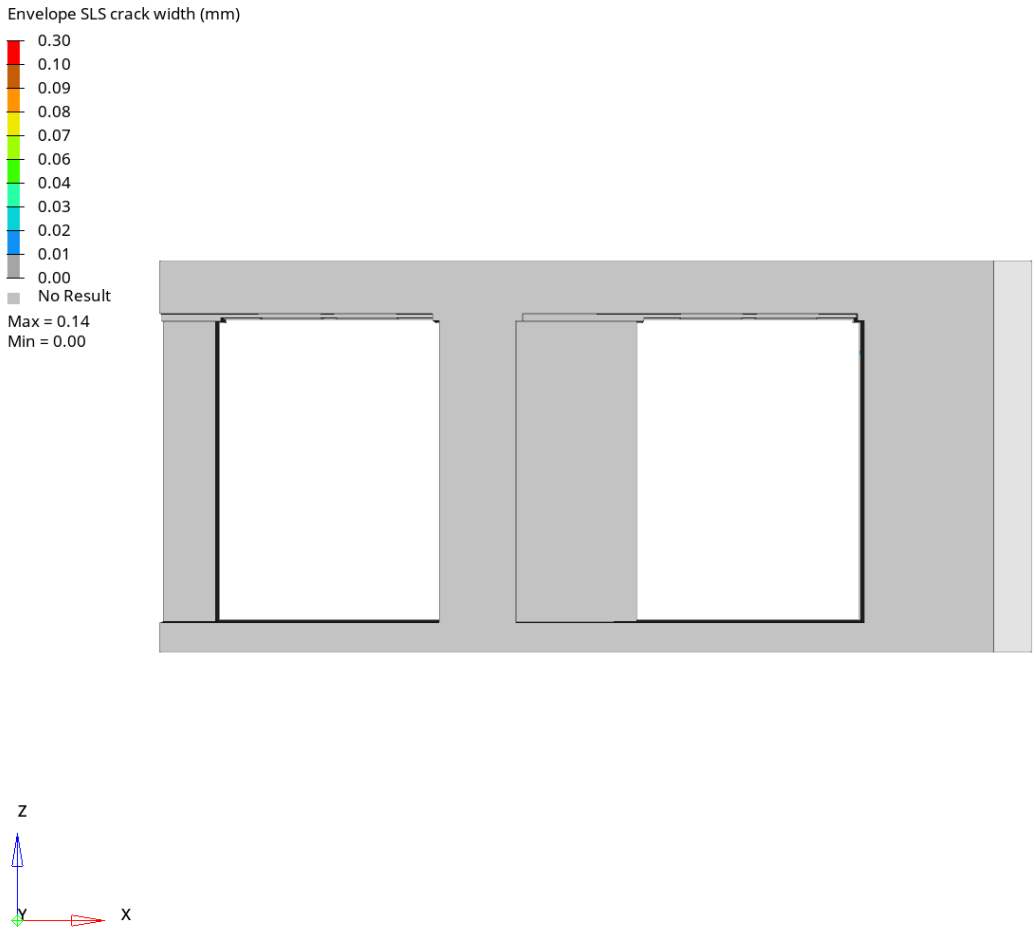


Figure 22 : SLS_envelope_crack_width_front_view

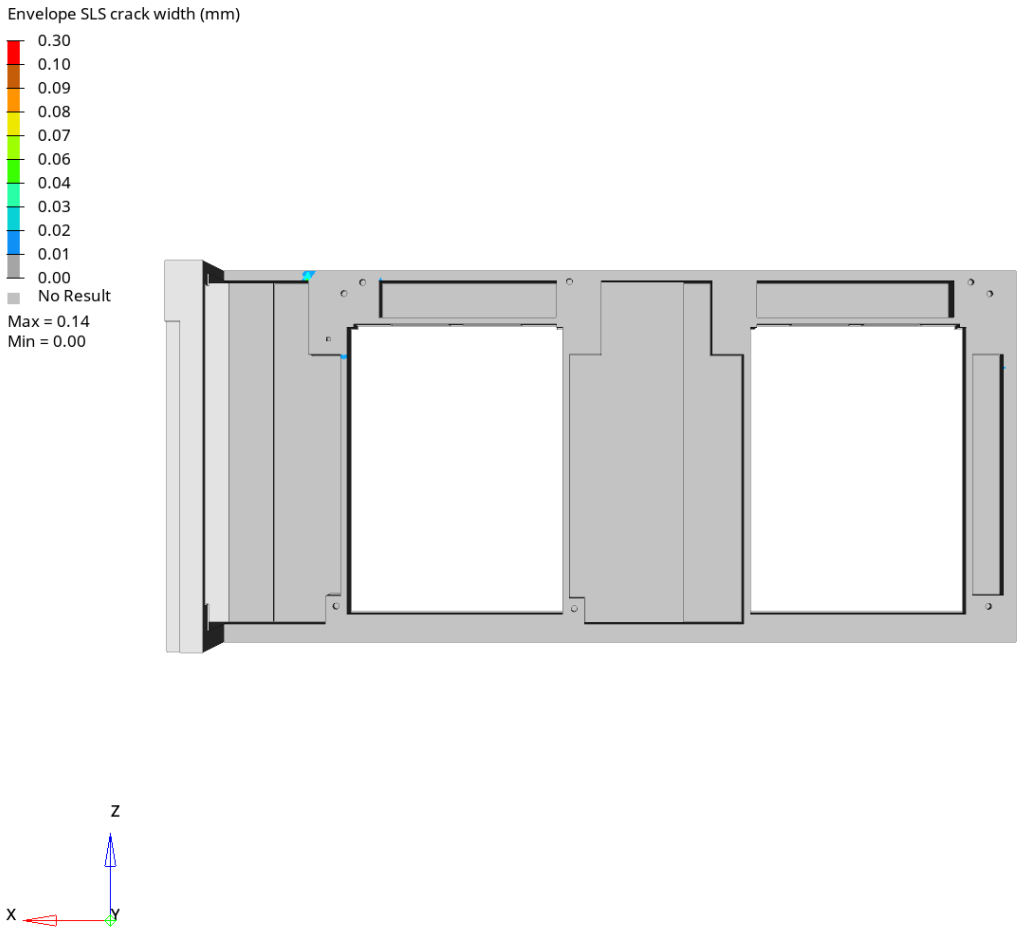


Figure 23 : SLS_envelope_crack_width_rear_view

APPENDIX D: Panel and skid stability against overturning and sliding due to wind loads

This documents provides stability calculations carried out for the VMU using wind loads determined in accordance with BS EN 1991-1-4:2005.

Project	24112 - Upper Brook Street		
Task	VMU Stability		
Made by	GWIJE		
Date	06/02/2025		
Distance between the overturning edge and		0.439	m
COGs of the Panel		0.250	m
Weight of Skid		1120	kg
Weighted averaged wind pressure		0.64	kN/m²

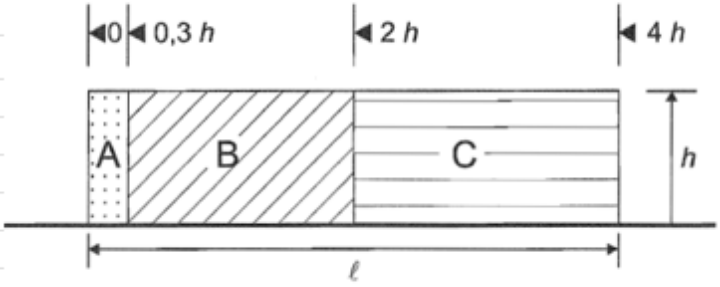
Panel ID	**INPUTS**				
	Total Panel Weight (kg)	**Total Window Weight kg)**	**Panel Length (m)**	**Panel Height (m)**	**Kentledge Weight, kg**
VMU	6625.6	785.2	6.515	3.14	2500

VMU is a free standing wall with a return corner

From Fig 7.19 of BS EN 1991-1-4:2005

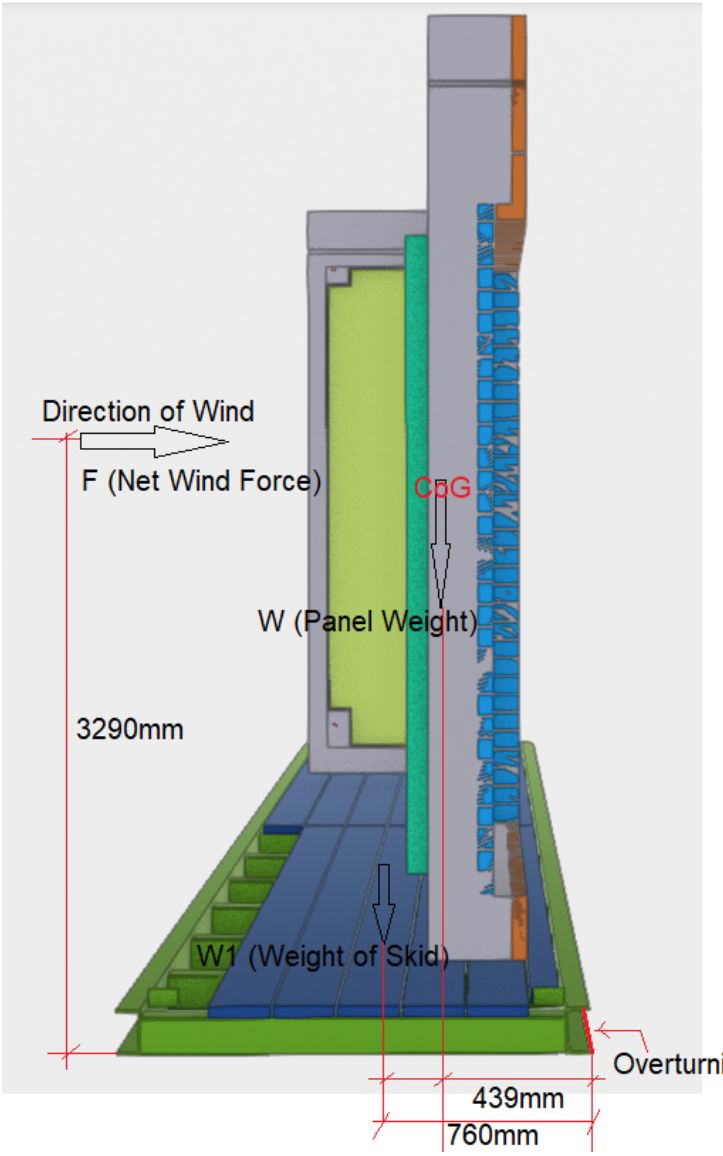
for $\ell \leq 4\,h$

Zone	Area, m²
A	2.96
B	16.76
C	0.74



From Table 7.9 of BS EN 1991-1-4:2005

Zone	$C_{p,net}$	$q_p(z)$, N - From Appendix B	Net wind pressure, kPa
A	2.30	422.6	0.97
B	1.40		0.59
C	1.20		0.51



For example, in the case of no skid inclination:

Load factor for wind	= 1.5
Load factor for panel self-weight	= 0.9 (beneficial effect)
Overturning moment	= F x 3.29 x 1.5
Resisting moment	= (W x 0.439 + W1 x 0.76) x 0.9

Overturning Moment with Window (kNm)				Overturning Moment without Window (kNm)			
No inclination	2.3 degrees	5 degrees	7.5 degrees	No inclination	2.3 degrees	5 degrees	7.5 degrees
31	31	31	31	15	15	15	15

Resisting Moment with Window (kNm)				Resisting Moment without Window (kNm)			
No inclination	2.3 degrees	5 degrees	7.5 degrees	No inclination	2.3 degrees	5 degrees	7.5 degrees
40	36	32	28	37	34	30	27

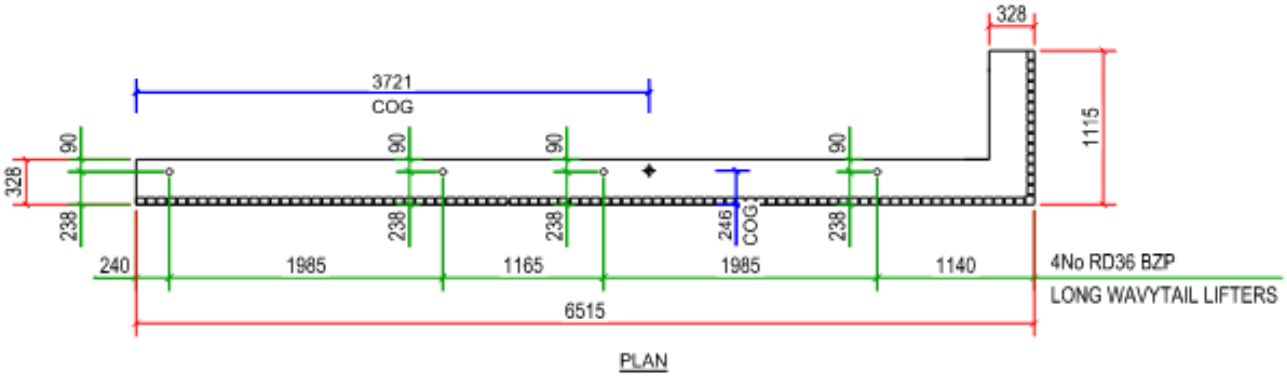
% Utilisation Against Overturning							
% Uti. against Overturning with Window				% Uti. Against Overtuning without Window			
No inclination	2.3 degrees	5 degrees	7.5 degrees	No inclination	2.3 degrees	5 degrees	7.5 degrees
77	86	96	109	41	45	50	57

% Utilisation Against Slidding	
% Uti. Against Sliding with Window	% Uti. Against Sliding without Window
All Inclinations	All Inclinations
61	33

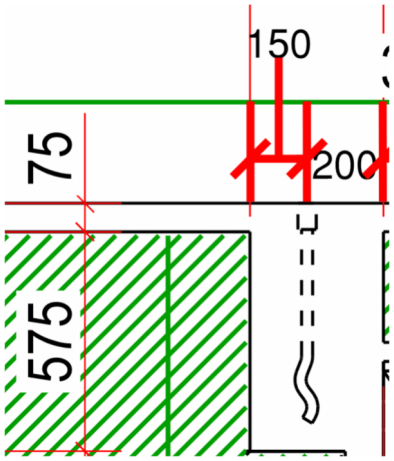
APPENDIX E: Panel lifting and handling checks

This document provides calculations to verify the lifting inserts during demoulding and handling in the factory yard.

Views from VMU panel drawing for manufacture (04)-SA8 revision C05:g



PLAN



Enlarged view of the local concrete body (worst-case)

Parameters

Panel self-weight at demould, $SW_{demould}$ = 5310Kg (no glazing)
Panel self-weight in the yard, SW_{total}= 6530kg (includes all fittings)
Lifter sizes = RD36 long wavy tail lifters CFS-WALS-36-570
* All lifts will be via the use of a spreader beam or equalizing device.

- Checks during demoulding

Distance from COG to centre of right-hand pair, A_{dis} = (240+1985+1165+1985/2)-3721
= 660mm

Distance from COG to centre of left-hand pair, B_{dis} = 3721-240-1985/2
= 2488mm

Distribution to right-hand pair = 2488/(660+2488)
= 0.790

Distribution to left-hand pair = 660/(660+2488)
= 0.210

During pitching out the mould, half the panel weight is on the formwork.

Pitching loads at pair A, F_{pitch} = ($SW_{demould}$/2 x Υ_{dyn} x Υ_{spread} x 0.790)/2 - Υ_{dyn} = 1.1 & Υ_{spread} = 1.16

= (5310(0.00981)/2) x 1.1 x 1.16 x 0.79

= 13.12kN per lifter (worst case)

Handling loads at pair A, $F_{hand} = (SW_{total} \times \Upsilon_{dyn} \times \Upsilon_{spread} \times 0.790)/2$ - where $\Upsilon_{dyn} = 1.3$ & $\Upsilon_{spread} = 1.16$

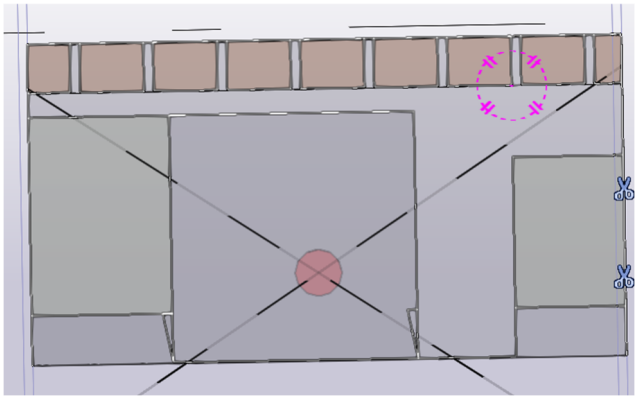
$= [(6530(0.00981)) \times 1.3 \times 1.16 \times 0.79]/2$

$= 38.15$kN per lifter (worst case)

RD36 shear capacity = 45.1kN

https://cfsfixings.com/products/lifting-inserts/long-wavy-tail-bzp

However, this must be reduced due to the given edge distances but can be amplified by the increased concrete strength, provided the amplified capacity does not exceed the CFS load of 113kN



As shown on the left, the breakout cone for edge shear is only impacted by one of the coffered locations (using 1.5xEdge distance per side of lifter). No other factors are effected besides the area of the break-out body and the $\psi_{s,v}$ factor.

Therefore reduced RD36 angled lift capacity = $45.1x\frac{(1.5(188)(1.5*188+150)}{(4.5(188^2))}x(0.7+0.3(\frac{(150)}{(1.5*188)}))x\sqrt{\frac{50MPa}{25MPa}}$

$= 42.0$kN > 13.12 kN

RD36 angled lift capacity = 113kN

https://cfsfixings.com/products/lifting-inserts/long-wavy-tail-bzp

However, this must be reduced due to the given edge distances but can be amplified by the increased concrete strength, provided the amplified capacity does not exceed the CFS load of 113kN

Therefore reduced RD36 angled lift capacity = $113x\frac{(150+200)(250)}{(500+500)(250)}x(0.7+0.3(\frac{(90)}{(250/2)}))x\sqrt{\frac{50MPa}{25MPa}}$

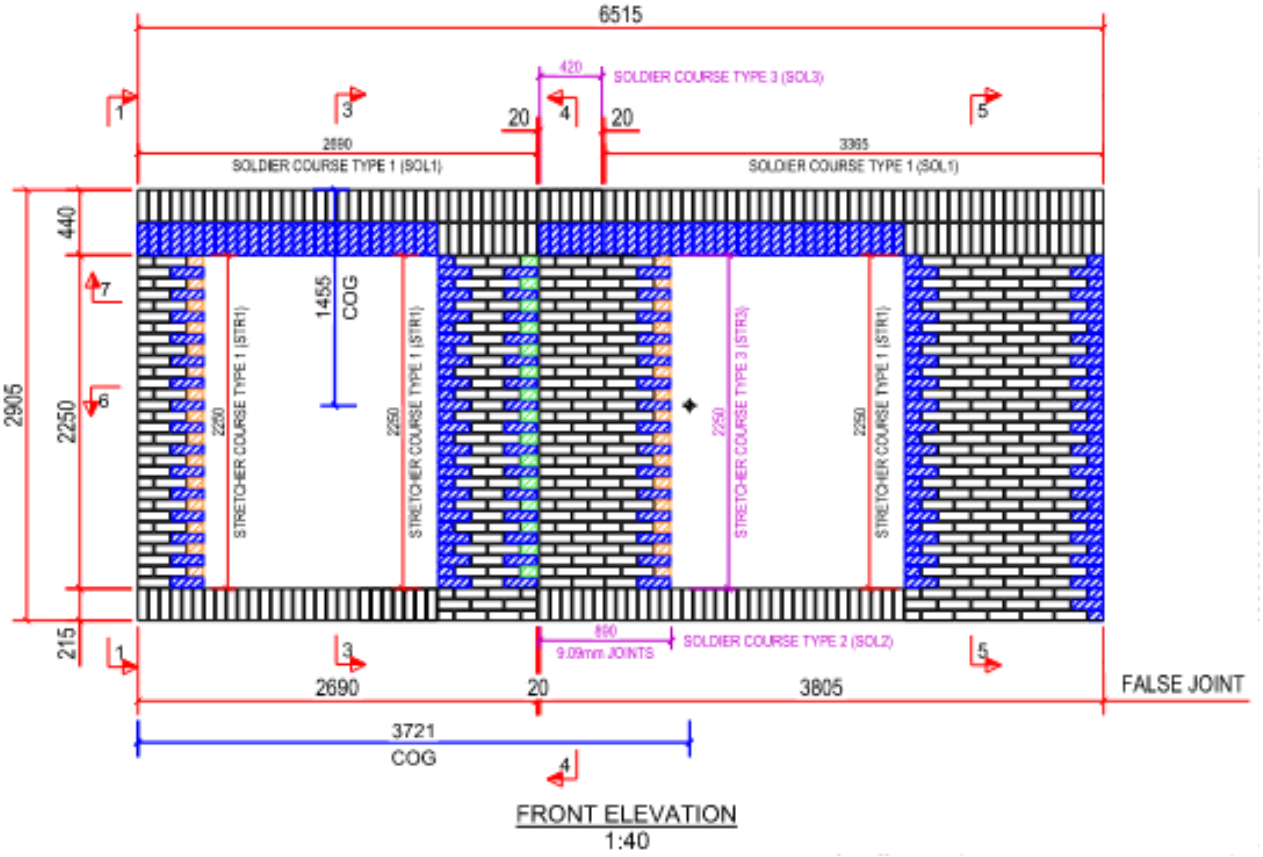
$= 51.2$kN > 38.15 kN

****Therefore RD36 long wavy tail lifters are sufficient to demould and handling the VMU panel.**

APPENDIX F: Stillage rack locking pin checks

This document provides verification that the Thorp Precast standard locking pin type 11's are suitable for this VMU arrangement.

Views from VMU panel drawing for manufacture (04)-SA8 revision C05:g

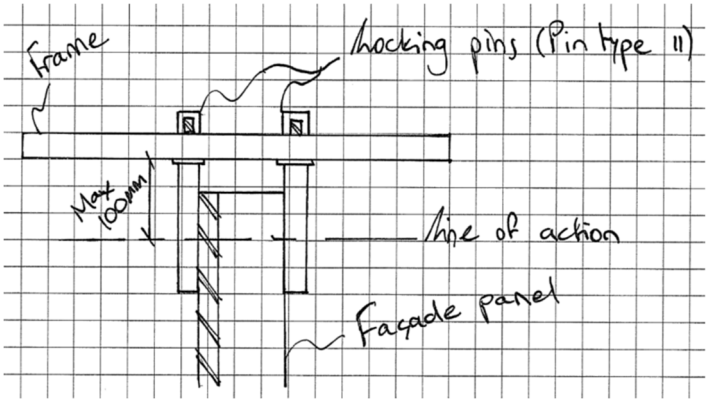


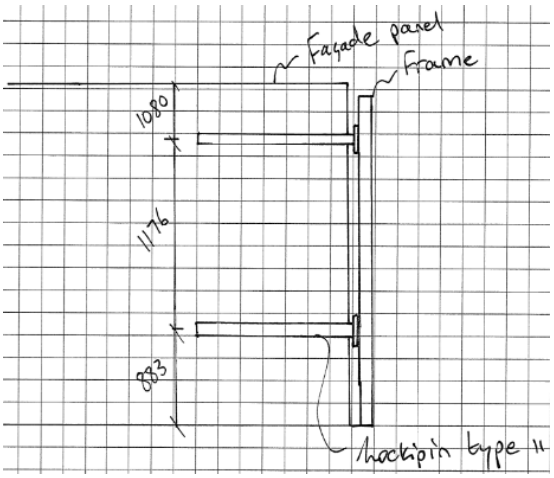
Parameters

* Temporary design wind load = 0.971kN/m^2 (See Appendix B)
* Pins are located 1176mm apart with two either side of the panel for inwards and outwards wind loads.
* It is assumed only 1 pin per side will be engaged at a time under wind loading (conservative)
* The pins are set out with a maximum of 100mm from the face of the frame
  the pins are type 11's at steel grade S355

Pin setting out

Sketches are not to scale





- Check pins in bending and shear

Pin steel profile = 50x5.0mm SHS grade S355

Pin shear area to BS EN 1993-1-1:

h_w = h-2 x thickness

= 50-2(5)

= 40mm

H_w / thickness = 8 =9.8 x ε / η < 72 x ε / η - Where η = 1.000 from 1993-1-5

Therefore shear buckling resistance can be ignored

Shear area to cl 6.2.6 (3), A_v = A x h/(b+h)

= 873mm^2 x 50/(50+50)

= 437mm^2

Design shear resistance cl 6.2.6(2), $V_{,Rd}$ = A_v x (f_y / $\sqrt{3}$)/Υ_{m0}

= 437 x(355/$\sqrt{3}$)/1.0

= 89.5kN

Design shear load, V_{Ed} = ϒQk x Wind load x panel width x (1.08m + 1.176m/2) - Where ϒQk = 1.5

= 0.971 x 6.515 x (1.08 +1.176/2)

= 15.82kN < 0.5V_{Rd} - No reduction to moment resistance

Design moment, M_{Ed} = V_{Ed} x lever arm

= 15.82(0.1)

= 1.582kNm

Required elastic modulus, $W_{el,y}$ = M_{Ed} / f_y/Υ_{m0}

= 1.582(10^6) / 355 / 1.0

= 4456mm^3 < 11600mm^3 for a 50x5.0mm SHS

Therefore, the Type 11 pin is sufficient.