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MANOR HOUSE, HAM STREET, TW10 7HA

Addendum to Structural Impact Assessment for Planning
P03

Status	Revision	Issued For	Date	Author
PRELIMINARY	P01	SIA addendum	29.08.23	LQ
PRELIMINARY	P02	SIA addendum – Statement regarding no further joists to be notched added to section 4.2	12.02.24	LQ
PRELIMINARY	P03	SIA addendum – Statement regarding timber flitches to be used whenever possible & discussion on roof trusses	13.02.24	LQ

CONTENTS

1 INTRODUCTION

This report presents Michael Barclay Project Ltd.'s ('MBP's') addendum to the existing Structural Impact Assessment in support of the planning application for the refurbishment of the Grade II* listed Manor House, Richmond TW10 7HA.

2 THE EXISTING BUILDING

The existing property is formed by a central three-storey Queen Anne house, extended to the North and the South during the Edwardian period with two wings at ground floor and part of the first floor. The property was extensively refurbished and remodelled in the 1970s. There is an existing basement underneath the original part of the house.

3 OVERVIEW OF THE PROPOSED WORKS

Alterations to the internal layouts are proposed to the existing main house to provide a more workable layout and the loft space to the south of the property is proposed to be converted into an attic bedroom with bathroom. This report focuses on revisions to these internal layouts since the last Structural Impact Assessment was produced.

Other proposed work includes: a single-storey extension to the north of the house, to form a new living room and a loggia. Also, the construction of a new basement pool and spa complex to the south of the existing building. This addendum does not put forward any further revisions to these proposed works already submitted for planning.

4 THE PROPOSED SCHEME

4.1 GROUND FLOOR

The proposed internal re-arrangement of rooms to the rear of the property includes the removal of non-structural walls that were previously installed as part of past remodelling works. At ground floor a non-structural wall is demolished that had created a corridor to the rear doors opening onto the garden. Internal doorways in nearby structural walls are moved slightly to create a better layout – precast concrete lintels are proposed for these openings.

4.2 FIRST FLOOR

Heavy masonry partition walls were added during past renovation work, we believe in the 1970s, to create multiple bedrooms and bathrooms. These walls are proposed to be removed to create a better layout with larger rooms at first floor. The heavy partition walls have caused the first-floor timber joist structure in the corridor area to sag excessively and so removing them will lessen the heavy loading on the floor joists and primary oak beams.

The existing timber joists in the corridor area have also been notched excessively for the addition of pipework serving the new bathrooms, adding to the sagging problem with the existing timber joists. It is proposed to add timber or steel flitch plates to these joists to strengthen them to prevent further damage and movement. Some of the existing notching may be re-used for new services. This will need to be carefully coordinated with the M&E consultant and contractor to work through pipe layouts and existing notching so that no further notching will be necessary for the joists.

All new walls proposed to be added to the first floor will be formed in a lightweight construction. Joists below the new walls will be strengthened with timber or steel flitches to support the additional loading.

A new stair is to be added to the first floor to connect to the new attic room at second floor – at the southernmost end of the building. The stair will require partial removal of the existing masonry below the original eaves level.

New openings in existing structural masonry walls are proposed to be formed using precast concrete lintels.

4.3 SECOND FLOOR

At second floor, the new staircase enters the existing attic space having passed beneath the existing historic timber wallplate and eaves – these bear onto what would have been an original external wall line.

The existing attic timberwork at the southernmost end of the building is of much later construction. The roof shape is formed using 3 large timber trusses and the attic floor is formed using deep timber joists.

The new stair void is proposed to be framed out using small steel beams spanning between structural masonry walls below. The existing attic joists are very deep, but since they span nearly 7m and the area is not used for occupancy or storage, they are not sufficient for domestic occupancy loading. We have therefore proposed to strengthen these joists using flitch plates (timber or steel flitches will be chosen appropriately for the loads, spans and stresses applied, but with a preference for timber flitches wherever possible). Partition walls are also to be added on top of the existing joists to create the perimeter walls of the bedroom and connected bathroom adding further load to the existing joists.

The existing trusses in the attic will need to be modified to enable removal of the diagonal struts that would hinder movement across the room. We have proposed small steel channel sections are added to the horizontals and verticals of the truss to enable the central 'V' to be removed. These channels will be bolted through the existing timber so that the steel and timber work together in unison. The channels are added to the 'back' face of the trusses, i.e., the face that is not visible from the middle of the room – this should reduce their visual impact. Installation, jointing and 'buildability' details are included in the appended calculations.

5 DESIGN AND PERFORMANCE PARAMETERS

5.1 OCCUPANCY LOADS

The new structure elements will be designed in accordance with the Eurocodes and associated National Annexes. The general design imposed loads for the building is as follows (values highlighted in red):

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to 2,0	2,0 to 3,0
- Stairs	2,0 to 4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores
[1)] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2 NOTE 3 See 6.3.2 for storage or industrial activity		

* defined by BS EN 1991-1-1:2002

5.2 ENVIRONMENTAL LOADS

All new structure will be designed to support loads from the wind in combination with the occupancy loads scheduled above in accordance with EN1991-1-4:2005+A1:2010 and the UK national annex.

5.3 PERMISSIBLE DEFLECTIONS

The design of new constructional steel and reinforced concrete elements will limit deflection and displacement in accordance with the following criteria:

Structural Elements	Limit – under full load	Limit- under full load for stone finishes	Limit- under full load where supporting walls
Simple Beams	Span / 360	Span/750	Span/500
Cantilever Beams	Span / 360	Span/750	Span/500

The above criteria must be read in conjunction with any performance specifications produced by MBP for individual works packages.

5.4 DURABILITY

The design life of the new building is taken as a minimum period of 60 years. This is in accordance with BS EN 1992-1-1:2002 Section 4 and corresponds to the standard durability used for buildings in this category, which includes new housing and high-quality refurbishment of public buildings.

5.5 DESIGN CODE AND STANDARDS

The following design codes will be used for the design of the proposed new dwelling:

- BS EN 1990:2002+A1:2005, Eurocode 0, Basis of structural design
- BS EN 1991-1-1:2002, Eurocode 1: Actions on structures, Part 1-1; General Actions -Densities, self-weight, imposed load for buildings.
- BS EN 1991-1-3:2003 + A1: 2015 Eurocode1: Actions on structures, Part 1-3: General Actions-Snow loads, including the UK National Annex
- BS EN 1991-1-4:2005+A1:2010, Eurocode 1: Actions on structures, Part 1-4: General Actions-Wind action, including the UK National Annex
- BS EN 1992-1-1:2004+A1:2014, Eurocode 2: Design of concrete structures, Part 1-1: General rules and rules for building, including the UK National Annex.
- BS EN 1993-1-1: 2005+A1:2014, Eurocode 3: design of steel structures, Part 1-1: General rules and rules for buildings, including the UK National Annex
- BS EN 1995-1-1:2004+A1:2014, Eurocode 5: Design of timber structures, Part 1-1: General - Common rules and rules for buildings, including the UK National Annex
- BS EN 1996-1-1:2005+A1:2012, Eurocode 6: Design of masonry structures, Part 1-1: General rules for reinforced and unreinforced masonry structures, including the UK National Annex
- BS EN 1997-1-1:2004, Eurocode 7: Geotechnical design -Part1: General rules
- The Building Regulations 1991- Approved Documents A, B, C, E, H, K & N

6 CONSTRUCTION HAZARDS

The proposed construction has standard materials and components and is of common form within the construction industry. Nevertheless, MBP will produce a separate document that will be developed as the detailed design proceeds.

7 SPECIFICATION

The proposed construction materials, components, workmanship etc. will be specified using the National Building Specification documents and a separate performance specification. Those sections that MBP will schedule for planning stage are:

Excavating and Filling	D20
Embedded Retaining Walls	D40
Underpinning	D50
In situ concrete construction generally	E05
In situ concrete mixes, casting and curing	E10
Formwork for in situ concrete	E20
Reinforcement for in-situ concrete	E30
Worked finishes to in situ concrete	E41
Brick/block walling	F10
Structural steel framing	G10
Carpentry / timber framing/ first fixing	G20
Intumescent coatings for fire protection of steelwork	M61
Holes/chases/covers/supports for services	P31

It is Michael Barclay Partnership's practice to specify materials and construction-practices that do not cause undue harm to the environment. For example, timber used in temporary and permanent works must be obtained from a certified sustainable source, and be identified as such. The paint specification will avoid red lead, zinc chromate or coal-tar content and have a low solvent (VOC) content and offer manufacturers with an Environmental Policy in operation. The Contractor will be encouraged to use Portland cement replacement materials for the reinforced concrete elements.

8 RECYCLING

MBP would intend to re-use and recycle as much of the existing construction materials as possible, re-using any timber or steel removed as part of the proposed remodelling works. Any new concrete will use cement substitutes and recycled aggregates and reinforcement bars made from recycled steel. Concrete and masonry arisings can be re-used as hardcore fill beneath new slabs.

9 APPENDED DOCUMENTS

The following documents are appended to this report:

A - Michael Barclay Partnership Drawings

B - Michael Barclay Partnership Roof Truss Calculations & Buildability Details.

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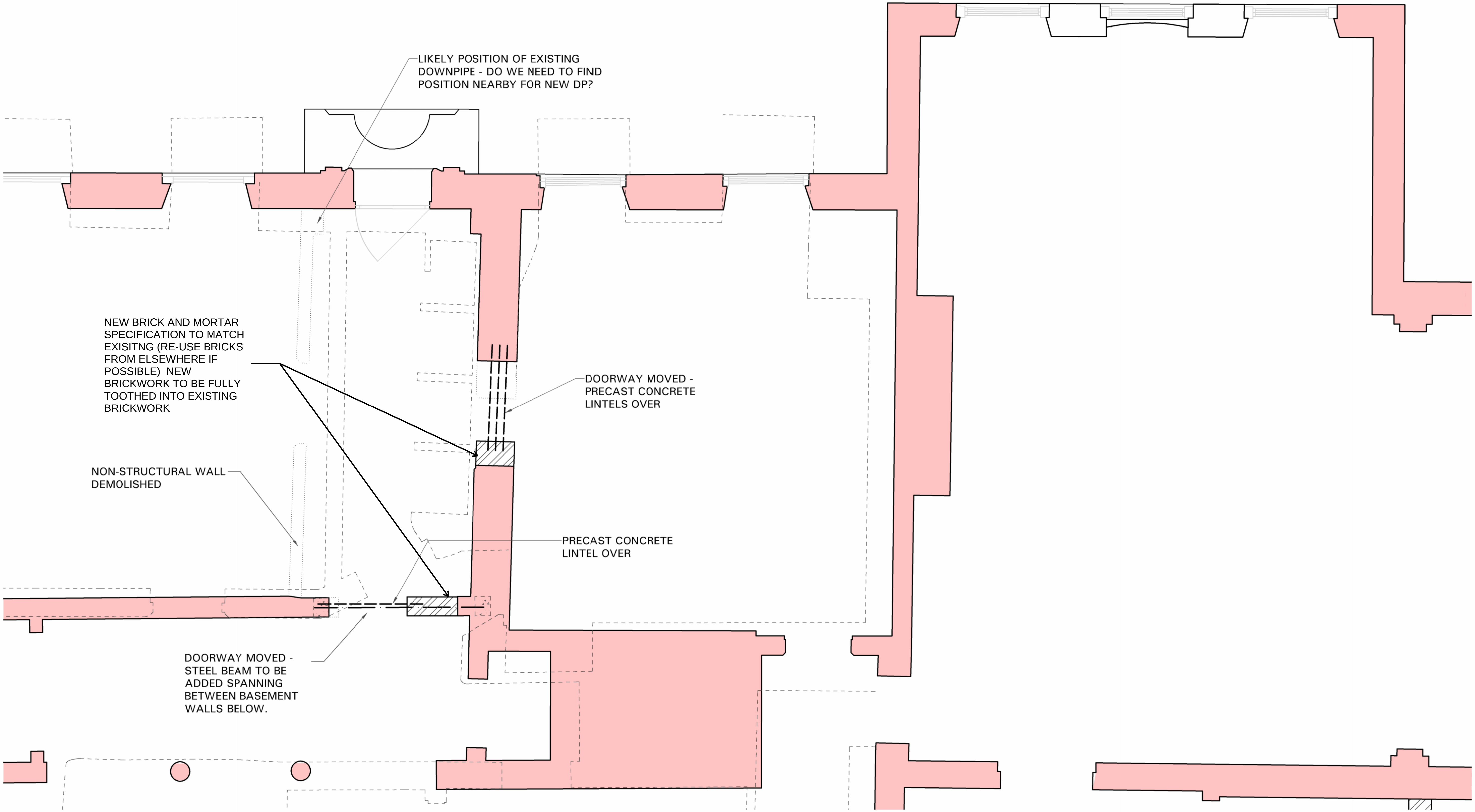
APPENDIX A - MICHAEL BARCLAY PARTNERSHIP DRAWINGS



Rev	Date	Description	By
P02	01/02/24	PRELIMINARY ISSUE	LQ
P1	23/08/23	PRELIMINARY ISSUE	LQ

Job
THE MANOR HOUSE,
HAM STREET,
RICHMOND TW10

Title
MAIN HOUSE
PART GROUND FLOOR PLAN

Scale @ A3	Date	By	Checked
1:50	AUG '23	LQ	

Status
PRELIMINARY

Drawing Number
MBP/8592/MH-00-DR-S-0300

Revision
P02

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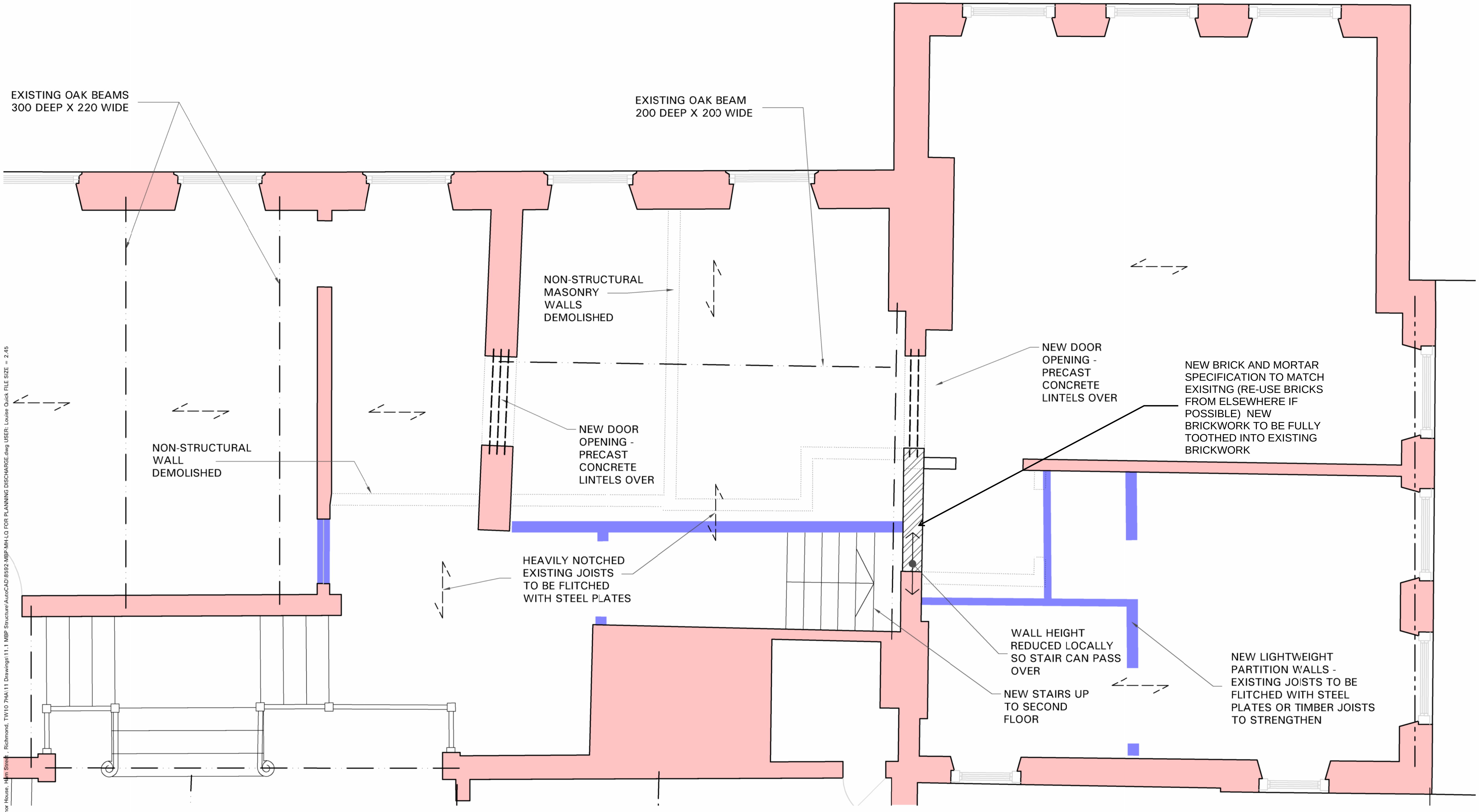

Rev	Date	Description	By
P02	01/02/24	PRELIMINARY ISSUE	LQ
P1	23/08/23	PRELIMINARY ISSUE	LQ

Job	THE MANOR HOUSE, HAM STREET, RICHMOND TW10
Title	MAIN HOUSE PART FIRST FLOOR PLAN
Scale @ A3	1:50
Date	AUG '23
By	LQ
Checked	
Status	PRELIMINARY
Drawing Number	MBP/8592/MH-01-DR-S-0301
Revision	P02

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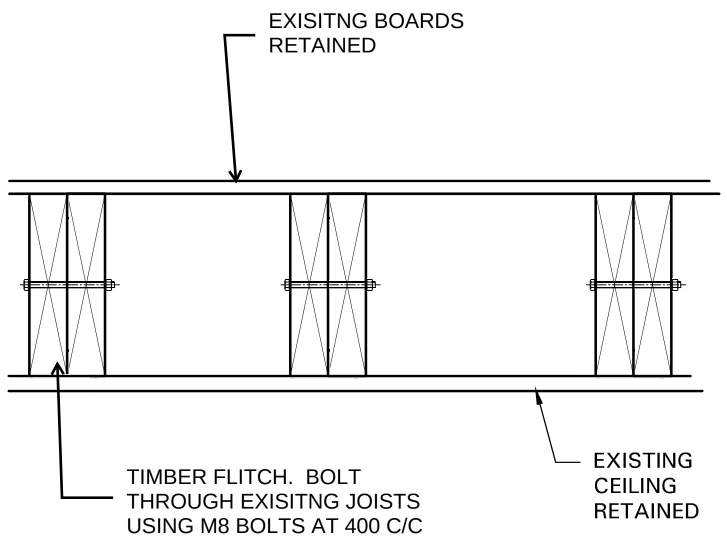
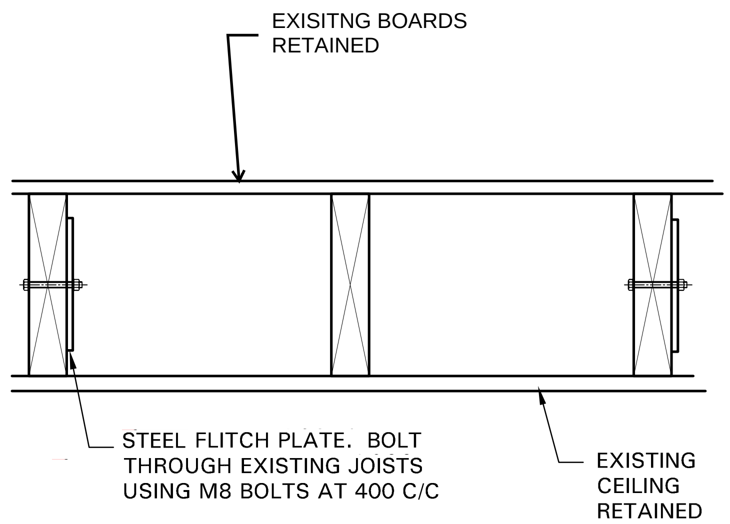


TYPICAL FLITCH PLATE DETAIL
TIMBER OPTION (PREFERRED)



TYPICAL FLITCH PLATE DETAIL
STEEL OPTION

Rev	Date	Description	By
P02	13/02/24	PRELIMINARY ISSUE	LQ
P1	01/02/24	PRELIMINARY ISSUE	LQ

Job
THE MANOR HOUSE,
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Scale @ A3	Date	By	Checked
1:50	AUG '23	LQ	

Status
PRELIMINARY

Title
MAIN HOUSE
FLITCH DETAIL

Drawing Number
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APPENDIX B - MICHAEL BARCLAY PARTNERSHIP ROOF TRUSS CALCULATIONS & BUILDABILITY DETAILS

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MANOR HOUSE (MAIN HOUSE)	8592	1	
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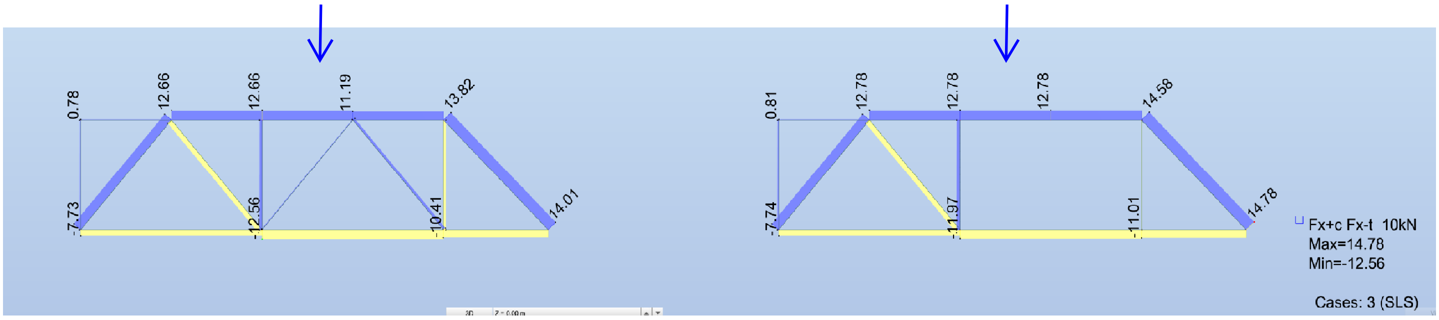
ROOF TRUSS MODIFICATIONS BEDROOM 8	01/02/2024		

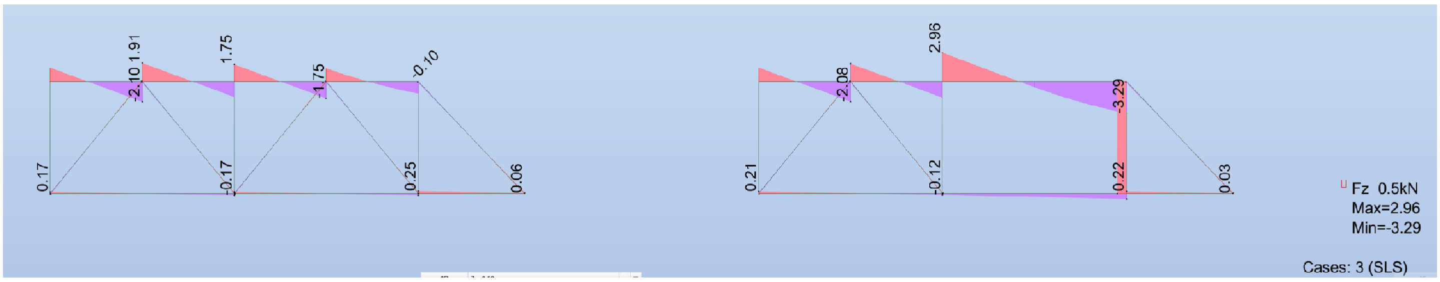
TRUSS X-X EXISTING AND PROPOSED TRUSS FORCES

EXISTING

PROPOSED



AXIAL LOADS SLS



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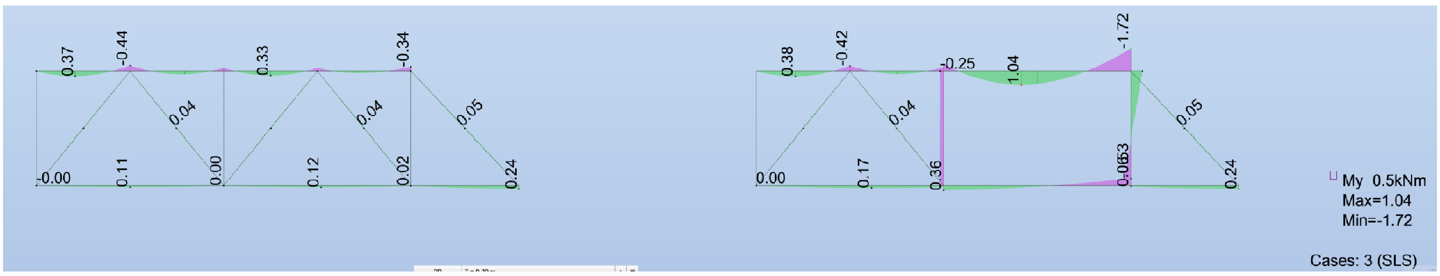

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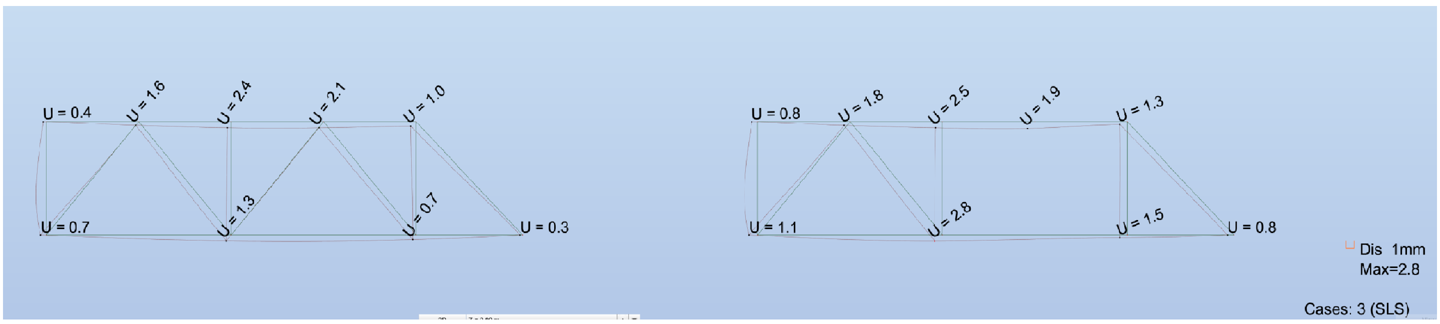

DEFORMATION SLS

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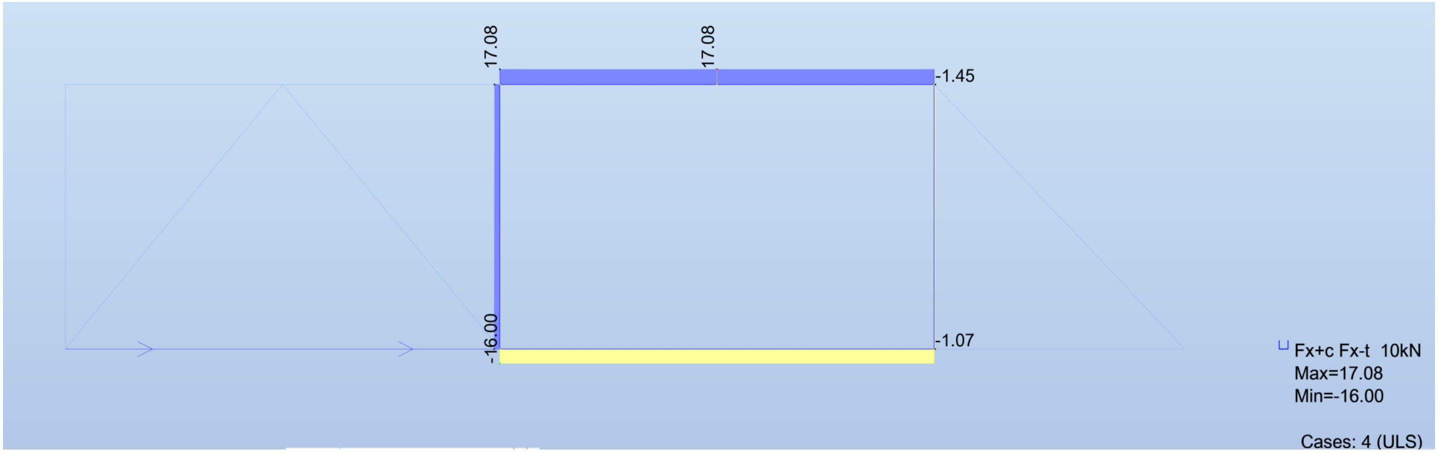
ROOF TRUSS MODIFICATIONS BEDROOM 8	01/02/2024		

TRUSS X-X PROPOSED BOX FRAME IN ISOLATION



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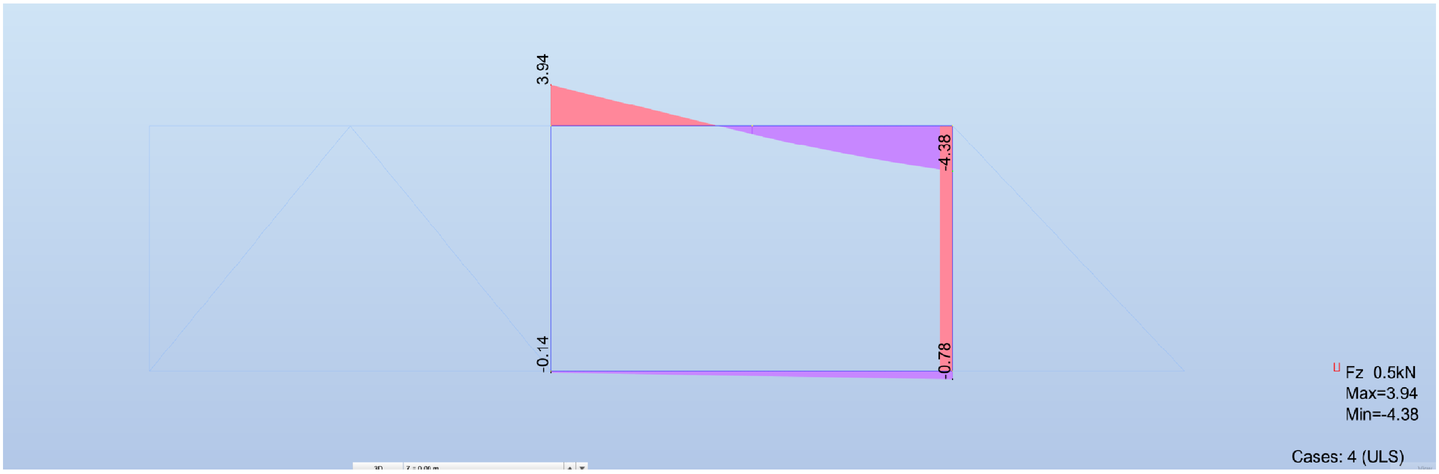

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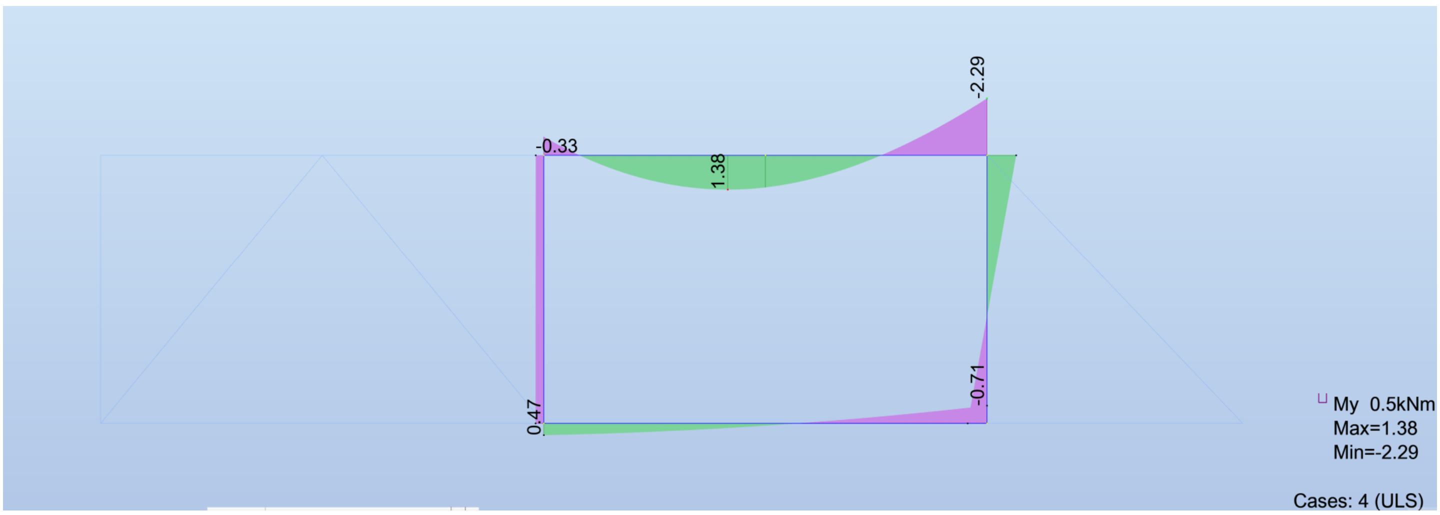

MOMENTS ULS

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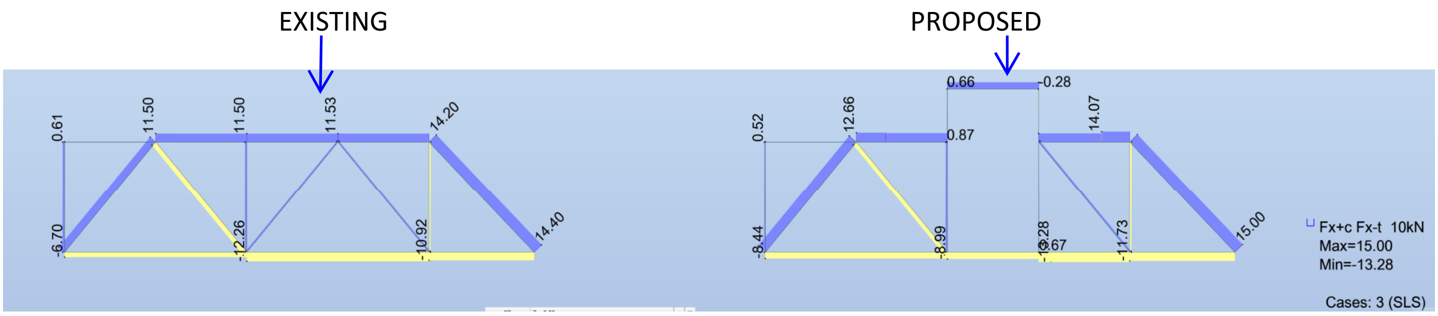
MANOR HOUSE (MAIN HOUSE)	8592	3	
Calculation / Sketch Title	**Date**	**Author**	**Checked**
ROOF TRUSS MODIFICATIONS BEDROOM 8	01/02/2024		

TRUSS Y-Y EXISTING AND PROPOSED TRUSS FORCES



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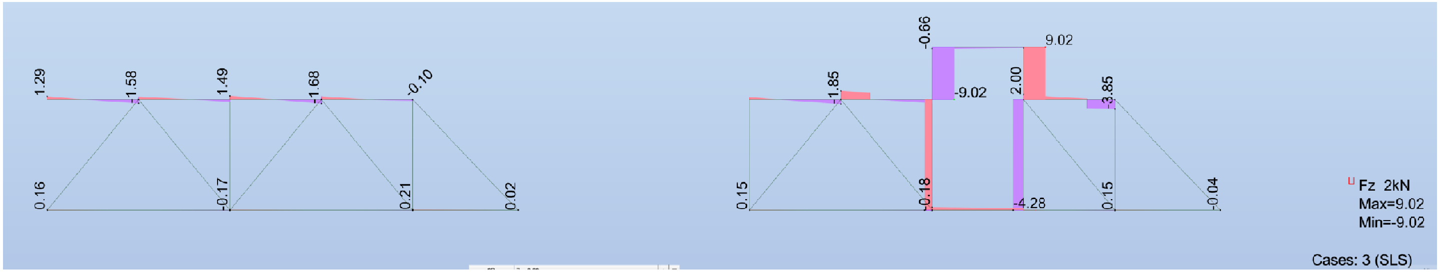

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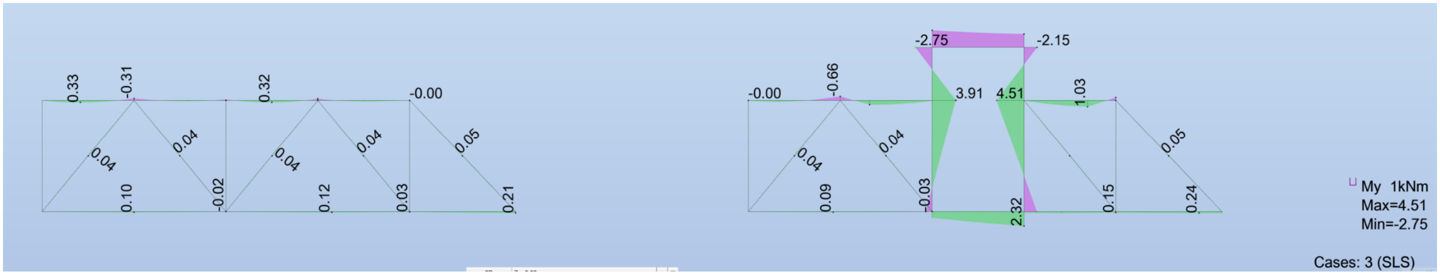

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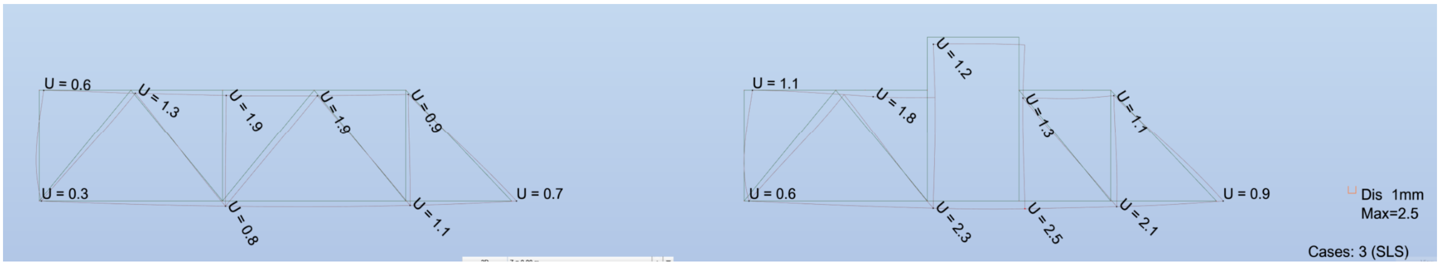

DEFLECTION SLS

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MANOR HOUSE (MAIN HOUSE)	8592	4	
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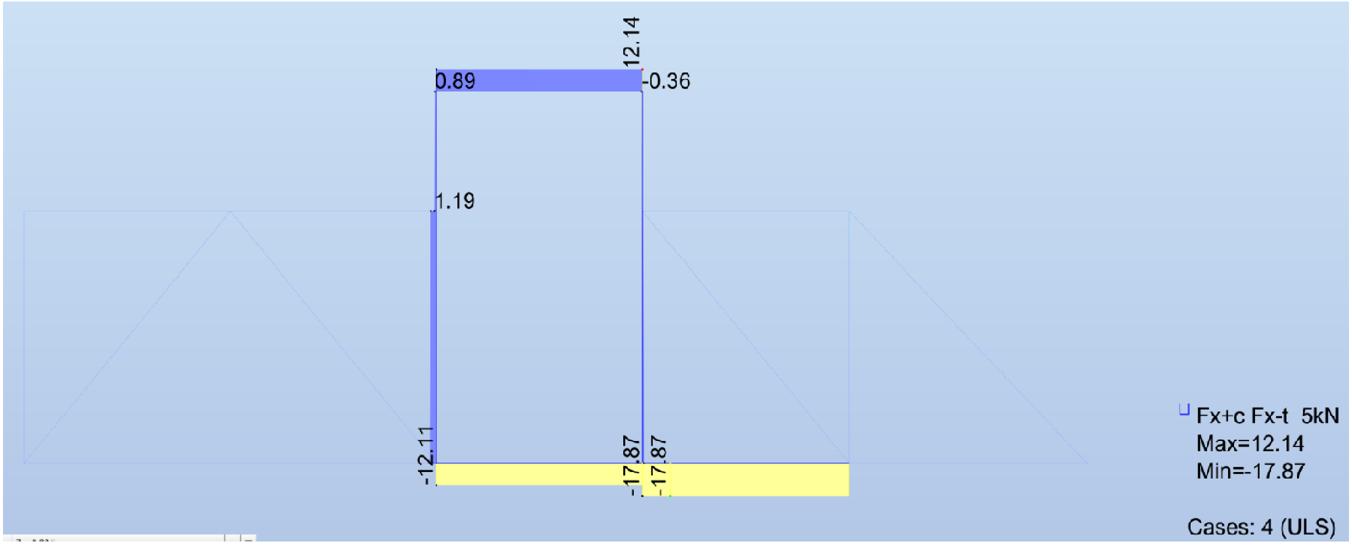
ROOF TRUSS MODIFICATIONS BEDROOM 8	01/02/2024		

TRUSS Y-Y PROPOSED BOX FRAME IN ISOLATION



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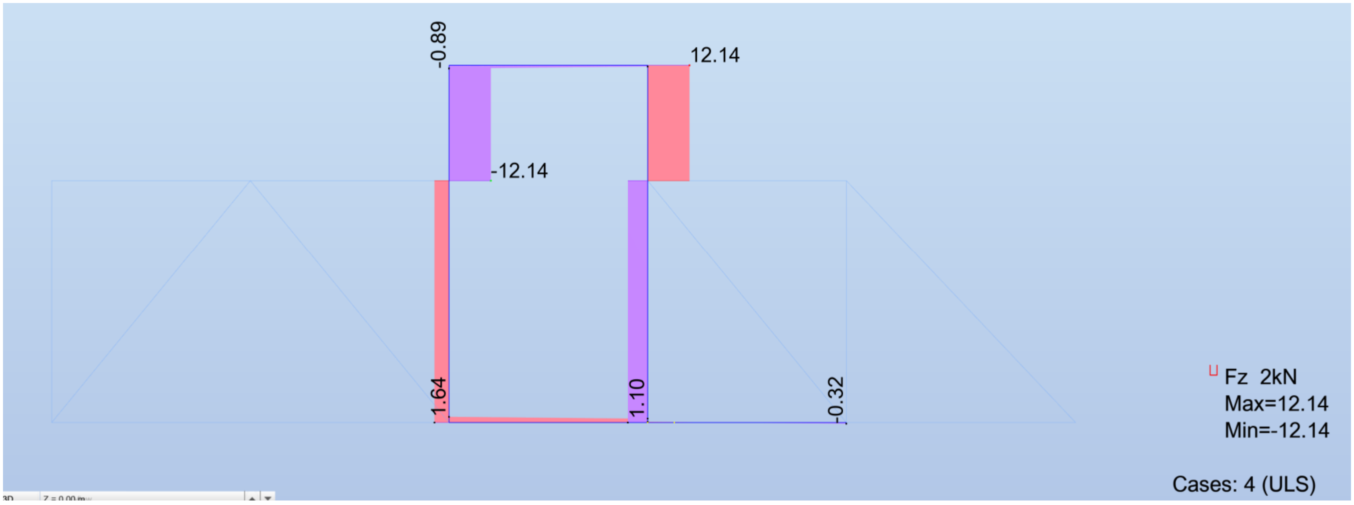

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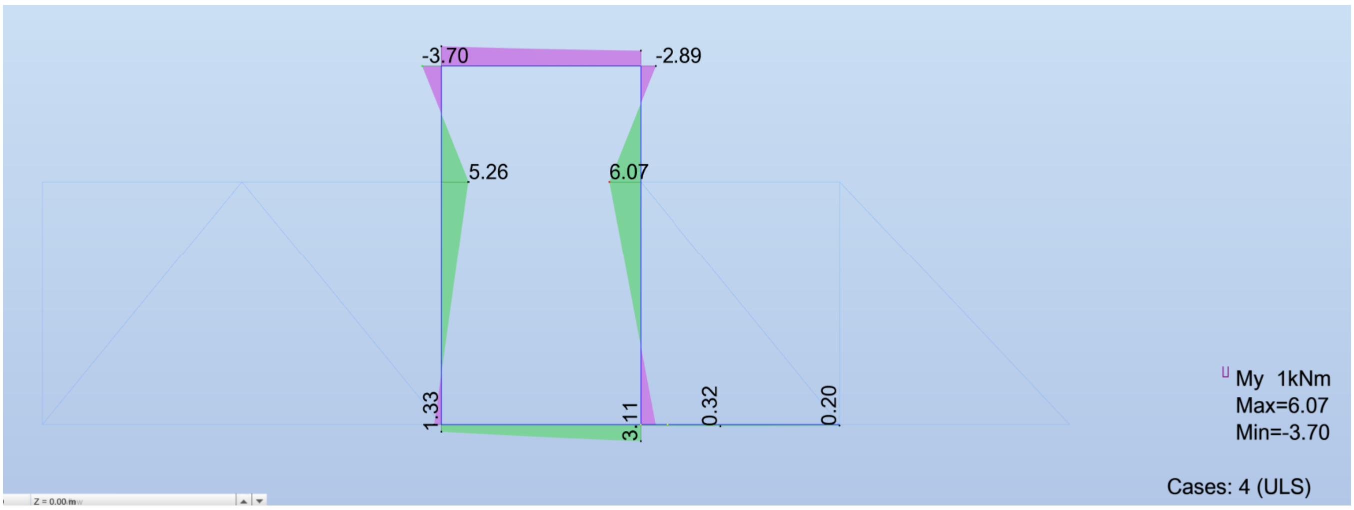

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MANOR HOUSE (MAIN HOUSE)	8592	5	
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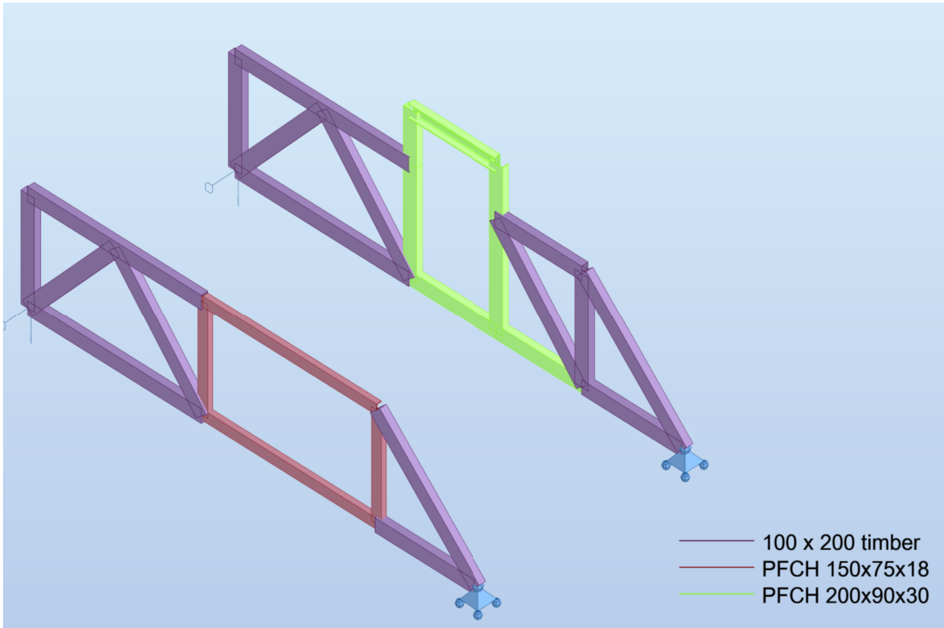
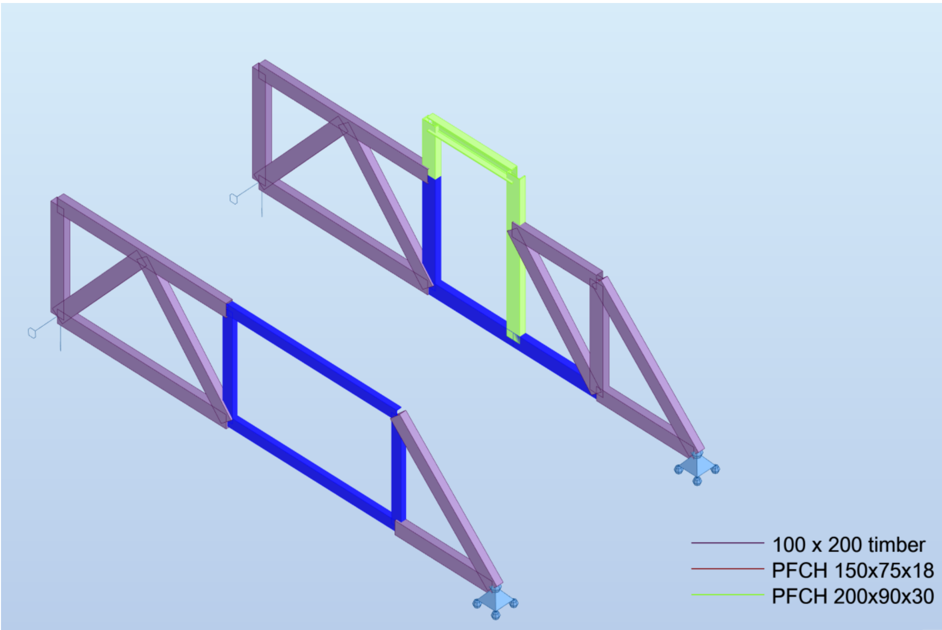
ROOF TRUSS MODIFICATIONS BEDROOM 8	01/02/2024		

DESIGN



TIMBER MEMBERS ARE REPLACED WITH STEEL IN THE ANALYSIS MODEL BUT IN REALITY THEY WILL BE RETAINED (AS PER MBP DRAWINGS) AND THE PFC WILL BE ADDED & BOLTED THROUGH THE EXISITNG TIMBER



THE BLUE MEMBERS WILL BE EXISTING TIMBER PLUS PFC

ONLY THE REMAINING GREEN MEMBERS (200 X 90 PFC) WILL BE WORKING IN ISOLATION

THE APPENDED TEDDS CALCULATION IS FOR A 150 X 75 PFC, 2.6m LONG (i.e., THE LONGEST) WITH MAX MOMENT, SHEAR AND COMPRESSION APPLIED. THE PFC SECTION IS ADEQUATE

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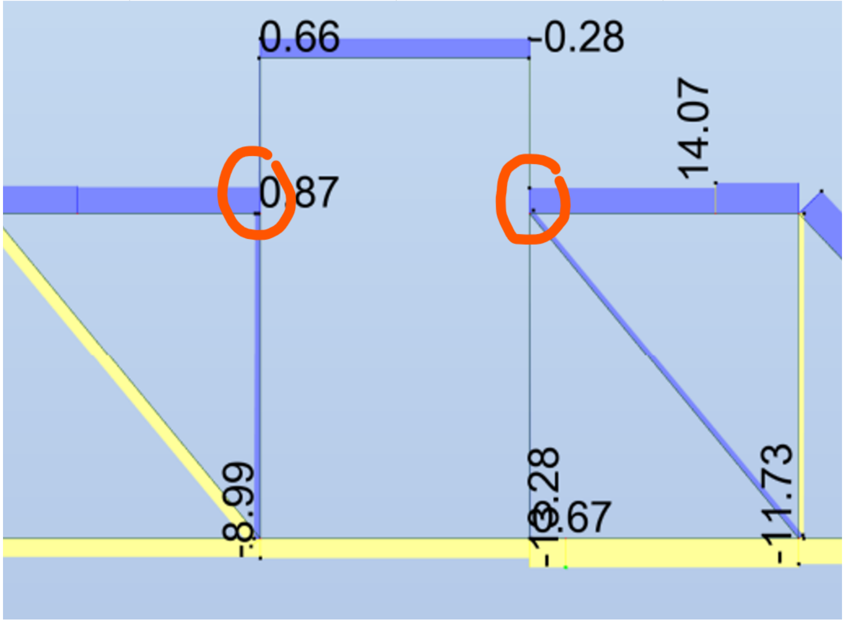
MANOR HOUSE (MAIN HOUSE)	8592	6	
Calculation / Sketch Title	**Date**	**Author**	**Checked**
ROOF TRUSS MODIFICATIONS BEDROOM 8	01/02/2024		

TIMBER TO STEEL CONNECTION



MOST CRITICAL IS MARKED.

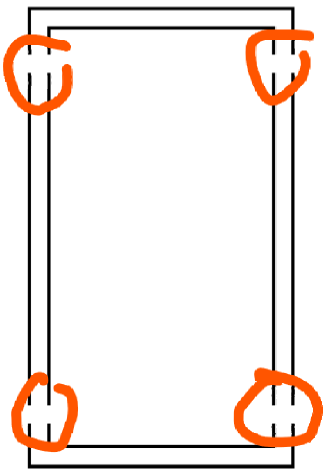
TOP TIMBER BOOM IS IN COMPRESSION & LOAD NEEDS TO BE TRANSFERRED INTO THE PFC FRAME

LONG 10mm THK X 195mm DEEP TAB PLATES TO BE WELDED TO PCF RAME AND BOLTED THROUGH TIMBER USING 4No. M16 BOLTS - SEE APPENDED TEDDS CALCULATION

LOADS ARE APPLIED CLOSE TO EH SHEAR CENTRE OF THE PFC SO THAT SECONDARY TORSIONAL EFFECTS ARE NOT A PROBLEM

FRAME BUILDABILITY

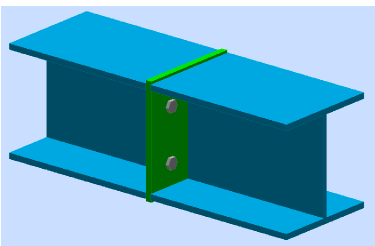
STEEL MEMBERS WILL BE BROUGHT IN AS 'STICK' ELEMENTS AND SPLICED INTO A BOX FRAME ON SITE. THE EXISITNG TRUSSES WILL NEED TO BE SURVEYED ACCURRATELY SO THAT THE FRAMES CAN BE FABRICATED TO SUIT THE EXISITNG SHAPE. WE CAN ALLOW FOR SOME LOOSE SHIM PLATES BETWEEN THE JOINTS. HIGH STRENGTH FRICTION GRIP BOLTS WILL BE SPECIFIED.



TYPICAL SPLICE LOCATIONS ARE MARKED. MAXIMUM FORCES ARE TAKEN FROM WORST LOCATIONS & APPLIED ALL AT ONE SPLICE LOCAITON. THIS IS CONSERVATIVE

MAX MOMENT	3.7	kNm ULS
MAX SHEAR	12.14	kN ULS
MAX AXIAL	17.87	kN ULS

FOR A QUICK CALCULATION ROBOT HAS BEEN USED FOR A SPLICE CONNECTION CHECK. THE SOFTWARE DOESN'T HANDLE PFCs SO WE HAVE DOUBLED UP THE FORCES AND APPLIED TO A 203 UC SECTION. THE BOLT AND PLATE ARRANGEMENT PER SIDE OF UC IS THE SAME AS THAT FOR THE PFC. BOLT, PLATE AND WELD SIZES WILL APPLY TO THE PFC SPLICE.



SEE APPENDED CONNECTION DESIGN OUTPUT.

MBP Consulting Engineers 1 Lancaster Place London WC2E 7ED T 020 7240 1191 E london@mbp-uk.com www.mbp-uk.com	Project MANOR HOUSE (MAIN HOUSE)				Job no. 8592	
	Calcs for BEDROOM 8 TRUSSES - WORST CASE PFC				Start page no./Revision 1	
	Calcs by LQ	Calcs date 01/02/2024	Checked by	Checked date	Approved by	Approved date

STEEL MEMBER 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 4.4.11

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Design section 1 - WORST CASE PFC

Section details

Section type UKPFC 150x75x18 (Tata Steel Advance)

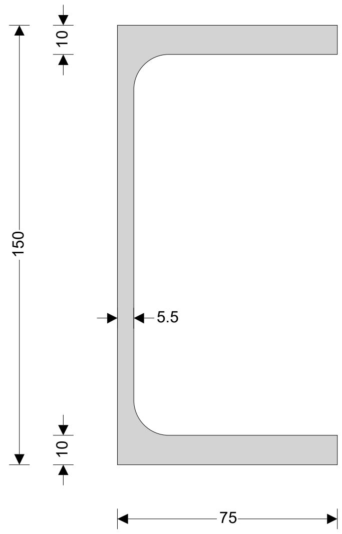
Steel grade - EN 10025-2:2004 S355

Nominal thickness of element $t_{nom} = \max(t_f, t_w)$ = **10** mm

Nominal yield strength f_y = **355** N/mm²

Nominal ultimate tensile strength f_u = **470** N/mm²

Modulus of elasticity E = **210000** N/mm²



UKPFC 150x75x18 (Tata Steel Advance)
Section height, h, 150 mm
Section breadth, b, 75 mm
Mass of section, Mass, 17.9 kg/m
Flange thickness, t_f, 10 mm
Web thickness, t_w, 5.5 mm
Root Radius, r_1, 12 mm
Area of section, A, 2277 mm²
Radius of gyration about y-axis, i_y, 61.493 mm
Radius of gyration about z-axis, i_z, 23.981 mm
Elastic section modulus about y-axis, $W_{el.y}$, 114793 mm³
Elastic section modulus about z-axis, $W_{el.z}$, 26608 mm³
Plastic section modulus about y-axis, $W_{pl.y}$, 132089 mm³
Plastic section modulus about z-axis, $W_{pl.z}$, 46581 mm³
Second moment of area about y-axis, I_y, 8609494 mm⁴
Second moment of area about z-axis, I_z, 1309331 mm⁴

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **6.07** kNm

Design shear force - Major axis $V_{y,Ed}$ = **12.14** kN

Design axial compression force N_{Ed} = **17.87** kN

Restraint spacing

Major axis lateral restraint L_y = **2500** mm

Minor axis lateral restraint L_z = **2600** mm

Torsional restraint L_T = **2600** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ \text{N/mm}^2 / f_y]}$ = **0.81**

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

Width of section c = d = **106** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r_1)] / c, 1)$ = **0.543**

$c / t_w = 19.3 = 23.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = b - t_w - r_1 =$ **57.5** mm

$c / t_f = 5.7 = 7.1 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} =$ **17.9** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **808.3** kN

$N_{Ed} / N_{c,Rd} =$ **0.022**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,y} = 1.2 \times L_{y\_s1} =$ **3000** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **1982.7** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.638**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 c

Imperfection factor - Table 6.1 $\alpha_y =$ **0.49**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.811**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.762**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **616.2** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.029**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,z} = 1.2 \times L_{z\_s1} =$ **3120** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **278.8** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **1.703**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 c

Imperfection factor - Table 6.1 $\alpha_z =$ **0.49**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **2.318**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.257**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **207.8** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.086**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{T\_s1} =$ **2600** mm

Distance from shear centre to centroid in y axis $y_0 = ((h - 2 \times t_f) \times t_w^2 / 2 + b^2 \times t_f) / ((h - 2 \times t_f) \times t_w + 2 \times b \times t_f) + 3 \times b^2 \times t_f / (h \times t_w + 6 \times b \times t_f) - t_f / 2 =$ **53.0** mm

Distance from shear centre to centroid in z axis $z_0 =$ **0.0** mm

Radius of gyration $i_0 = \sqrt{(i_y^2 + i_z^2 + y_0^2)} =$ **84.6** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) =$ **882** kN

Torsion factor $\beta_T = 1 - (y_0 / i_0)^2 =$ **0.608**

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}] =$ **720.8** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **720.8** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **1.059**

Design resistance for torsional and torsional-flexural buckling - Section 6.3.1.1

Buckling curve - Table 6.2 c

Imperfection factor - Table 6.1 α_T = **0.49**

Buckling reduction determination factor $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.271**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.507**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **409.4** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.044**

PASS - Design buckling resistance exceeds design compression

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **130** mm η = **1.000**

$h_w / t_w = 23.6 = 29.1 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **12.1** kN

Shear area - cl 6.2.6(3) $A_v = A - 2 \times b \times t_f + (t_w + r_1) \times t_f$ = **952** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **195.1** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.062**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **6.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **46.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.129**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - User defined k_c = **1**

$C_1 = 1 / k_c^2$ = **1**

Poissons ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained effective length $L = 1.2 \times L_{z\_s1} + 2 \times h$ = **3420** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **36.4** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.135**

Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 d

Imperfection factor - Table 6.3 α_{LT} = **0.76**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.263**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.487**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.487**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **22.8** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.266**

PASS - Design buckling resistance moment exceeds design bending moment

Resistance of cross-section - Section 6.2.1

Interaction formula - eq.6.2 $N_{Ed} / N_{c,Rd} + M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.152}$

PASS - Utilisation of combined bending and axial force is acceptable

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 $C_{my} = \mathbf{1.000}$

$C_{mz} = \mathbf{1.000}$

$C_{mLT} = \mathbf{1.000}$

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance $M_{y,Rk} = W_{el.y} \times f_y = \mathbf{40.8}$ kNm

Characteristic moment resistance $M_{z,Rk} = W_{el.z} \times f_y = \mathbf{9.4}$ kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y = \mathbf{808.3}$ kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(0.6 \times \bar{\lambda}_y, 0.6) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = \mathbf{1.011}$

$k_{zy} = 1 - 0.05 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = \mathbf{0.994}$

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.338}$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.39}$

PASS - Combined bending and compression checks are satisfied

Design section 2

Section details

Section type UKPFC 150x75x18 (Tata Steel Advance)

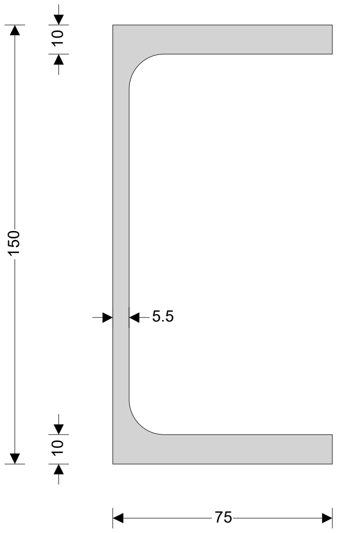
Steel grade - EN 10025-2:2004 S355

Nominal thickness of element $t_{nom} = \max(t_f, t_w) = \mathbf{10}$ mm

Nominal yield strength $f_y = \mathbf{355}$ N/mm^2

Nominal ultimate tensile strength $f_u = \mathbf{470}$ N/mm^2

Modulus of elasticity $E = \mathbf{210000}$ N/mm^2



UKPFC 150x75x18 (Tata Steel Advance)
Section height, h, 150 mm
Section breadth, b, 75 mm
Mass of section, Mass, 17.9 kg/m
Flange thickness, t_f, 10 mm
Web thickness, t_w, 5.5 mm
Root Radius, r_1, 12 mm
Area of section, A, 2277 mm^2
Radius of gyration about y-axis, i_y, 61.493 mm
Radius of gyration about z-axis, i_z, 23.981 mm
Elastic section modulus about y-axis, $W_{el.y}$, 114793 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 26608 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 132089 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 46581 mm^3
Second moment of area about y-axis, I_y, 8609494 mm^4
Second moment of area about z-axis, I_z, 1309331 mm^4

Analysis results

Restraint spacing

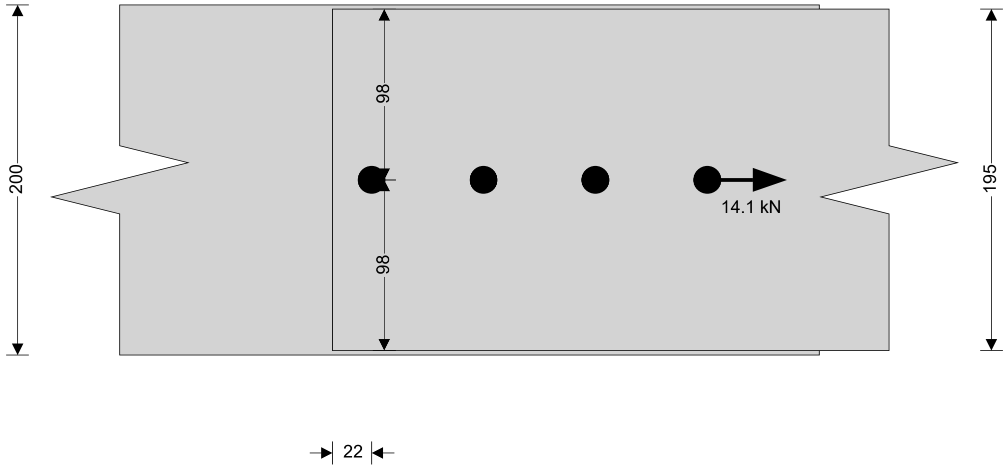
Major axis lateral restraint $L_y = \mathbf{0}$ mm

Minor axis lateral restraint $L_z = \mathbf{0}$ mm

Torsional restraint L_T = **0** mm

BOLTED TIMBER TO STEEL CONNECTION DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.10



Main timber member details

Main timber member thickness b_m = **100** mm

Main timber member depth h_m = **200** mm

Number of timbers in main member N_m = **1**

Strength class C24 timber (Table 8 BS5268:Pt 2:2002)

Connected steel plate details

Connected steel plate thickness b_c = **10.0** mm

Connected steel plate depth h_c = **195** mm

Number of steel plates connected N_c = **1**

Connection details

Angle of connected member α = **0°**

Load applied to connected member F = **14.070** kN

Duration of Loading **Long term**

Member service class **1**

Bolting details

Bolt diameter ϕ_b = **16** mm

Number of rows of bolts N_{rows} = **1**

Number of bolts per row N_{conns} = **4 (64 mm centres)**

Total number of bolts $N_{total} = N_{conns} \times N_{rows}$ = **4**

The bolts are aligned with the main member

Number of interfaces $N_{int} = (N_m + N_c) - 1$ = **1**

Check main member

Effective width of timber b_t = **100** mm

From BS 5268 : Part 2 : 2002 ..... Table 71

Basic bolt shear load parallel to loading F_{basic} = **5.103** kN

Modification factors

Steel to timber K_{46} = **1.25**

Moisture content K_{56} = **1.00**

Bolts in line with grain K_{57} = **0.91**

Capacity of bolted connection

Capacity of connection $F_m = N_{total} \times F_{basic} \times K_{46} \times K_{56} \times K_{57} \times N_{int} = \mathbf{23.220}$ kN

PASS - Connection capacity exceeds applied load

Minimum bolt spacings

Loaded end spacing $S_{end\_l} = 7 \times \phi_b = \mathbf{112}$ mm

Unloaded end spacing $S_{end\_u} = 4 \times \phi_b = \mathbf{64}$ mm

Loaded edge spacing $S_{edge\_l} = 4 \times \phi_b = \mathbf{64}$ mm

Unloaded edge spacing $S_{edge\_u} = 1.5 \times \phi_b = \mathbf{24}$ mm

Minimum bolt spacing $S_{bolt} = 4 \times \phi_b = \mathbf{64}$ mm

Washer details

Minimum washer diameter $\phi_w = 3 \times \phi_b = \mathbf{48}$ mm

Minimum washer thickness $t_w = 0.25 \times \phi_b = \mathbf{4.0}$ mm

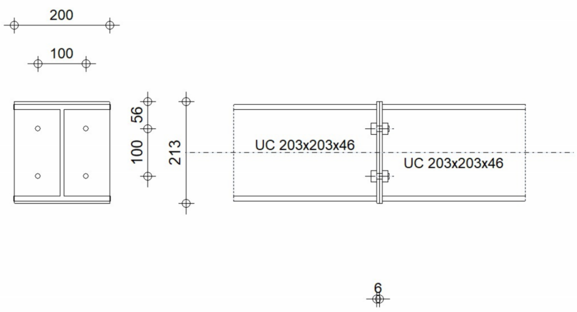
Robot Structural Analysis Professional 2024

Design of fixed beam-to-beam connection

EN 1993-1-8:2005/AC:2009

OK

Ratio **0.72**



GENERAL

Connection no.: 1
Connection name: Beam-Beam

GEOMETRY

LEFT SIDE

BEAM

Section: UC 203x203x46

α =	-180.0	[Deg]	Inclination angle
h_{bl} =	203	[mm]	Height of beam section
b_{fbl} =	204	[mm]	Width of beam section
t_{wbl} =	7	[mm]	Thickness of the web of beam section
t_{fbl} =	11	[mm]	Thickness of the flange of beam section
r_{bl} =	10	[mm]	Radius of beam section fillet
A_{bl} =	5870	[mm^2]	Cross-sectional area of a beam
I_{xbl} =	45680000	[mm^4]	Moment of inertia of the beam section

Material: S355

f_{yb} =	355.00	[MPa]	Resistance

RIGHT SIDE

BEAM

Section: UC 203x203x46

α =	0.0	[Deg]	Inclination angle
h_{br} =	203	[mm]	Height of beam section
b_{fbr} =	204	[mm]	Width of beam section
t_{wbr} =	7	[mm]	Thickness of the web of beam section
t_{fbr} =	11	[mm]	Thickness of the flange of beam section
r_{br} =	10	[mm]	Radius of beam section fillet
A_{br} =	5870	[mm^2]	Cross-sectional area of a beam
I_{xbr} =	45680000	[mm^4]	Moment of inertia of the beam section

Material: S355

f_{yb} =	355.00	[MPa]	Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d =	12	[mm]	Bolt diameter
Class =	8.8		Bolt class
F_{tRd} =	48.56	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	2		Number of bolt rows
h_1 =	56	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 100 [mm]
Vertical spacing p_i = 100 [mm]

PLATE

h_{pr} =	213	[mm]	Plate height
b_{pr} =	200	[mm]	Plate width
t_{pr} =	6	[mm]	Plate thickness

Material: STEEL 43-275

f_{ypr} =	275.00	[MPa]	Resistance

FILLET WELDS

a_w =	5	[mm]	Web weld
a_f =	5	[mm]	Flange weld

MATERIAL FACTORS

γ_{M0} =	1.00		Partial safety factor	[2.2]
γ_{M1} =	1.00		Partial safety factor	[2.2]
γ_{M2} =	1.25		Partial safety factor	[2.2]
γ_{M3} =	1.25		Partial safety factor	[2.2]

LOADS

Ultimate limit state

Case: Manual calculations.

$M_{b1,Ed}$ =	7.40	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	24.28	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	-35.74	[kN]	Axial force in the right beam

RESULTS

BEAM RESISTANCES

COMPRESSION

A_b = 5870 [mm^2] Area EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$

$N_{cb,Rd}$ = 2083.85 [kN] Design compressive resistance of the section EN1993-1-1:[6.2.4]

SHEAR

A_{vb} = 1694 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ = 347.28 [kN] Design sectional resistance for shear EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \le 1,0$ 0.07 < 1.00 verified (0.07)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} = 497000 [mm^3] Plastic section modulus EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd}$ = 176.44 [kN*m] Plastic resistance of the section for bending (without stiffeners) EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

W_{el} = 449606 [mm^3] Elastic section modulus EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{el} f_{yb} / \gamma_{M0}$

$M_{cb,Rd}$ = 159.61 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

BENDING WITH AXIAL FORCE ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

n = 0.02 Ratio of the axial force to the sectional resistance EN1993-1-1:[6.2.9.1.(5)]

$M_{Nb,Rd} = M_{cb,Rd} (1 - n)$

$M_{Nb,Rd}$ = 156.87 [kN*m] Reduced resistance (axial force) of the section for bending EN1993-1-1:[6.2.9.2.(1)]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ = 159.61 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

h_f = 192 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ = 830.44 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	41	-	50	-	100	256	259	256	259	228	196	196	196
2	41	-	50	-	100	256	225	225	225	228	163	163	163

m – Bolt distance from the web
m_x – Bolt distance from the beam flange
e – Bolt distance from the outer edge
e_x – Bolt distance from the horizontal outer edge
p – Distance between bolts
$l_{eff,cp}$ – Effective length for a single bolt row in the circular failure mode
$l_{eff,nc}$ – Effective length for a single bolt row in the non-circular failure mode
$l_{eff,1}$ – Effective length for a single bolt row for mode 1
$l_{eff,2}$ – Effective length for a single bolt row for mode 2
$l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode
$l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode
$l_{eff,1,g}$ – Effective length for a group of bolts for mode 1
$l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

$N_{j,Rd} = \mathrm{Min}(N_{cb,Rd})$

$N_{j,Rd}$ = 2083.85 [kN] Connection resistance for compression [6.2]

$N_{b1,Ed} / N_{j,Rd} \leq 1,0$	0.02 < 1.00	verified	(0.02)

CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd}$ = 48.56 [kN] Bolt resistance for tension [Table 3.4]
$B_{p,Rd}$ = 70.03 [kN] Punching shear resistance of a bolt [Table 3.4]

$F_{t,fc,Rd}$ – column flange resistance due to bending
$F_{t,wc,Rd}$ – column web resistance due to tension
$F_{t,ep,Rd}$ – resistance of the front plate due to bending
$F_{t,wb,Rd}$ – resistance of the web in tension

$F_{t,fc,Rd} = \text{Min} (F_{T,1,fc,Rd} , F_{T,2,fc,Rd} , F_{T,3,fc,Rd})$ [6.2.6.4] , [Tab.6.2]
$F_{t,wc,Rd} = \omega\, b_{eff,t,wc}\, t_{wc}\, f_{yc} / \gamma_{M0}$ [6.2.6.3.(1)]
$F_{t,ep,Rd} = \text{Min} (F_{T,1,ep,Rd} , F_{T,2,ep,Rd} , F_{T,3,ep,Rd})$ [6.2.6.5] , [Tab.6.2]
$F_{t,wb,Rd} = b_{eff,t,wb}\, t_{wb}\, f_{yb} / \gamma_{M0}$ [6.2.6.8.(1)]

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$F_{t,ep,Rd(1)} = 62.20$	62.20	Front plate - tension
$F_{t,wb,Rd(1)} = 654.33$	654.33	Beam web - tension
$B_{p,Rd} = 140.06$	140.06	Bolts due to shear punching
$F_{c,fb,Rd} = 830.44$	830.44	Beam flange - compression
$F_{t1,Rd} = \text{Min} (F_{t1,Rd,comp})$	62.20	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,ep,Rd(2)} = 54.79$	54.79	Front plate - tension
$F_{t,wb,Rd(2)} = 576.31$	576.31	Beam web - tension
$B_{p,Rd} = 140.06$	140.06	Bolts due to shear punching
$F_{c,fb,Rd} - \Sigma_1^1 F_{tj,Rd} = 830.44 - 62.20$	768.23	Beam flange - compression
$F_{t,ep,Rd(2+1)} - \Sigma_1^1 F_{tj,Rd} = 87.20 - 62.20$	24.99	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \Sigma_1^1 F_{tj,Rd} = 917.25 - 62.20$	855.05	Beam web - tension - group
$F_{t2,Rd} = \text{Min} (F_{t2,Rd,comp})$	24.99	Bolt row resistance

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1	146	62.20	-	-	62.20	654.33	97.11	140.06
2	46	24.99	-	-	54.79	576.31	97.11	140.06

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \Sigma\, h_j\, F_{tj,Rd}$
$M_{j,Rd}$ = 10.25 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \leq 1,0$	0.72 < 1.00	verified	(0.72)

CONNECTION RESISTANCE FOR SHEAR

α_v = 0.60 Coefficient for calculation of $F_{v,Rd}$ [Table 3.4]
$F_{v,Rd}$ = 43.43 [kN] Shear resistance of a single bolt [Table 3.4]
$F_{t,Rd,max}$ = 48.56 [kN] Tensile resistance of a single bolt [Table 3.4]
$F_{b,Rd,int}$ = 61.92 [kN] Bearing resistance of an intermediate bolt [Table 3.4]
$F_{b,Rd,ext}$ = 61.92 [kN] Bearing resistance of an outermost bolt [Table 3.4]

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1	97.11	-17.87	62.20	44.91	27.04	69.58
2	97.11	-17.87	24.99	18.05	0.18	86.75

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension
$F_{tj,Ed,N}$ – Force due to axial force in a bolt row
$F_{tj,Rd,M}$ – Bolt row resistance for simple bending
$F_{tj,Ed,M}$ – Force due to moment in a bolt row
$F_{tj,Ed}$ – Maximum tensile force in a bolt row
$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed}\ F_{tj,Rd,N} / N_{j,Rd}$
$F_{tj,Ed,M} = M_{j,Ed}\ F_{tj,Rd,M} / M_{j,Rd}$
$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$
$F_{vj,Rd} = \text{Min}\ (n_h\ F_{v,Ed} / (1 - F_{tj,Ed} / (1.4\ n_h\ F_{t,Rd,max})),\ n_h\ F_{v,Rd},\ n_h\ F_{b,Rd})$

$V_{j,Rd} = n_h\ \Sigma_1^n\ F_{vj,Rd}$ [Table 3.4]
$V_{j,Rd}$ = 156.33 [kN] Connection resistance for shear [Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \le 1,0$	0.16 < 1.00	verified	(0.16)

WELD RESISTANCE

A_w =	4332	[mm^2]	Area of all welds	[4.5.3.2(2)]
A_{wy} =	2724	[mm^2]	Area of horizontal welds	[4.5.3.2(2)]
A_{wz} =	1608	[mm^2]	Area of vertical welds	[4.5.3.2(2)]
I_{wy} =	25186718	[mm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max}$ =	-26.43	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_{\perp}=\tau_{\perp}$ =	-25.01	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
τ_{II} =	15.10	[MPa]	Tangent stress	[4.5.3.2(5)]
β_w =	0.85		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	52.85 < 404.71	verified	(0.13)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	56.45 < 404.71	verified	(0.14)
$\sigma_{\perp} \le 0.9*f_u/\gamma_{M2}$	26.43 < 309.60	verified	(0.09)

CONNECTION STIFFNESS

t_{wash} =	3	[mm]	Washer thickness	[6.2.6.3.(2)]
h_{head} =	9	[mm]	Bolt head height	[6.2.6.3.(2)]
h_{nut} =	12	[mm]	Bolt nut height	[6.2.6.3.(2)]
L_b =	34	[mm]	Bolt length	[6.2.6.3.(2)]
k_{10} =	4	[mm]	Stiffness coefficient of bolts	[6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	hj	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j}\ h_j$	$k_{eff,j}\ h_j^2$
1	146	∞	∞	1	0	39	5630
2	46	∞	∞	0	0	10	472
					Sum	49	6102

$k_{eff,j} = 1 / (\Sigma_3^5\ (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \Sigma_j\ k_{eff,j}\ h_j^2 / \Sigma_j\ k_{eff,j}\ h_j$
z_{eq} = 125 [mm] Equivalent force arm [6.3.3.1.(3)]

$k_{eq} = \Sigma_j\ k_{eff,j}\ h_j / z_{eq}$
k_{eq} = 0 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

$S_{j,ini} = E\ z_{eq}^2\ k_{eq}$ [6.3.1.(4)]
$S_{j,ini}$ = 1250.96 [kN*m] Initial rotational stiffness [6.3.1.(4)]

μ = 1.24 Stiffness coefficient of a connection [6.3.1.(6)]
$S_j = S_{j,ini} / \mu$ [6.3.1.(4)]
S_j = 1008.56 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.
$S_{j,rig}$ = 14983.04 [kN*m] Stiffness of a rigid connection [5.2.2.5]
$S_{j,pin}$ = 936.44 [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,pin} \le S_{j,ini} < S_{j,rig}$ SEMI-RIGID

WEAKEST COMPONENT:

FRONT PLATE - TENSION

REMARKS

Bolts vertical spacing is too large. 100 [mm] > 84 [mm]

Connection conforms to the code Ratio 0.72