

Structural Calculations

Relating to:

31-34 Commercial St, Newport NP20 1HJ

Document Prepared By:

Raisehold Limited

DATE: 24/02/2025 REVISION: 2

Job Reference: KU130125

The following calculations have been undertaken for the proposed works at the above address.

(All notes here in the document must be read in conjunction)

All the dimensions for structural elements included within the calculations are to be site checked by the contractor.

Do not use these drawings and calculations before building control approval.

The contractor is to ensure adequate temporary supports and propping is provided when the beams and structural elements are installed or removed. The contractor is to ensure overall stability of the structure is continuously maintained during the alteration works and if in doubt consult relevant parties.

THESE DOCUMENTS REQUIRE A COMPREHENSIVE REVIEW BY THE CLIENT AND BUILDER OR CONTRACTOR ALONGSIDE OTHER RELEVANT DOCUMENTS SUCH AS STRUCTURAL ENGINEERING NOTES AND SPECIFICATIONS, ARCHITECTURAL DRAWINGS, AND BUILDING REGULATION DRAWINGS.

IF ANY DISCREPANCIES ARE IDENTIFIED, THE ENGINEER MUST BE PROMPTLY NOTIFIED

General Notes and Guidance

To be read in conjunction with Architects drawings, any inconsistencies between the drawings should be reported. If any site conditions or existing details are found that may affect the structural design, Structural Engineer is to be notified immediately.

The dimensions provided on these drawings are for advisory purposes only. Please refer to the planning drawings for precise dimensions. All dimensions must be verified on-site by the contractor

The Contractor is to ensure that all existing construction is adequately supported, using needles and props as required. Where a new beam supports the existing construction, adequate pre-load is to be applied and suitable packs such as driven dry-slate introduced, then pointed up with mortar.

All steelwork to be mechanically wire brushed and painted two coats of red oxide. Steelwork located in the cavity or below DPC to be suitably protected with 2 coats of bituminous paint.

All steelwork connections to use grade 8.8 bolts unless stated otherwise. These are to be spanner tightened using the appropriate podger spanner (min length 460mm) or suitable power tools in accordance with BS2583. If a torque wrench is used the torque applied should be around 90Nm for M16 bolts, 110Nm for M20 & 130Nm for M24.

All timber to be grade C24 (SC4), unless stated otherwise. Preservative treated to Architects details.

For details of fire protection, see Architects drawings.

All blockwork to be 7.3 N/mm² in class III mortar below DPC in accordance with BS 5628 : Part 3 : 2005 or suitable 3.6 N/mm² foundation quality trench blocks in class II mortar in accordance with the manufacturer's instructions. All brickwork below DPC to be Engineering Bricks DPC in accordance with BS 5628 : Part 3 : 2005.

The project requires the introduction of heavy structural elements such as steel beams or concrete lintels. Through the Construction (Design and Management) Regulation 2015 The designer has an obligation to foresee risks and bring to the attention of the builder such risks. In consequence, the builder is to take into consideration the placement of all structural elements, ensuring that the method of lifting and placement is safely carried out. Responsibility for this element lies with the Contractor. As the existing walls need to be propped in order to introduce some of the lintels, this should also be considered in relationship to the risk assessment of the Contractor. Safe working procedures must be adopted. Responsibility for this element lies with the Contractor. Splice details for long-span beams can often be accommodated if required.

All construction products should be CE marked in accordance current legislation. This includes all fabricated structural steelwork in accordance with BS EN 1090-1 and BS EN 1090-2. The consequence class is CC2 unless noted otherwise. The service class is SC1 for all buildings, SC2 for all lifting beams, sculptures & fall arrest systems. Production category will be PC1 unless noted otherwise. All site welded items, S355 steelwork & CHS lattice girders will be PC2. As such the execution class for buildings will be EXC2.

CLIMATE CHANGE: The Building Research establishment have produced a document CBG 63 "Climate Change: impact on building design and construction". Part of their recommendations are that designers and builders should give consideration to:

- Increased wind loading by providing additional laps and fixings to roof coverings.
- Consider foundation depth on shrinkable clays and to avoid future problems, increase the depth above standard requirements if there is a risk. This should be in accordance with the NHBC Standards, Chapter 4.2 Guidance on Building near Trees. If the calculations do not specifically design the depths of the foundations to take into account any local trees, then this should be checked and agreed with the Building Inspector on site.

Party Wall etc. Act 1996: If part of the work is adjacent to the boundary, the adjacent neighbours right to support could be affected; the issues associated with Party Wall Act may need to be considered. This may include providing information to the adjoining owner, giving sufficient notice of works in compliance with the Act. If the following list applies to this project then the Party Wall Act will apply:

- Installing a new beam into the shared wall between properties
- Demolishing, building or under-pinning an existing shared wall
- Building a new wall at or on the boundary or junction of two properties
- Damp-proofing all the way through a party wall
- Digging foundations that are within 3m of a Party Wall, where the new foundations are deeper than the existing ones
- Where the new foundations are within 6m and lower than a 45° line from the bottom of the existing foundations

Construction (Design and Management) Regulations 2015: As a designer, I am required by the Construction (Design and Management) Regulations 2015 to ensure that you are aware of your duties as a client under these regulations. As a domestic client you are absolved of most duties, however should more than one contractor be involved in this project, then you should appoint in writing a lead contractor. Before that the lead designer in control of the preconstruction phase to be appointed in writing as principal designer. These principal designer duties are in excess of what you may be familiar with as a traditional design role, and the principal designer will be responsible for planning, managing and monitoring the preconstruction phase of a project in the same way that the principal contractor is responsible for planning, managing and monitoring the construction phase.

Structural Engineering Notes

1. Introduction

This document pertains to the construction work at the specified address and includes design calculations for structural elements, along with approximate schematic arrangements of those elements.

2. Definitions

- Engineer : SE (Structural engineer)
- Client : The individual or organization that has commissioned the engineer for structural engineering consultancy work.
- Architect : The individual or organization that provided the information used in these calculations.
- Builder : The contractor responsible for the construction work referred to in this document.

3. Important Guidance on the Use of This Document (To Be Read by All Parties)

This document should be used in conjunction with all relevant architectural and engineering drawings, and all pertinent documentation should be reviewed prior to the start of work. Engineer's drawings relevant to this document will be explicitly referenced. It is essential to view this document in color to avoid misinterpretation that may occur when viewed in black and white.

The entire document, along with any other relevant documentation, must be reviewed by the builder, architect (if applicable), and client before beginning the work. All layouts, instructions, and recommendations should be considered. Any deviations from the outlined proposals must be approved by the engineer before the work begins. Deviations made without the engineer's consent are beyond the scope of this document, and the engineer is not liable for any adverse outcomes from such deviations.

The specifications in this document are made in good faith based on the data provided by the client, builder, or architect. Relevant extracts of the provided information are included in this document. The architect (if applicable) or client must notify the engineer of any changes to the information so the design can be reviewed and adjusted as necessary.

Approval of these calculations and drawings by the Local Authority Building Control must be obtained before ordering materials or fabrication. No liability is accepted for changes required due to work commencing before such approval.

When information about existing building arrangements is unavailable (e.g., floor/roof span orientations or load-bearing wall arrangements), the engineer will make conservative assumptions, clearly outlined in the document. These assumptions should be confirmed on-site by a qualified individual. Any discrepancies must be reported to the engineer before work starts, as design changes may be necessary.

If drawings, construction specifications, method statements, or additional design calculations are omitted, it is because they were not requested by the client. These can be provided upon request.

IF IN DOUBT: ASK!!

It is often impossible to foresee the exact existing structural arrangements (e.g., floor and roof span directions, construction materials, existing beam locations) until the existing structure is exposed. It is advisable to remove finishes from areas being modified as early as possible to confirm this information and avoid delays. Early ground investigations are also recommended to prevent unexpected construction requirements that could cause delays, such as the need for piled foundations. Any uncertainties will be highlighted, but effective project planning and management by the contractor/client are crucial for smooth project execution.

4. Construction Notes

Scope of Service

SE is not responsible for any elements outside the scope of these calculations. It is assumed that designs for structural elements not covered in this document are being prepared or provided by others. If additional calculations, drawings, or specifications are needed, please contact SE for a fee proposal.

Beam Spans / Column Heights

Any span or height dimensions shown in this document are for calculation purposes only and should not be used as final dimensions for the fabrication or machining of structural elements. Beam spans and column heights must be measured on-site before fabrication.

Dimensions

All dimensions must be checked on-site by the builder, contractor, or fabricator before the commencement of fabrication, machining, or construction. Any discrepancies between the document and on-site dimensions must be reported to the engineer.

Architects Drawings

This document should be read in conjunction with architectural drawings and details. Any discrepancies should be reported to the engineer immediately. Final steel levels and dimensions must be confirmed by the architectural consultant.

Building Regulations

All structural work must be carried out by a competent builder in accordance with The Building Regulations Part A and the recommendations in BS8103 Parts 1-3.

Site Inspections

SE has not visited the site and therefore takes no responsibility for the quality of construction or its compliance with this document. It is the contractor's responsibility to ensure that all works comply with the drawings, notes, and assumptions made within these calculations.

Checking Existing Loadbearing Walls

Beam bearing information provided on the layout assumes the presence of load-bearing masonry with suitable foundations/support. This should be confirmed by a qualified individual before starting work. The masonry should be inspected for suitability.

Deflections

The client/contractor should be aware of beam, frame, and column deflections and their impact on existing or adjacent elements, such as beam deflection on bi-fold doors. Typical beam deflections are $\text{span}/200$ total (Dead+Live) for beams and $\text{height}/300$ (Wind) for columns. Anticipated deflections are provided in the relevant sections. SE should be notified if specific deflection criteria need to be met.

Potential Cracking of Existing Masonry / Plaster

Clients should be aware that minor cracking may occur within the masonry above installed beams due to load redistribution. Contractors should allow for suitable repairs within their contract.

Temporary Works

Temporary works are the sole responsibility of the builder/contractor. Temporary works method statements should be provided to the engineer by the builder/contractor before work begins.

CDM Regulations

All parties are assumed to be aware of their responsibilities under the Construction Design and Management (CDM) Regulations 2015. If unsure, please contact the engineer.

Proprietary Items

All proprietary items specified in this document must be installed according to the manufacturer's recommendations. This includes, but is not limited to, restraint straps, lintels, chemical/resin anchors, and fixing brackets.

4. Health & Safety

Structural work, especially involving the transit and installation of large, heavy structural elements, can be hazardous. Specific risks are identified within these calculations or related drawings where possible.

Risk	Description	Mitigation Measures
Lifting and Handling of Heavy Elements	Risk of injury from lifting heavy beams and columns.	Use appropriate lifting equipment, ensure proper training for all personnel involved.
Working at Heights	Falls from scaffolding, ladders, or raised platforms.	Use fall protection systems, secure scaffolding, ensure proper training and supervision.
Structural Stability During Construction	Risk of collapse or instability during the installation of elements.	Follow proper sequencing of work, use temporary supports where needed.
Crushing Hazards	Risk of being crushed by falling or shifting structural elements.	Ensure clear communication and use of proper lifting techniques, barriers, and spotters.
Exposure to Hazardous Materials	Risk of exposure to asbestos, lead, or other hazardous materials in existing structures.	Conduct thorough site surveys and use protective equipment and procedures.
Power Tool and Equipment Use	Risk of injury from the use of power tools and heavy machinery.	Provide proper training, use personal protective equipment (PPE), and follow safety protocols.
Electrical Hazards	Risk of electric shock from exposed wiring or equipment.	Ensure proper isolation of electrical systems, use insulated tools and PPE.
Fire Risk	Risk of fire during welding or cutting operations.	Have fire extinguishers on site, maintain a clean work area, follow hot work protocols.
Noise and Vibration	Risk of hearing damage from loud equipment and operations.	Use hearing protection and limit exposure to noisy environments.
Dust and Fume Exposure	Risk of respiratory issues from dust and fumes generated during construction.	Use dust suppression methods, provide respiratory protection.

	Description of Risk	Risk of Occurrence	Possible Consequences	Possible Mitigation Measures
1	Crushing due to falling structural elements	High	Death Serious Injury Damage to Property	a) Reduce weight of elements b) Find alternative method of construction c) Splicing of large / heavy structural elements to reduce handling weights d) Produce method statement for installation e) Use of suitable lifting equipment f) Ensure suitable temporary works in place
2	Collapsing trenches / banks or retained earth	High	Death Serious Injury	a) Stabilise earth using box shutters / raking shores / sheet piles or other during construction b) Restricting persons from working in deep trenches or adjacent to steep banks of un-retained earth
3	Rupture of concrete shuttering	Medium	Serious Injury Damage to Property	a) Ensure concrete pour heights of not more than 0.75m b) Ensure suitable shuttering in place
4	Fire from site welding	High	Death Serious Injury Damage to Property	a) Check for combustible materials in vicinity. Implement suitable precautionary measures e.g. removal or <u>shielding</u> of combustible <u>materials</u> b) Avoid <u>shot-firing</u> except where absolutely necessary
5	Fire from shot-firing	Medium	Death Serious Injury Damage to Property	a) Check for combustible materials in vicinity. Implement suitable precautionary measures e.g. removal or <u>shielding</u> of combustible <u>materials</u> b) Avoid <u>shot-firing</u> except where absolutely necessary
6	General site risk e.g. falls from height, falling objects, hazardous/heavy machinery etc.	High	Death Serious Injury Damage to Property	a) Use PPE (i.e. hard hat, gloves, goggles, h -viz clothing, earplugs, site boots et al) b) Implement general precautionary measures (i.e. installation of necessary barriers, signage, alarms) c) Conduct site-specific H&S assessments d) Produce Method Statements

4. Party Wall Agreements

If the building work involves:

- Work on an existing wall shared with another property,
- Building on the boundary with a neighboring property, or
- Excavating near a neighboring building,

then the client must determine whether the work falls under the Party Wall Act. If it does, the Act requires the client to notify all affected neighbours of the proposed works. For guidance, refer to your architect or visit [Party Wall Act 1996 Guidance] (<https://www.gov.uk/guidance/party-wall-etc-act-1996-guidance>).

Steelwork Notes

Steelwork Grade

All steelwork is of Grade 50B (Grade S355 to EN 10025: 1993). All bolts are grade 8.8 black, and all fillet welds are E35, unless noted otherwise.

Steelwork Levels

For steelwork levels, refer to the architect's drawings.

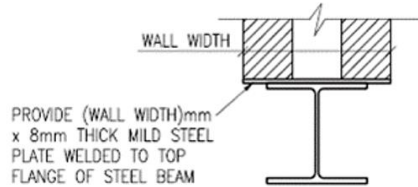
Column Levels

If a column is indicated as going from "ground floor level," this may also mean that the column needs to extend further below ground floor level to the top surface of a pad foundation, depending on the appropriate foundation type.

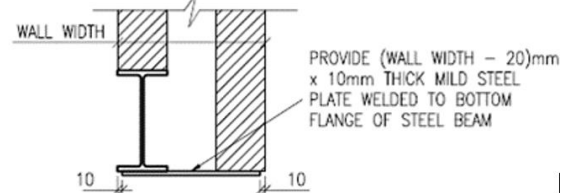
Beam Shelf Plates

Shelf plates will be required in certain instances as agreed with the architectural consultant and contractor. Shelf plates should extend for the full beam length (including bearings) and be grouted into the masonry bed joints at the bearings. The following are typical shelf plate arrangements:

a) Single Beams supporting cavity Walls



b) Beams acting as lintels in external walls



Clearance to Underside of Beams

Ensure that clearance is provided to the underside of steelwork at intersecting wall locations where no bearing information is shown. This is to prevent unintended load transfer to non-load-bearing walls.

Beam/Post Alignment on Bearings

Beams and posts must be seated centrally on padstones and posts unless otherwise noted. Generally, the beam location relative to the supported wall will be dictated by the padstone or post location as indicated on the element layouts. If there is any doubt about the location of the padstone or post, contact the engineer immediately. Final locations should be confirmed in conjunction with the architectural consultant and contractor. In

most instances, the beam will be central to the wall or cavity wall above, except in cases where it acts as a lintel in an external-facing cavity wall.

Beam Bearing Lengths

Unless otherwise noted in the design or layout information, beams should bear over the full width of any spreader or post. This does not apply when a supporting padstone runs in line with the beam; in such cases, refer to the shim detail in the padstone notes.

Bolt Fixings to Padstones

Where possible, beams should be bolted to the centerline of the supporting masonry wall, column, or padstone using 2 No. M12 chemically bonded anchors, and the padstone should be strapped down to the adjacent masonry using 2 No. mild steel restraint straps. **DO NOT BOLT AWAY FROM THE CENTER LINE OF THE MASONRY / PADSTONE.** Where the center line of the masonry or padstone is not accessible, the beam itself must be strapped.

Column to Wall Fixings

Columns or posts situated adjacent to a wall, as shown on the indicative layouts, must be fixed back to the wall. This can be achieved by either welding or shot-firing frame cramps to the web or flange at 450mm vertical centers and coursing into the masonry bed joints. Alternatively, bolt to the face of the wall by welding mild steel brackets to the flanges of the column at 450mm vertical centers and bolting through using chemically bonded M12 anchors.

Pairs of Beams

Where possible, beams installed in pairs should be bolted together through the center of the webs using M12 bolts at 500mm centers with spacer tubes in between.

Cranked Beams

Cranked elements specified should be full-strength butt-welded at the cranked joint by a suitably qualified steel fabricator. Joints should be tested in accordance with the relevant British or European standards.

Steel Fabricators Drawings

All steel fabricator's drawings and specifications must be forwarded to the engineer for approval prior to the commencement of fabrication.

Fabrication Specification

All steelworks must be fabricated and erected in accordance with the latest edition of the National Structural Steelwork Specification (NSSS).

Galvanizing / Painting

Steelwork finish and paint systems should comply with the recommendations of the Corus / Tata guide, "The Prevention of Corrosion on Structural Steelwork." For typical situations, the following is recommended:

- Internal steelwork in a heated building with a clean atmosphere: typically primer only.
- Steelwork in environments subject to moisture variations (e.g., within a cavity, with outer leaf shelf plates, or external steelwork): consider galvanizing to relevant European Standards (typically 85µm galvanized) or a paint applied system approved by Building Control.

Fire Protection

Fire protection of steelwork is to be specified by the architect. If intumescent painting is required (e.g., columns), the paint system must be compatible with the primer or underlying corrosion protection system.

Padstone Notes

Beams Sat Directly on Masonry (No Padstone)

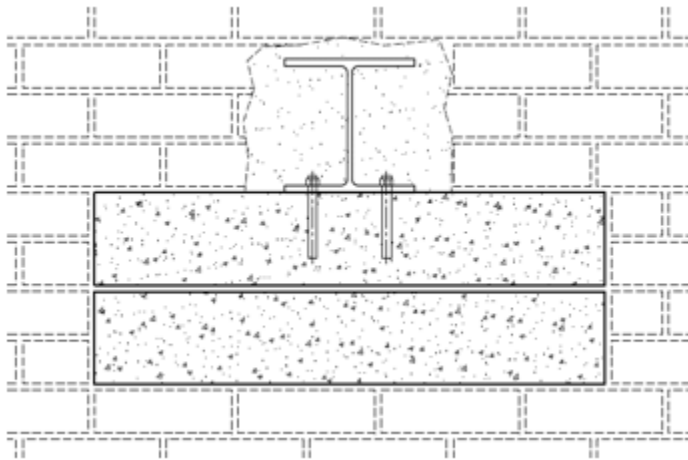
Where steel beams bear directly onto masonry without padstones, they must be bedded onto a dry, level mortar bed. Unless noted otherwise, all concrete blocks should be bedded on their narrow edge, i.e., not laid flat.

Old (Historic) Masonry

In many instances, historic buildings, particularly in the southeast, have poor quality masonry and degrading mortar capable of sustaining only a limited amount of compressive force. In such cases, the engineer should be notified, as the padstone sizes specified may need to be increased.

Concrete Padstone Options

All padstones specified should be C35 concrete. Where it is not possible to find "off-the-shelf" padstone sizes, it may be necessary to cast in-situ padstones or to cut/stack lintels to meet the required padstone dimensions. Lintels can be stacked as shown below if a padstone with significant depth is required (the example below shows two 800mm long, 140x100 pre-cast concrete lintels stacked on top of each other to create an 800x100x300 padstone). Lintels can be stacked to achieve the required depth but must not be positioned side-by-side under any circumstances.

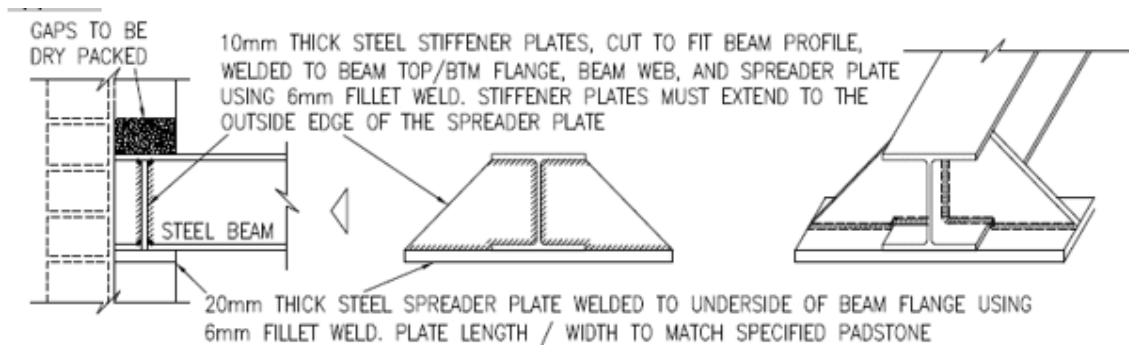


Steel Spreader Plates

Mild steel spreader plates may be used as an alternative to padstones, provided the following detail is applied to prevent the bearing plates from bending:

1. **Plate Thickness** : The spreader plates should be of sufficient thickness to distribute the load effectively without bending. The required thickness will depend on the load and the dimensions of the bearing area.

2. **Stiffeners** : Stiffeners must be welded to the spreader plates to prevent bending. These should be positioned to provide adequate support across the entire length and width of the plate.
3. **Material Specification** : The spreader plates and stiffeners should be made from mild steel, typically complying with Grade S275 or higher.
4. **Installation** : Ensure the spreader plates are installed level and firmly bedded onto the supporting masonry or concrete. Use a dry, level mortar bed to achieve proper seating and load distribution.
5. **Approval and Design** : All details regarding the use of steel spreader plates, including the design and placement of stiffeners, must be approved by the engineer prior to fabrication and installation.

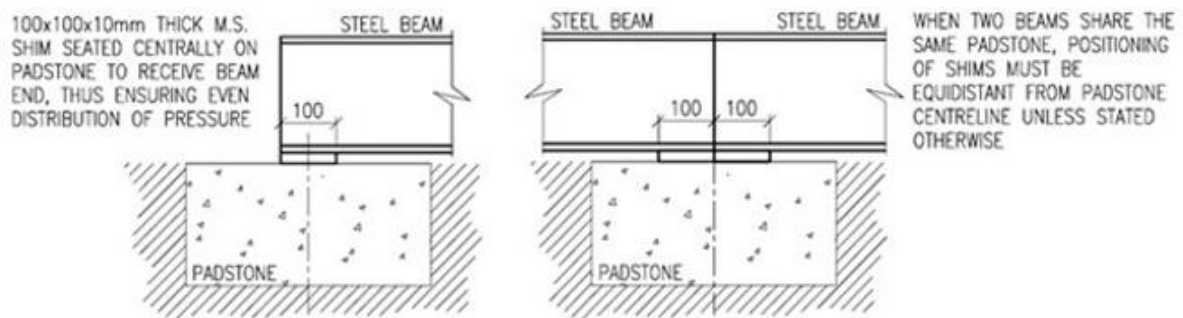


Steel Shims

When beams are oriented parallel to the padstone, mild steel shims must be used to ensure even pressure distribution on the padstone. The following detail should be applied:

1. **Shim Material** : Shims should be made from mild steel, typically complying with Grade S275 or higher.

2. **Thickness and Size** : The thickness and size of the shims should be appropriate to distribute the load evenly across the padstone. Multiple shims may be stacked to achieve the required height, ensuring stability and even load distribution.
3. **Placement** : Shims should be placed directly under the beams, aligned with the load-bearing area. They must cover the full width of the beam and be evenly spaced to prevent uneven pressure on the padstone.
4. **Welding** : Shims should be welded to the beam to prevent any movement during load application. Ensure the welds are strong enough to withstand the imposed loads.
5. **Engineer Approval** : The use of steel shims, including the specific thickness, size, and placement, must be approved by the engineer prior to installation.



Padstone Dimensions

Padstone specifications are denoted as follows: 440 x 100 x 215 (LENGTH x WIDTH x HEIGHT).

Padstone Width ("(wall width)"): If the padstone width is referred to as "(wall width)", it means that the padstone width should equal the full supporting wall width, including the cavity where applicable. In no instance should the padstone width be less than 200mm.

Padstone Width Restrictions

Padstones should never be wider than the wall (or single leaf of a cavity wall) that supports them. If the supporting wall width is found to be less than the specified padstone width, notify the engineer immediately.

Masonry Notes

Brickwork / Blockwork / Mortar Strength

- Facing bricks: Use standard formal clay 20N/mm² bricks unless otherwise specified.
- Concrete blocks: 3.5N blocks are recommended unless otherwise specified in the layout, with 7N strength blocks recommended for areas subjected to high loads (such as beneath heavily loaded beams).
- Mortar: Above DPC should be designation (iii) M4 and below DPC should be designation (ii) M6.

Old (Historic) Existing Masonry

In historic buildings, particularly in the southeast, masonry and mortar may be of poor quality and capable of sustaining only limited compressive force. Notify the engineer if this is the case, as padstone sizes may need adjustment.

Existing Internal Loadbearing Walls

When using existing internal masonry walls to support steel beams, ensure the wall thickness is at least 100mm. Do not use 75mm thick walls to support steel beams.

Masonry Quality

If existing masonry is of poor quality or the mortar has degraded, the brickwork should be repointed or replaced as appropriate.

Orientation of Blockwork

Unless otherwise noted, all concrete blocks should be bedded on the narrow edge, not laid flat.

Restraint Straps

Provide horizontal and vertical restraint strapping to floors and roofs as indicated in Building Regulations Part A "Lateral Support by Roofs and Floors" (Diagrams 15 and 16).

Movement Joints

Movement joints should be provided at maximum centres of 6m in blockwork and 12m in brickwork. Joint locations should follow the architect's recommendations, with a preference for positioning adjacent to wind-posts, columns, or buttressing walls. Reduce joint spacing at points of restraint such as cranks and corners. If the recommended joint centres cannot be achieved, use bed joint reinforcement at 450mm vertical centres.

Timber Notes

Grade Strength

All timber should be Grade C24 to BS5268:2-2005 unless otherwise noted. Timber should be factory treated in accordance with BS5268 'Code of Practice for the Preservation Treatment of Structural Timber'. Ensure such treatment does not harm any fixings in contact with the timber members.

Timber Beams (Built-up Beams)

When specifying two or more pieces of timber together as constituent parts of the same member, bolt the timbers together along the vertical centreline using M12 bolts at 500mm centres.

Joist Hangers

Unless fully built into masonry, support all new timber joists using hangers with side flanges to prevent rotation. Alternatively, provide full depth blocking at the ends of the joists.

Bearing Lengths

For timbers built into masonry, ensure an end bearing length of no less than 100mm.

Beam to Post Fixings

When supported by timber posts, ensure the bearing covers the full width of the post. Position beams centrally on the posts unless otherwise noted.

Notches

Notify the engineer if members are notched at the supports to a depth greater than $\frac{1}{3}$ of the member's depth.

Post Restraint Straps

At the top and bottom of timber columns/posts, provide a minimum of 2 horizontal restraint straps and noggins running orthogonally to the joist/rafter for restraint. These straps should be at least 1200mm long, 30mm wide, and 5mm thick, made of galvanized mild steel or a similar proprietary product. Straps must be securely fixed to both the column and noggin.

Post to Masonry Fixings

For columns/posts adjacent to existing masonry, use M12 Hilti-HY70 anchors (or similar) resin-anchored into the masonry at 450mm vertical centres. Alternatively, angle brackets may be used for direct fixing if preferred.

Prevention of Unintended Load Transfer

In loft conversions, ensure there is a 25mm gap between the top of the existing ceiling joist and the underside of the steel beam to prevent unintended load transfer.

Joints

Create joints between members using either traditional joinery techniques or proprietary fixings. Contact the engineer if additional input is required.

Rafter / Roof Joist Sizes

Insulation and ventilation requirements are not considered in this document when specifying joist/rafter sizes. The mentioned sizes may need to be increased in depth to accommodate the necessary insulation and ventilation. Refer to the architect's details for minimum joist/rafter depth requirements.

Roof Triangulation

If no beam is shown at the ridge position in the sloping roof structural layout, ensure roof triangulation by providing 1250mm M.S. restraint straps between rafters and ceiling/floor joists at intervals not exceeding 1000mm. Where joists run parallel to the wall plate, provide noggins between joists. At beam/wall locations where joists are interrupted, provide restraint straps between joists to maintain continuity. Additionally, nail or screw boarding to all joists at intervals not exceeding 450mm.

SPECIAL NOTES TO THE CONTRACTOR

(These notes must be read in conjunction with the structural specification for the works and the notes on all Raisehold Limited drawings)

Temporary propping and restraint of the existing building or structure is the sole responsibility of the main contractor and must be maintained for the duration of the construction works. Temporary works load will be provided on request by the contractor. This means that when preparing a tender price, the contractor should allow adequate monies for a complete and competent temporary works design. The design calculation and detailing of all steelwork connections is the responsibility of the main contractor. Calculations and details must be issued to Raisehold Limited and the Architect for comment, then incorporated into the fabrication drawings. Connection loads will be provided on the construction drawings. This means that the contractor tendering for the works should ensure that the steel fabricator can prepare fabrication drawings and competent calculations for the connection design. All connections of box frames, cantilevers, and splice connections must use either HSFG bolts or ordinary bolts with close tolerance holes (maximum shank diameter plus 0.3mm). Contact surfaces of HSFG bolted connections must be shot blasted and unpainted. No site welding is permitted without prior written authorization by Raisehold Limited.

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1.0 GENERAL

1.1 This specification is to be read with Preliminaries/General Conditions.

1.2 Do not scale drawings. The Contractor is to check all dimensions on site before carrying out any works.

1.3 This specification, together with the Structural Engineer's drawings, is to be read in conjunction with the Architect's and all other Consultant's drawings and specifications, which should be used to verify layout, setting out, finishes, etc. Any discrepancies are to be reported to the Architect before proceeding with the works.

1.4 The Contractor must ensure that all necessary party wall/party floor notices have been agreed prior to carrying out works under, on, or adjacent to party walls, floors, boundary walls, or other boundary structures.

1.5 The works described and specified on the drawings and associated drawings shall be undertaken in accordance with all current health and safety legislation.

1.6 The contractor shall, at the outset, establish with the local authority their requirements for inspecting the works and adhere to these.

1.7 All dimensions and levels shown on the drawings are based on survey or archive drawings, and the contractor is to satisfy himself that dimensions, levels, etc., are sufficiently accurate and complete for fabrication within the specified tolerances of all prefabricated elements.

1.8 Setting out details are shown on the Architect's drawings unless noted otherwise on the drawings.

1.9 The Contractor is to inform the Architect and Structural Engineer if the existing fabric/structure, including foundations, is opened and found to be inadequate, unsuitable to support the proposed works, or at variance from the details shown on the drawings.

1.10 Items noted on the drawings "to be verified on site" are to be exposed by the Contractor for inspection by the Structural Engineer at the earliest opportunity.

1.11 Holes or chases must not be cut through any structural members without the written consent of the Structural Engineer.

1.12 The Contractor is to ensure that the Building Control Officer is notified to carry out inspections prior to concreting new foundations, slabs, etc., and other structural items prior to them being covered up.

1.13 Fixings for lifts, signs, balustrades, etc., which have been designed by others, are to be installed in accordance with the manufacturer's details and specifications.

1.14 The contractor shall allow for drawing, detailing, and scheduling of all concrete reinforcement, based on the typical details and reinforcement quantities provided by Raisehold Limited.

1.15 Nothing included or omitted from this outline specification will relieve the Contractor of his duty to carry out the works in accordance with current standards of safety and good building practice.

2.0 TOLERANCES

2.1 All tolerances are to be agreed with the Architect, and the Contractor will be responsible for ensuring that sufficient tolerances are provided and integrated throughout all elements of the works.

2.2 The Contractor is to take account of tolerances detailed elsewhere in the drawings, appended specifications, and British Standards when complying with the above clause. Construction details are to take account of fabrication tolerance, erection tolerances, material tolerance, and movement/deflection of the structural elements.

3.0 MATERIALS AND WORKMANSHIP

3.1 All articles, materials, and goods shall be new and of good quality, suitable for the required purpose, and shall conform to the appropriate British Standard where such exists. Where references to the above are made, it shall be inferred that the latest edition applies, together with subsequent amendments, unless otherwise specified.

3.2 The following deleterious materials shall not be used in any part of the works:

- (a) High alumina cement or concrete
- (b) Wood slabs as permanent formwork or in structural elements
- (c) Concrete or mortar additives containing calcium chloride
- (d) Aggregates for use in reinforced concrete which do not comply with British Standard specification 882:1992 and aggregates for use in concrete which do not comply with the provisions of British Standard 8110:1985
- (e) Calcium silicate bricks or tiles
- (f) Any products or materials containing urea formaldehyde foam or materials which may release formaldehyde in quantities which may be hazardous with reference to the limits at the time of the specification by the Health and Safety Executive
- (g) Asbestos or asbestos-containing products whatsoever
- (h) Lead or any materials containing lead which may be ingested, inhaled, or absorbed, except where copper alloy fittings containing lead are specifically required in drinking water pipework by any relevant statutory requirements
- (i) Slip bricks
- (j) Lightweight or air-entrained concrete bricks and blocks
- (k) Vermiculite plaster
- (l) Polyisocyanurate foam
- (m) Extruded polystyrene other than low ozone depletion materials
- (n) Bitumen coated polythene; the use of bitumen coated polythene may be permitted for tanking and DPM but express permission must be sought from the beneficiary
- (o) Concrete that may be susceptible to alkali/silica reaction (ASR)
- (p) Proprietary open web lattice joists or beams (nailable type)
- (q) All tropical rain forest hardwoods
- (r) Any products which contain or use Montreal listed CFC gases in their manufacture

(s) PTFE fabrics— only permitted use of PTFE is as jointing tape in plumbing applications such as valve seats

(t) Resin-coated blocks

(u) Other substances or materials which the contractor in business in the United Kingdom at the time of the specification, using reasonable skill, care, and diligence, ought to know to be deleterious

(v) Other substances or materials not in accordance with British Standards or codes of practice where such exist at the time of specification

The contractor shall use all reasonable skill, care, and diligence to ensure that the above materials are not used in any part of the works/project.

4.0 TEMPORARY WORKS AND STABILITY

4.1 The Contractor is entirely responsible for maintaining the stability of all existing buildings and structures, within and adjacent to the works, and of all the works from the date of possession of the site until practical completion of the works.

4.2 The Contractor shall design, install, and maintain all necessary temporary works and shall advise both the Architect and Structural Engineer at least ten working days from the commencement of the works, of his proposals for temporary supports and sequence of construction for the works. These proposals shall be supported by design calculations if requested. The contractor shall clearly indicate all provisions and costs related to this for temporary works design and installation at the tender stage. Under no circumstances will any structural alterations be carried out prior to the Structural Engineer commenting on the Contractor's temporary works proposals.

4.3 The design of temporary works shall include an assessment of the loads to be resisted and is to be undertaken by a competent person. Due regard shall be given to lateral stability as well as to the support of vertical loads.

4.4 The Contractor is to familiarise himself with the building and its structure so that he is aware of the nature and magnitude of the loads to be supported.

4.5 Care is to be taken to ensure that temporary props remain adequately seated and tightened so that support to the structure above is not allowed to yield during building operations.

4.6 The Contractor is to ensure that any temporarily propped structure is adequately wedged, pinned, or packed off the permanent works using suitably sized and spaced steel shims fixed in position prior to removal of any temporary supports. In addition, the contractor shall ensure that loads are transferred from temporary supports/props to the permanent structure without resulting in excessive deflection/movement of the temporary or permanent structure. The Contractor shall assess the need for jacking or pre-deflection of permanent structural members prior to transferring loads from temporary supports/props to ensure that deflections of the permanent structure, when loaded

, do not result in excessive deflection due to slack connections or any damage (including cracking) to any of the permanent structures or buildings, etc. This requirement is especially significant when dealing with cantilever beams/structures and corner box frames.

4.7 The Contractor shall ensure that any completed or partially completed structural element is not overloaded. Details of design loads may be obtained from the Structural Engineer upon request.

4.8 All temporary works to support the sides of excavations for new foundations shall be designed in accordance with BS 8000 Part 1: 1989 and any other approved documents.

4.9 Excavations shall in no circumstances encroach within 45° of the bottom near side of any existing footing.

5.0 DEMOLITION

5.1 Demolition is to be carried out to and in accordance with BS 6187: 1982, Health and Safety Executive Guidance Note GS 29/1 paragraph 32, and any other relevant statutory undertakings or regulations.

5.2 Demolition is to be undertaken in the reverse order of construction. No part of the structure is to be left in an unsupported condition overnight or for long periods.

5.3 Demolition is to be undertaken in a manner that avoids excessive noise and nuisance. All work is to be well-watered to minimise dust. All material is to be removed from site as soon as practicable.

5.4 The contractor shall ensure that all noise and vibration are minimised and shall ensure that the works do not cause damage or movement to buildings/structures in the vicinity.

6.0 EXCAVATING AND FILLING

6.1 Where applicable, the soil type, foundation depths, etc., are shown on the drawings or recorded in the site investigation report (if a site investigation report or trial pit information has been provided for this project). Refer to the project Structural Engineer for confirmation of the extent of investigative work already completed and any further investigative work required.

6.2 Ground water level on the site is not known unless noted on drawings or trial pit records or in a site investigation report.

6.3 Inspect all available drawings and make enquiries about existing services on site. Verify positions and depth of all services before commencement of work on site. Services that are being retained during any phase of the works are to be protected.

6.4 Before starting work, verify with the Architect which existing fences, gates, walls, paved areas, trees, shrubs, hedges, bushes, and any other site features are to be removed. Materials arising are to be removed from site.

6.5 Workmanship for excavating to comply with BS 8000: Part 1, sections 3.1, 3.2, and 3.3.

6.6 Where an excavation encroaches below a line drawn at an angle of 45° from the horizontal from the nearest formation level of another higher excavation, the lower excavation, all work within it, and backfilling thereto must be completed before the higher excavation is made.

6.7 Make advance arrangements with the Building Control Officer and/or Architect for inspection of foundations and trenches requested at the beginning of the works. Remove the last 150mm of excavations just before inspection. Trim excavations to required profiles and levels and remove all loose materials. Unless otherwise instructed, seal formations within four hours of inspection with concrete or another specified fill.

6.8 Backfill any excavations for foundations taken deeper than required with lean mix concrete. Excavations other than foundations taken deeper than required may be backfilled with well-graded granular material.

6.9 Hardcore to be granular material, free from harmful matter, well-graded, passing a 75mm BS sieve, and one of the following:

- (a) Crushed concrete, brick, or tile, free from old plaster.
- (b) Gravel.

6.10 Spread and level both backfilling and general filling in layers not exceeding 150mm. Thoroughly compact each layer with a vibratory roller, vibrating plate compactor, vibro-tamper, power rammer, or other suitable means appropriate to the area being worked.

6.11 Hardcore under bearing concrete slabs to be as above and not less than 150mm thick, unless noted otherwise on the drawings. Excavate extra material as necessary. Increase thickness of hardcore as necessary to make up levels from stripped site levels to the underside of slabs. Dig out areas of soft material and backfill with hardcore compacted in 150mm thick layers.

6.12 Surfaces over hardcore to receive sheet overlays or concrete to be blinded with sufficient sand or fine gravel to fill interstices and provide a close, smooth surface (50mm min thickness), unless noted otherwise on the drawings. Permissible deviations on surface level to be +0 -15mm. Provide gauge 1200 polythene curing membrane on top of blinding prior to casting concrete (note: this polythene is not intended to act as a DPM; refer to Architect's details for DPM). Gauge 1200 polythene sheet is to lap a minimum of 300mm at joints and be sealed.

6.13 Minimum void under suspended ground floors to be 225mm for precast concrete and 300mm for timber floors. Suspended in-situ concrete floors shall be cast over a suitable void former to provide a void of 150mm under the slab, unless noted otherwise on the drawings.

7.0 FOUNDATIONS

7.1 Foundations are to be cast to the profiles indicated on the drawings. They are to be cast symmetrically about piers, stanchions, or walls, unless noted otherwise on the drawings.

7.2 New foundations have been designed to impose a safe bearing pressure of 100kN/m² on clay/undisturbed soil (unless noted otherwise on drawings) 1100mm below final/existing external ground level—whichever is the deepest. The Contractor is to ensure that all new foundations bear a minimum of 150mm into firm clay/undisturbed soil. Where poor ground conditions are encountered or the Building Control Officer requests amendments, the Structural Engineer is to be notified immediately.

7.3 Where tree roots are encountered, foundations are to extend a minimum of 600mm below the last trace of any root activity. Where the foundations are sited within a zone 1.25x mature tree height and the bearing strata is a shrinkable soil (e.g., clay), the foundation depths shall be as a minimum that which is specified in the NHBC Standards "Building near Trees". The Contractor is required to familiarise himself with this document and allow for the appropriate foundation depths in his quotation for the contract.

7.4 Foundations sited within a zone 1.25x mature tree height for high water demand trees, 0.75x mature tree height for moderate water demand trees, and 0.5x mature tree height for low water demand trees which are to remain or to be removed, or over trees after removal, are to be protected against clay heave as follows:

(a) Trench Fill Foundations: Provide compressible material against the inside face of all external wall foundations greater than 1.5m deep to give a 35mm void. The compressible material is to be positioned 500mm above the bottom of the foundation.

(b) Pier and Ground Beam Foundations

Provide compressible material against faces of all pier foundations greater than 1.5m deep to create a 35mm void. Position the compressible material 500mm above the bottom of the foundations. Additionally, provide compressible material or a void former below all ground beams and against the inside face of external ground beams to create a 150mm and 35mm void respectively.

(c) Pile and Ground Beam Foundations

Provide compressible material or a void former below all ground beams and against the inside face of the external ground beams to create a 150mm and 35mm void respectively. Piles are to be designed in accordance with the performance piling specification. The contractor must consider that surrounding buildings are at risk of being undermined by the piling works. During construction, ensure that any open bores do not collapse or cause ground movements that could impact the stability of surrounding buildings and structures. This must be factored into the contractor's design to eliminate the risk of ground movements affecting any surrounding buildings or structures.

Where construction joints in trench fill foundations are unavoidable, they should not be positioned near a return in the foundation. Construction joints should be formed using vertical shutters with 4 No. 20mm diameter, 1200mm long high tensile dowel bars projecting through, cast 600mm into the first pour. Apply a retarder to the face of the shutter. After 24 hours, strip the shutter and clean the concrete face to expose clean aggregate ready for the next pour.

7.6 The contractor is to ensure that the Building Control Officer is notified for inspection of the bottom of all foundations prior to concreting.

8.0 INSITU CONCRETE

Materials and workmanship are to comply with BS 8110.

Concrete for new foundations is to be designated mix FND 2 to BS 5328, suitable for Class 2 Sulphate conditions, unless noted otherwise on the drawings.

Concrete for reinforced concrete structures, including ground bearing slabs, is to be designated mix RC3S to BS 5328 unless noted otherwise on the drawings.

Concrete for the encasement of steel beams and for padstones is to be GEN 3 to BS 5328 with 10mm maximum aggregate and 260 kg/m³ of cement unless noted otherwise on the drawings.

DESIGNATED MIX FOR INDOOR REINFORCED CONCRETE

- (a) Mix RC3S to BS 5328
- (b) Max W/c ratio 0.6
- (c) Minimum cement content 300 kg/m³
- (d) Nominal maximum size of aggregate: 20 mm

DESIGNATED MIX FOR OUTDOOR EXTERNAL EXPOSED REINFORCED CONCRETE

- (a) Mix RC4S to BS 5328
- (b) Max W/c ratio 0.5
- (c) Minimum cement content 350 kg/m³
- (d) Nominal maximum size of aggregate: 20 mm

DESIGNATED MIX FOR UNREINFORCED CONCRETE

- (a) Mix ST4 to BS 5328
- (b) Minimum cement content 275 kg/m³

DESIGNATED MIX FOR BURIED REINFORCED CONCRETE (SULPHATES CLASS 1)

- (a) Mix RC3S to BS 5328
- (b) Cover 40mm
- (c) Max W/c ratio 0.6
- (d) Minimum cement content 300 kg/m³
- (e) Minimum strength 30 N/mm²
- (f) Characteristic strength 35 N/mm²

DESIGNATED MIX FOR BURIED REINFORCED CONCRETE (SULPHATES CLASS 2)

- (a) Mix RC3S to BS 5328
- (b) Cover 40mm
- (c) Cement OPC
- (d) Max W/c ratio 0.50
- (e) Minimum cement content 330 kg/m³
- (f) Minimum strength 30 N/mm²
- (g) Characteristic strength 35 N/mm²

8.10 DESIGNATED MIX FOR BURIED REINFORCED CONCRETE (SULPHATES CLASS 3)

- (a) Mix FND3 to BS 5328
- (b) Cover 40mm
- (c) Cement SRPC
- (d) Max W/c ratio 0.50
- (e) Minimum cement content 330 kg/m³
- (f) Minimum strength 35 N/mm²
- (g) Characteristic strength 40 N/mm²

8.11 DESIGNATED MIX FOR BURIED REINFORCED CONCRETE (SULPHATES CLASS 4)

- (a) Mix FND4 to BS 5328
- (b) Cover 40mm
- (c) Cement SRPC
- (d) Max W/c ratio 0.45
- (e) Minimum cement content 370 kg/m³
- (f) Minimum strength 40 N/mm²
- (g) Characteristic strength 45 N/mm²

8.12 SUBSTITUTION OF STANDARD FOR DESIGNATED MIXES:

- (a) Where appropriate, standard mix(es) to BS 5328: Part 2, Section 4 will be permitted in substitution for specified designated mixes in accordance with BS 5328: Part 1, Table 13, and subject to approval.
- (b) If standard mixes are made on-site, comply with BS 8000: Section 2.1, Subsections 2, 3, and 4.

8.13 EQUIVALENT GRADE DESIGNED MIX FOR INDOOR REINFORCED CONCRETE

- (a) To the relevant clauses of BS 5328: Parts 2, 3, and 4.
- (b) Equivalent grade as BS 5328: Part 1, clause 8.5: C3S
- (c) Nominal maximum size of aggregate: 20 mm
- (d) Aggregates:
 - (i) Coarse: To BS 882
 - (ii) Sand: To BS 882
- (e) Cement: OPC to BS 12
- (f) Minimum cement content: 300 kg/m³
- (g) Maximum free water/cement ratio: 0.6

- (h) Admixtures: Do not use any which provide water reduction.
- (i) Maximum total percentage of chloride ion by mass of cement: 0.4%
- (j) Rate of sampling for compressive strength testing: One sample per 6 cu m or 1 batch, whichever represents the lesser volume, but not less than one for each day of use.
- (k) Information to be provided by the producer: As BS 5328: Part 3, clause 3.

8.14 EQUIVALENT GRADE DESIGNED MIX FOR OUTDOOR REINFORCED CONCRETE

- (a) To the relevant clauses of BS 5328: Parts 2, 3, and 4.
- (b) Equivalent grade as BS 5328: Part 1, clause 8.5: C4S
- (c) Nominal maximum size of aggregate: 20 mm
- (d) Aggregates:
 - (i) Coarse: To BS 882
 - (ii) Sand: To BS 882
- (e) Cement: OPC to BS 12
- (f) Minimum cement content: 350 kg/m³
- (g) Maximum free water/cement ratio: 0.5
- (h) Admixtures: Do not use any which provide water reduction.
- (i) Maximum total percentage of chloride ion by mass of cement: 0.4%
- (j) Rate of sampling for compressive strength testing: One sample per 6 cu m or 1 batch, whichever represents the lesser volume, but not less than one for each day of use.
- (k) Information to be provided by the producer: As BS 5328: Part 3, clause 3.

Ready-mix concrete is to be used unless otherwise allowed by the Structural Engineer. This must be obtained from a plant that holds a current Certificate of Accreditation under the Quality Scheme for Ready Mix Concrete. Details of cement type, aggregate grading, and sources, with chloride and sulphate content of mixes, must be submitted to the Structural Engineer for approval prior to ordering any concrete. The total chloride ion content of concrete from aggregates, cement, and any other constituent material shall

not exceed the limits shown below when expressed as a percentage of chloride ion by mass of cement (inclusive of PFA or GGBFS when specified):

- (a) Prestressed concrete or heat-cured concrete: 0.2%
- (b) Concrete made with cement complying with BS 4027 or BS 4248: 0.2%
- (c) Plain or reinforced concrete made with cement complying with BS 12, BS 1370, or composite cements or combinations with GGBFS or PFA when specified: 0.4%

8.16 The use of site-mixed concrete for structural elements may only be used following the written approval of the Structural Engineer. Batching and mixing equipment will need to comply with BS 1305 and BS 4251.

8.17 The contractor is responsible for the design and installation of all formwork. Design and striking of the formwork are to be in accordance with BS 8110.

Do not place concrete when the ambient air temperature is less than 5°C.

8.19 All holes shall be formed, and all inserts cast in at the time of pouring concrete. No part of the concrete works shall be drilled or cut away without the approval of the Structural Engineer.

8.20 Reinforcement shall be:

- (a) Deformed bars to BS 4449:2005, grade 500 (high yield) type 2, prefix B on drawings and schedules.
- (b) Mesh to BS 4483.

Reinforcement shall be fixed adequately using tying wire

or steel clips. Concrete cover is to be as specified on the drawings. Chairs and spacers are to be provided by the contractor as necessary to maintain the specified cover.

Unless noted otherwise on drawings, all reinforcement is to be lapped 50d (where d is the diameter of the smaller bar).

8.23 The rate of sampling for compressive testing of concrete is to be agreed with the Structural Engineer prior to the commencement of any concrete works.

8.24 Use mechanical vibration to fully compact concrete for structural elements. Compact concrete to full depth (until air bubbles cease to appear on the top surface), especially around reinforcement, cast-in accessories, into corners of formwork, and at joints.

Before placing structural concrete (not blinding) on hardcore or other absorbent substrates, lay building paper to BS 1521 Class 3 or polythene sheet 250 microns thick. Lap edges 150mm. This is not an OPM; see architect's details for this.

8.26 Joints in Beams and Slabs

(a) At joint locations, provide straight and vertical shutter with all longitudinal reinforcement projecting through a minimum of 45 x bar diameter.

(b) Apply proprietary retarder to the inside face of the shutter and 24 hours after casting, strike the shutter and clean off the concrete face and reinforcing bars to expose a clean aggregate surface ready for the next pour.

(c) Wet the exposed concrete surface immediately prior to the next concrete pour.

8.27 The contractor is responsible for the production of drawings, detailing, and scheduling of all reinforcement drawings based on the typical details and reinforcement quantities provided by Raisehold Limited.

Where existing reinforced concrete is to be broken out, all remaining edges and surfaces must be protected with the following. The following specification is for internal use only. External, cavity, or exposed surfaces must be protected in accordance with all Fosroc recommendations. For internal surfaces where concrete is flush with the proposed surface, or no infill is required, paint exposed surfaces of steel reinforcement bars with a blob of galvafruid immediately after cutting concrete. Next, roughen the surface of the concrete with a needle gun, dust off, and immediately apply a 1.0mm thick coat of Fosroc Nitocote CM210 applied with a roller (white). For areas that require filling, use Fosroc Renderoc HB25 applied as follows:

- (a) Break out the complete repair area to a minimum depth of 10mm with a sawn edge. Clean the surface and remove any dust, unsound material, plaster, oil, paint, and grease.
- (b) Prime exposed reinforcing steel bars and apply one full coat of Nitoprime Zincrich and allow to dry before continuing.
- (c) The concrete surface must be surface dry before wetting with clean water to fully saturate the surface. Wipe off excess/standing water, then apply one coat of Nitoprime HAR.
- (d) As soon as the primer is tacky, apply Fosroc Renderoc HB40. Take care to ensure that Fosroc Renderoc HB40 is thoroughly mixed using a forced-action mixer and applied in one layer up to a maximum thickness of 40mm for vertical application and 30mm for horizontal (overhead) application.

9.0 MASONRY

Workmanship is to comply generally with BS 5628 Parts 1 & 3. Brickwork is to comply with BS 3921. Blockwork is to comply with BS 6073.

New brickwork above DPC is to be a minimum of class 3 clay bricks (20 N/mm²) set in mortar unless noted otherwise on the drawings.

New medium dense or dense blockwork above DPC is to have a minimum strength of 3.5 N/mm² set in mortar unless noted otherwise on the drawings. Lightweight blockwork above DPC is to have a minimum strength of 3.5 N/mm² set in 1:2:8 mortar unless noted otherwise on the drawings.

Brickwork and blockwork are to be laid properly bonded as agreed with the Architect and fully bonded into existing work or as specified otherwise on the drawings. All perpendics must be fully filled with mortar.

New facing brickwork below DPC is to be minimum FL durability bricks set in 1:2:8 mortar with SRPC unless noted otherwise on the drawings.

9.6 All other masonry below DPC is to be class B engineering bricks, set in 1:2:8 mortar with SRPC or as noted otherwise on the drawings.

Do not lay masonry when the ambient air temperature is less than 3°C.

9.8 Cavity Wall Ties

For new masonry in domestic or small commercial buildings up to 15m in height and where the cavity is 50 to 100mm, use Ancon 'Staifix' RT2 Type 2 stainless steel ties to DD140-2, spaced at 450mm vertically, 900mm horizontally staggered, and at 225mm vertically, 150mm away from all openings, corners, movement joints, and reveals. Ties must have a minimum 50mm embedment into each masonry leaf. Additional ties may be needed to positively restrain cavity insulation, subject to manufacturer's requirements.

9.9 New Openings

Where new openings are formed in existing external cavity walls, provide Hilti stainless steel HRR Resin wall ties or similar approved, at 225mm vertical and 450mm horizontal centres. These are to be 150mm from the vertical and 225mm from the horizontal sides, ensuring that the cavity tray above the opening is not punctured.

9.10 Movement Joints in Facing Brickwork

Movement joints in facing brickwork are to be formed by building in 'Aerofil' joint filler by Serviced Ltd as the works proceed, ensuring no projections into cavities. In joints to be pointed with sealant, position the filler accurately at the recommended distance from the face of the wall. Leave joints open for as long as possible before sealing with appropriate sealant to Architect's specification.

9.11 Pinning Up to Soffits

Where pinning up to soffits is required, completely fill the joint at the top of loadbearing walls with 1:3 cement/sharp sand dry pack mortar, well rammed into position using temporary shuttering.

Carry up work with no portion or section of wall more than 1.2m above another at any time, raking back between levels. Do not carry up work higher than 1.5m in one day.

9.12 Spacing of Movement Joints

Spacing of movement joints in brickwork and blockwork is not to exceed 12.0m and 6.0m (6.0m and 3.0m from a corner) respectively. Locations of movement joints must be agreed with the Structural Engineer and Architect prior to commencing work.

9.13 Bed Joint Reinforcement

Provide stainless steel bed joint reinforcement, "Bricktor" by BRC, at 225mm and 450mm vertical centres above external doors and above and below windows. Bed joint reinforcement is to extend 500mm minimum past the door/window line.

Clause 9.14 - In Conjunction with Section 16 of the Specification

All existing walls will require the following remedial works:

1. Removal of Existing Steelwork and Timbers:

- Carefully remove all existing steelwork and timbers not required.
- Obtain approval from relevant local authorities and the Consulting Structural Engineer before proceeding.

2. Repair of Minor Cracks:

- Repair all minor cracks using a 1:28 mortar mix.

3. Rebuilding Deteriorated Mortar/Masonry:

- Remove and rebuild any mortar or masonry judged to be crumbly, soft, or deteriorated.
- Use bonded engineering brick, facing brick, or block as appropriate.
- Match existing brickwork as required by the Architect.

4. Infill of Unused Openings:

- Infill all unused internal masonry wall openings with bonded brick/block to match existing.
- Break out and replace all straight joints with fully bonded brickwork as required by the Structural Engineer.

5. Stitching Cracks Over 2mm:

- For cracks greater than 2mm, provide stitching using 900mmL x 100mmW x 75D precast concrete lintels at 900mm centres vertically.
- Insert lintels on a mortar bed and pack with a 1:1:6 semi-dry mortar, well rammed.
- For cracks adjacent to corners, install reinforced concrete elbows (each leg 675mm long, 100mm wide x 150mm deep) reinforced with 1 No. bent T12 bar, 40mm cover top, and 1 No. bent T12 40 cover bottom at 900mm centres vertically.

Structural Timber

New timber for the works must meet the following specifications:

1. Timber Quality:

- Use selected structural timber, not inferior to European Redwood/Whitewood grade C24 to BS 5268: Part 2, unless noted otherwise on the drawings.
- Vacuum impregnate with preservative to BS 5268: Part 5 and manufacturer's recommendations.
- Treat cut ends with brush-applied coats of appropriate preservative before fixing.
- Store timber in a dry, sheltered location on-site; do not use timber with a moisture content greater than 20%.

2. Cutting and Drilling for Services:

- Notch joists at the top, between 0.1 and 0.25 of the spans from the support.
- Notches must not be deeper than 0.125 of the joist depth.
- Holes in joists should be along the centre with a maximum diameter of 0.125 of the joist depth.

3. Fixtures and Fittings:

- Allow for all necessary clips, shoes, brackets, joist hangers, straps, bracing, fixings, screws, etc., to execute the work successfully.
- Use galvanized screws, nails, timber connectors, joist hangers, and steel straps.
- Install all proprietary fixings according to the manufacturer's details/instructions.

4. Inspection and Repair of Existing Timbers:

- Inspect all existing timbers for rot and infestation at the beginning of the works by a specialist subcontractor.
- Agree on-site details for replacing or strengthening any defective timbers recommended by the specialist.

5. Re-tiling or Re-cladding Roofs:

- Re-nail connections between rafters, ceiling joists, and wall plates to enhance the original framed construction joints.
- Provide noggins 3/4 depth of existing joists by 50mm wide along lines of support and at mid-spans for spans over 4500mm.

6. Trimmers and Doubling Up Joists:

- Use joist hangers for trimmers to openings in floors and ceiling construction unless noted otherwise.
- Double up joists under new partitions running parallel to the joist span and bolt together at 600mm centres using M12 bolts and oversize washers.
- Provide solid noggins under new partitions running perpendicular to the joist span.

7. Floor and Roof Construction:

- Construct new floors with 18mm flooring grade fixed to joists with No.6 x 50mm long countersunk head sherardized screws at 300mm centres unless otherwise noted on the drawings.
- Ensure joints between plywood sheets or where plywood sheets are butted together are supported by joists or noggins.
- Provide full depth noggins 50mm wide along lines of support and at mid-span for spans exceeding 2500mm and at 1/3 and 2/3 span positions for spans exceeding 4500mm.

8. Adhesives for Timber Joints:

- Comply with BS 1204 Part I for all adhesives used.
- Use 'Weather and Boil' proof Phenol-resorcinol formaldehyde (PRF) for bonding as per manufacturer's recommendations.
- Follow a specified sequence for adhesive application, ensuring clean, dry, and accurately machined surfaces.

Steelwork

All steelwork must comply with the following requirements:

1. Workmanship and Materials:

- Adhere to BS 5950: Part 2 and British Steel's Structural Steelwork specification.
- Use Grade S275 structural steel sections to BS EN 10025 unless noted otherwise on the drawings.
- Use grade 8.8 Precision Bolts to BS 3692 with a class 1 sherardized finish to BS 4921.

2. Welding and Fabrication:

- Ensure all welding complies with BS 5135; site welding requires the Structural Engineer's written approval.
- Conduct an accurate site survey to determine final dimensions for all new steelwork prior to fabrication.
- Design and detail all steelwork connections in agreement with the Structural Engineer and Architect.

3. Protective Coatings:

- Apply cold galvanizing compound or paint to site connections upon completion of installation.
- Power brush steelwork clean to bright steel and apply two coats of alkyd-based high-build zinc phosphate paint before erection.
- Additional WRC Bitumen paint coat required for beam ends built into masonry walls.

4. Fire Protection and Galvanizing:

- Comply with Architect's details and Building Regulation requirements for fire protection.
- Complete all cutting, welding, and drilling before galvanizing, ensuring a minimum coating thickness of 140 microns.

Concrete and Underpinning

Mass Concrete Underpinning

1. Workmanship and Responsibilities:

- Adhere to the Engineer's drawings and instructions, approved by the Architect and Building Control Officer.
- Ensure safety and stability of the excavation and structure with necessary shoring, strutting, and bracing.

2. Construction Sequence:

- Undertake underpinning in sections not exceeding 1 metre in length on a 'hit and miss' sequence.
- Allow a minimum of 48 hours between casting adjacent pins.

3. Excavation and Inspections:

- Excavations should be to the depth and width shown on the drawings; extend new underpins 600mm below any root activity.
- Inspections by the Engineer and/or Building Control Officer required before proceeding.

4. Concrete Placement:

- Use mass concrete, compacted with a mechanical vibrator, leaving 75mm below the existing footing.

- Fill the top 75mm with a well-rammed cement and sharp sand mix after 24 hours.

5. Concrete Grade:

- Use FND2 mix for non-specified works, suitable for Class 2 sulphate soils with a 28-day strength of 35N/mm².

6. Spoil and Records:

- Remove all spoil not suitable for backfilling.
- Keep detailed records of each underpinning section, available for inspection.

Structural Render

Purpose and Application

1. Binding and Stabilization:

- Apply structural render to bind and stabilize poorly bonded, fractured, or loose masonry.
- Comply with BS 5262: 1991 for all works.

2. Preparation:

- Hack off old plaster/render and damaged brickwork to expose sound masonry.
- Clean and scabble surfaces before applying render.

3. Application:

- Fix 50 x 50mm x 10-gauge steel mesh to masonry using washers and plugged wood screws.
- Apply internal and external render coats as specified, ensuring proper moisture control and reinforcement coverage.

By adhering to these detailed specifications, the contractor ensures that the remedial works, structural timber installations, steelwork, concrete underpinning, and structural render applications meet the required standards and are executed with the necessary precision and care.

Top Of Ground Floor

Floor Joists

Max. Length $\leq$ 4800mm

Floor Loads:

Permanent: 1.2 kN/m²

Live: 1.5 kN/m²

47/275

(See Calculations Below)

New Beam Loads

Max. Span 7200mm

Max. Loads:

Permanent: 7 kN/m

Live: 7 kN/m

ADOPT Section 2 no UB 305x165x40 S355

Max. Span 6400mm

Max. Loads:

Permanent: 5.8 kN/m

Live: 6.5 kN/m

ADOPT Section UB 305x165x40 S355

(See Calculations Below)



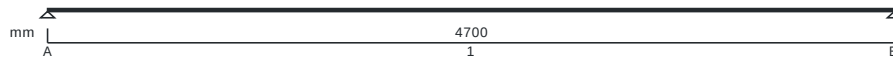
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TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

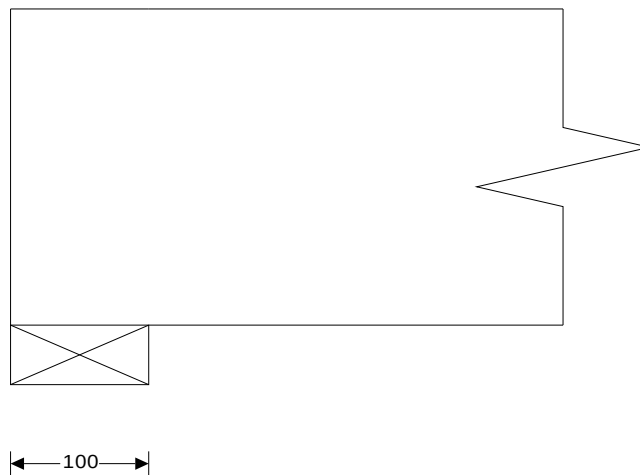
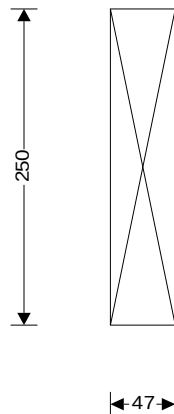
Joist details

Joist breadth	b = 47 mm
Joist depth	h = 250 mm
Joist spacing	s = 400 mm
Timber strength class	C24
Service class of timber	1



Span details

Number of spans	N _{span} = 1
Length of bearing	L _b = 100 mm
Effective length of span	L _{s1} = 4700 mm



Section properties

Second moment of area	I = b × h ³ / 12 = 61197917 mm ⁴
Section modulus	Z = b × h ² / 6 = 489583 mm ³

Loading details

Joist self weight	F _{swt} = b × h × ρ _{char} × g _{acc} = 0.04 kN/m
Dead load	F _{d_udi} = 1.20 kN/m ²
Imposed UDL(Long term)	F _{i_udi} = 1.50 kN/m ²
Imposed point load (Medium term)	F _{i_pt} = 1.40 kN

Modification factors

Service class for bending parallel to grain	K _{2m} = 1.00
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Service class for compression $K_{2c} = 1.00$
Service class for shear parallel to grain $K_{2s} = 1.00$
Service class for modulus of elasticity $K_{2e} = 1.00$
Section depth factor $K_7 = 1.02$
Load sharing factor $K_8 = 1.10$

Consider long term loads

Load duration factor $K_3 = 1.00$
Maximum bending moment $M = 3.094$ kNm
Maximum shear force $V = 2.633$ kN
Maximum support reaction $R = 2.633$ kN
Maximum deflection $\delta = 11.238$ mm

Check bending stress

Bending stress $\sigma_m = 7.500$ N/mm²
Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 8.417$ N/mm²
Applied bending stress $\sigma_{m_max} = M / Z = 6.319$ N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress $\tau = 0.710$ N/mm²
Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.781$ N/mm²
Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) = 0.336$ N/mm²

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) $\sigma_{cp1} = 2.400$ N/mm²
Permissible bearing stress $\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.640$ N/mm²
Applied bearing stress $\sigma_{c_max} = R / (b \times L_b) = 0.560$ N/mm²

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000$ mm
Bending deflection (based on E_{mean}) $\delta_{bending} = 10.770$ mm
Shear deflection $\delta_{shear} = 0.468$ mm
Total deflection $\delta = \delta_{bending} + \delta_{shear} = 11.238$ mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor $K_3 = 1.25$
Maximum bending moment $M = 3.082$ kNm
Maximum shear force $V = 2.623$ kN
Maximum support reaction $R = 2.623$ kN
Maximum deflection $\delta = 10.050$ mm

Check bending stress

Bending stress $\sigma_m = 7.500$ N/mm²
Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 10.521$ N/mm²
Applied bending stress $\sigma_{m_max} = M / Z = 6.295$ N/mm²

PASS - Applied bending stress within permissible limits



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Check shear stress

Shear stress

$$\tau = 0.710 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976 \text{ N/mm}^2$$

Applied shear stress

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.335 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = 2.400 \text{ N/mm}^2$$

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.300 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = 0.558 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = 9.584 \text{ mm}$$

Shear deflection

$$\delta_{shear} = 0.466 \text{ mm}$$

Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = 10.050 \text{ mm}$$

PASS - Actual deflection within permissible limits

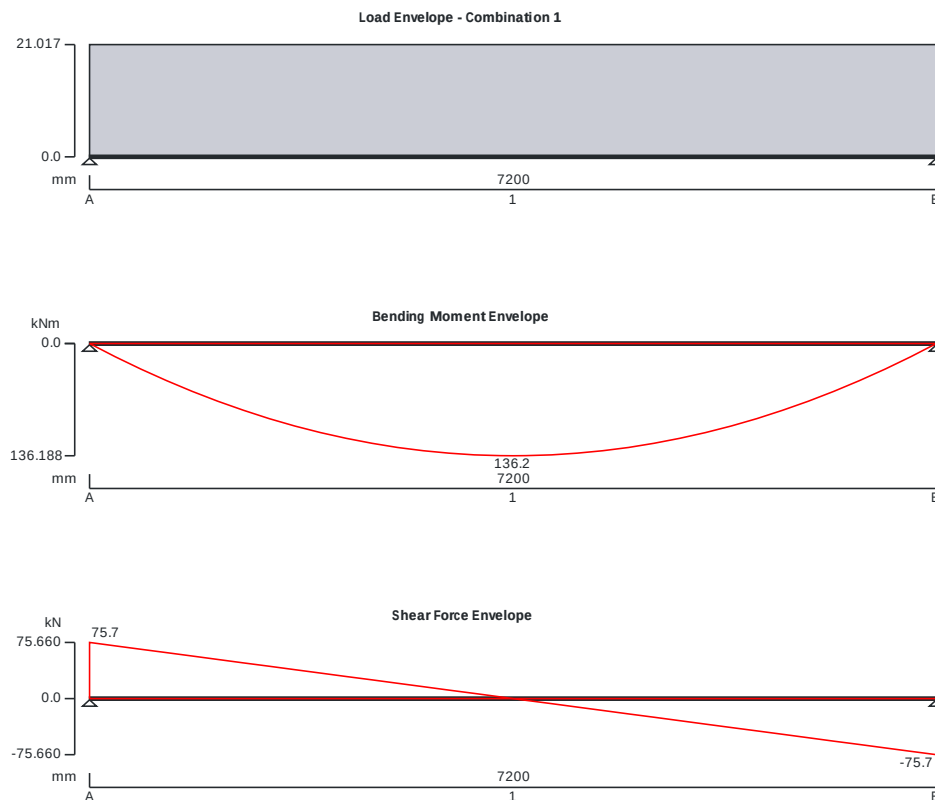


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 7 kN/m

Variable full UDL 7 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



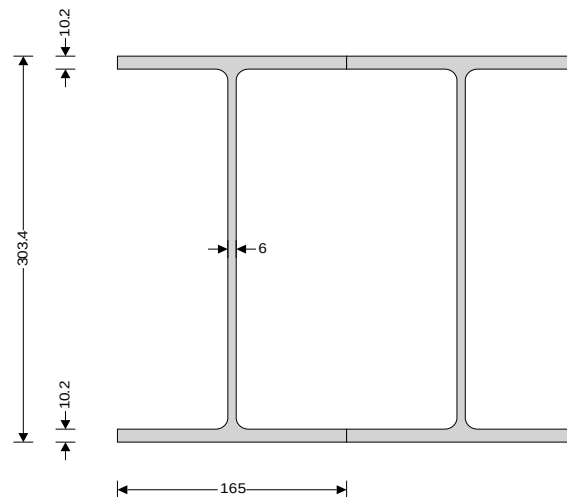
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Analysis results

Maximum moment	$M_{\max} = 136.2 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 75.7 \text{ kN}$	$V_{\min} = -75.7 \text{ kN}$
Deflection	$\delta_{\max} = 14.5 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 75.7 \text{ kN}$	$R_{A_{\min}} = 75.7 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 28 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 25.2 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 75.7 \text{ kN}$	$R_{B_{\min}} = 75.7 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 28 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 25.2 \text{ kN}$	

Section details

Section type	2 x UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_r, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = \mathbf{265.2 \text{ mm}}$$

$$c / t_w = 47.8 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{70.6 \text{ mm}}$$

$$c / t_f = 7.5 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{283 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{75.7 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2009 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{637.9 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{136.2 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = N \times W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{342.7 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.954}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{7200 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{89.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{1.387}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.389}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.479}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.991}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.484}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{165.8 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment



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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 20 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 14.492 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

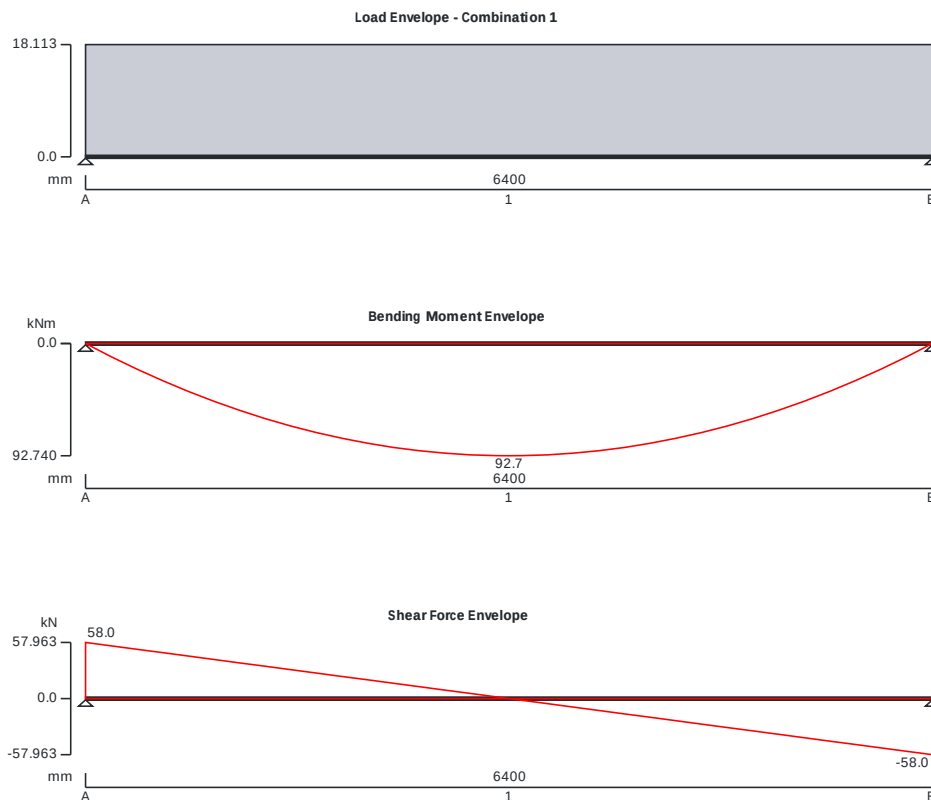


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In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 5.8 kN/m

Variable full UDL 6.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

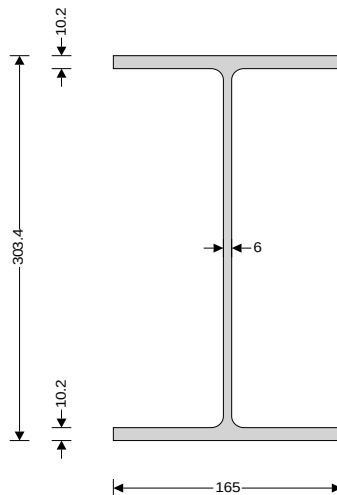
Maximum moment	$M_{\max} = 92.7 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 58 \text{ kN}$	$V_{\min} = -58 \text{ kN}$
Deflection	$\delta_{\max} = 15.5 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 58 \text{ kN}$	$R_{A_{\min}} = 58 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 19.8 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 20.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 58 \text{ kN}$	$R_{B_{\min}} = 58 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 19.8 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 20.8 \text{ kN}$	

Section details

Section type	UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_r, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = \mathbf{265.2 \text{ mm}}$$

$$c / t_w = 47.8 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{70.6 \text{ mm}}$$

$$c / t_f = 7.5 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{283 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{58 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2009 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{318.9 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{92.7 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{171.3 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.954}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{6400 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{104.9 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{1.278}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.262}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.535}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.984}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.544}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{93.3 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment



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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 17.8 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 15.532 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

Top Of First Floor

Load Bearing Structural Elements Check



Floor Joists

Max. Length $\leq$ 4700mm

Floor Loads:

Permenant: 1.2 kN/m²

Live: 1.5 kN/m²

Existing Joist C16+ 47/275 SUFFICIENT

(See Calculations Below)

Beams Check



GENERAL ARRANGEMENT
TOP OF FIRST FLOOR

Existing Steel Beam 1 Check

Span $\leq$ 5100mm

Loads:

Permanent: 1.2 kN/m²

Live: 1.5 kN/m²

Effective Length: 5.1 m

Min. Required Section UB 254x146x31 S275

Existing Beam **SUFFICIENT**

(See Calculations Below)

Steel Beam Lintel L1

Length $\leq$ 2000 mm

Max. Loads:

Permanent: 60.17 kN/m

Live: 13.5 kN/m

ADOPT Section UC 203x203x46 S355

(See Calculations Below)

New Beam 1-2-3 Loads

Max. Span 5100mm

Max. Loads:

Permenant: 3.4 kN/m

Live: 4.2 kN/m

ADOPT Section UB 203x133x25 S355

(See Calculations Below)

New Beam 4-5 Loads

Max. Span 6300mm

Max. Loads:

Permenant: 5.2 kN/m

Live: 6.5 kN/m

ADOPT Section UB 305x165x40 S355

(See Calculations Below)

New Beam 6 Loads

Max. Span 6800mm

Max. Loads:

Permenant: 7 kN/m

Live: 7 kN/m

ADOPT Section UB 305x165x40 S355

(See Calculations Below)

New Beam 7-8 Loads

Max. Span 2000mm

Max. Loads:

Permenant: 3.4 kN/m

Live: 4.2 kN/m

ADOPT Section UB 178x102x19 S355

(See Calculations Below)



Concrete Lintel Check L2

LINTEL LOAD / SPAN TABLE

Service RM (kN/m)		100 x 65 Flat	100 x 65 Upright	140 x 65 Flat	140 x 65 Upright	140 x 100 Flat	100 x 140 Upright
		0.93	1.35	1.03	2.77	3.17	4.52
Length	Effective Length	UDL (kN/m)	UDL (kN/m)	UDL (kN/m)	UDL (kN/m)	UDL (kN/m)	UDL (kN/m)
450	n/a	82.35	120.43	91.72	245.91	281.55	401.73
600	450	36.60	53.52	40.77	109.29	125.13	178.55
750	600	20.59	30.11	22.93	61.48	70.39	100.43
900	750	13.18	19.27	14.68	39.35	45.05	64.28
1050	900	9.15	13.38	10.19	27.32	31.28	44.64
1200	1050	6.72	9.83	7.49	20.07	22.98	32.79
1350	1200	5.15	7.53	5.73	15.37	17.6	25.11
1500	1350	4.07	5.95	4.53	12.14	13.90	19.84
1650	1500	3.29	4.82	3.67	9.84	11.26	16.07
1800	1650	2.72	3.98	3.03	8.13	9.31	13.28
2100	1950	1.95	2.85	2.17	5.82	6.66	9.51
2400	2250	1.46	2.14	1.63	4.37	5.01	7.14
2700	2550	1.14	1.67	1.27	3.40	3.90	5.56
3000	2850	0.91	1.33	1.02	2.72	3.12	4.45
3300	3150	0.75	1.09			2.55	3.64
3600	3450	0.62	0.91			2.13	3.04

Length <= 2200mm

Loads:

Permenant: 1.2 kN/m²

Live: 1.5 kN/m²

Effective Length: 4m

UDL=(1.2+1.5)x4=10.8 kN/m

Existing Beam **INSUFFICIENT**



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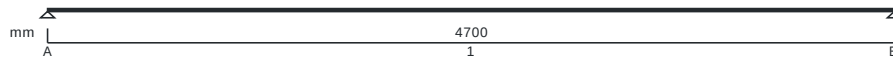
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Calcs for TOP OF FIRSR FLOOR FLOOR JOISTS				Start page no./Revision 1	
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TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

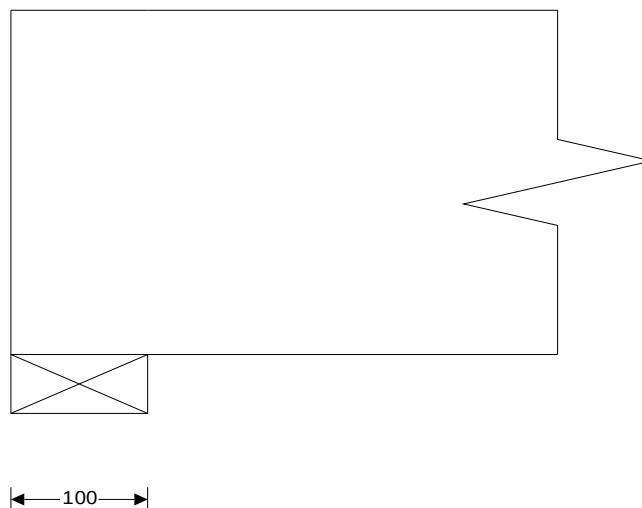
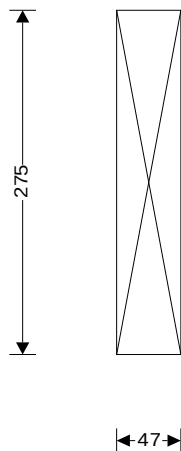
Joist details

Joist breadth $b = 47 \text{ mm}$
 Joist depth $h = 275 \text{ mm}$
 Joist spacing $s = 400 \text{ mm}$
 Timber strength class **C16**
 Service class of timber **1**



Span details

Number of spans $N_{\text{span}} = 1$
 Length of bearing $L_b = 100 \text{ mm}$
 Effective length of span $L_{s1} = 4700 \text{ mm}$



Section properties

Second moment of area $I = b \times h^3 / 12 = 81454427 \text{ mm}^4$
 Section modulus $Z = b \times h^2 / 6 = 592396 \text{ mm}^3$

Loading details

Joist self weight $F_{\text{swt}} = b \times h \times \rho_{\text{char}} \times g_{\text{acc}} = 0.04 \text{ kN/m}$
 Dead load $F_{\text{d_udl}} = 1.20 \text{ kN/m}^2$
 Imposed UDL(Long term) $F_{\text{i_udl}} = 1.50 \text{ kN/m}^2$
 Imposed point load (Medium term) $F_{\text{i_pt}} = 1.40 \text{ kN}$

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Modification factors

Service class for bending parallel to grain	$K_{2m} = 1.00$
Service class for compression	$K_{2c} = 1.00$
Service class for shear parallel to grain	$K_{2s} = 1.00$
Service class for modulus of elasticity	$K_{2e} = 1.00$
Section depth factor	$K_7 = 1.01$
Load sharing factor	$K_8 = 1.10$

Consider long term loads

Load duration factor	$K_3 = 1.00$
Maximum bending moment	$M = 3.091 \text{ kNm}$
Maximum shear force	$V = 2.630 \text{ kN}$
Maximum support reaction	$R = 2.630 \text{ kN}$
Maximum deflection	$\delta = 10.443 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 5.886 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 5.217 \text{ N/mm}^2$
PASS - Applied bending stress within permissible limits	

Check shear stress

Shear stress	$\tau = 0.670 \text{ N/mm}^2$
Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Applied shear stress	$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.305 \text{ N/mm}^2$
PASS - Applied shear stress within permissible limits	

Check bearing stress

Compression perpendicular to grain (no wane)	$\sigma_{cp1} = 2.200 \text{ N/mm}^2$
Permissible bearing stress	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.420 \text{ N/mm}^2$
Applied bearing stress	$\sigma_{c_max} = R / (b \times L_b) = 0.560 \text{ N/mm}^2$
PASS - Applied bearing stress within permissible limits	

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$
Bending deflection (based on E_{mean})	$\delta_{bending} = 9.921 \text{ mm}$
Shear deflection	$\delta_{shear} = 0.522 \text{ mm}$
Total deflection	$\delta = \delta_{bending} + \delta_{shear} = 10.443 \text{ mm}$
PASS - Actual deflection within permissible limits	

Consider medium term loads

Load duration factor	$K_3 = 1.25$
Maximum bending moment	$M = 3.079 \text{ kNm}$
Maximum shear force	$V = 2.620 \text{ kN}$
Maximum support reaction	$R = 2.620 \text{ kN}$
Maximum deflection	$\delta = 9.347 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 7.358 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 5.197 \text{ N/mm}^2$



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PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress

$$\tau = 0.670 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.921 \text{ N/mm}^2$$

Applied shear stress

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.304 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = 2.200 \text{ N/mm}^2$$

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.025 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = 0.558 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = 8.828 \text{ mm}$$

Shear deflection

$$\delta_{shear} = 0.520 \text{ mm}$$

Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = 9.347 \text{ mm}$$

PASS - Actual deflection within permissible limits

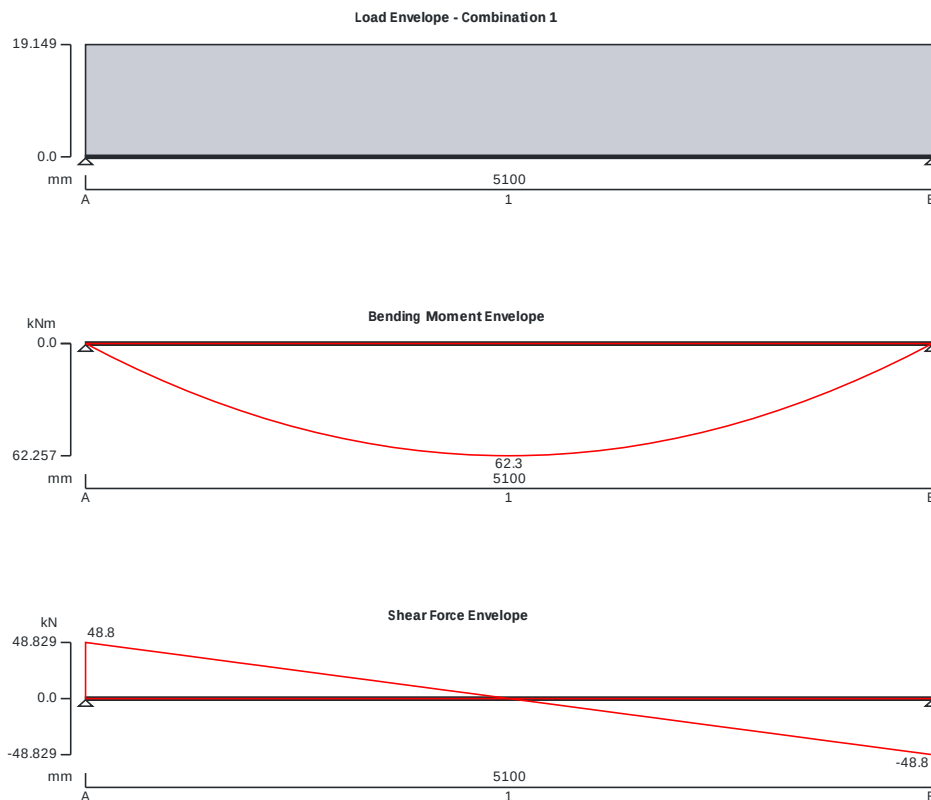


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 6.12 kN/m

Permanent full UDL 7.65 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

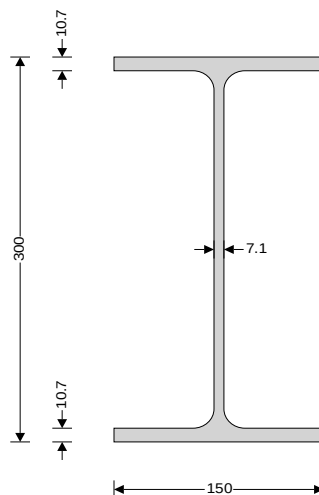
Maximum moment	$M_{\max} = 62.3 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 48.8 \text{ kN}$	$V_{\min} = -48.8 \text{ kN}$
Deflection	$\delta_{\max} = 7.1 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 48.8 \text{ kN}$	$R_{A_{\min}} = 48.8 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 36.2 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 48.8 \text{ kN}$	$R_{B_{\min}} = 48.8 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 36.2 \text{ kN}$	

Section details

Section type	IPE 300 (Arcelor)
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 10.7 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT.A} = 1.000$
	$K_{LT.B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 248.6 \text{ mm}$
 $c / t_w = 37.9 \times \varepsilon \leq 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 56.5 \text{ mm}$
 $c / t_f = 5.7 \times \varepsilon \leq 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 278.6 \text{ mm}$
Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 48.8 \text{ kN}$
Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2568 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 407.8 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 62.3 \text{ kNm}$
Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 172.8 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$
 $C_1 = 1 / k_c^2 = 1.132$
Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = 0.963$
Poissons ratio $\nu = 0.3$
Shear modulus $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length $L = 1.0 \times L_{s1} = 5100 \text{ mm}$
Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 131.8 \text{ kNm}$
Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.145$
Limiting slenderness ratio $\bar{\lambda}_{LT,0} = 0.4$
 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b
Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$
Correction factor for rolled sections $\beta = 0.75$
LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.118$
LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.612$
Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.977$
Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.626$
Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 108.1 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads
Limiting deflection $\delta_{lim} = L_{s1} / 360 = 14.2 \text{ mm}$

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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 7.12 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

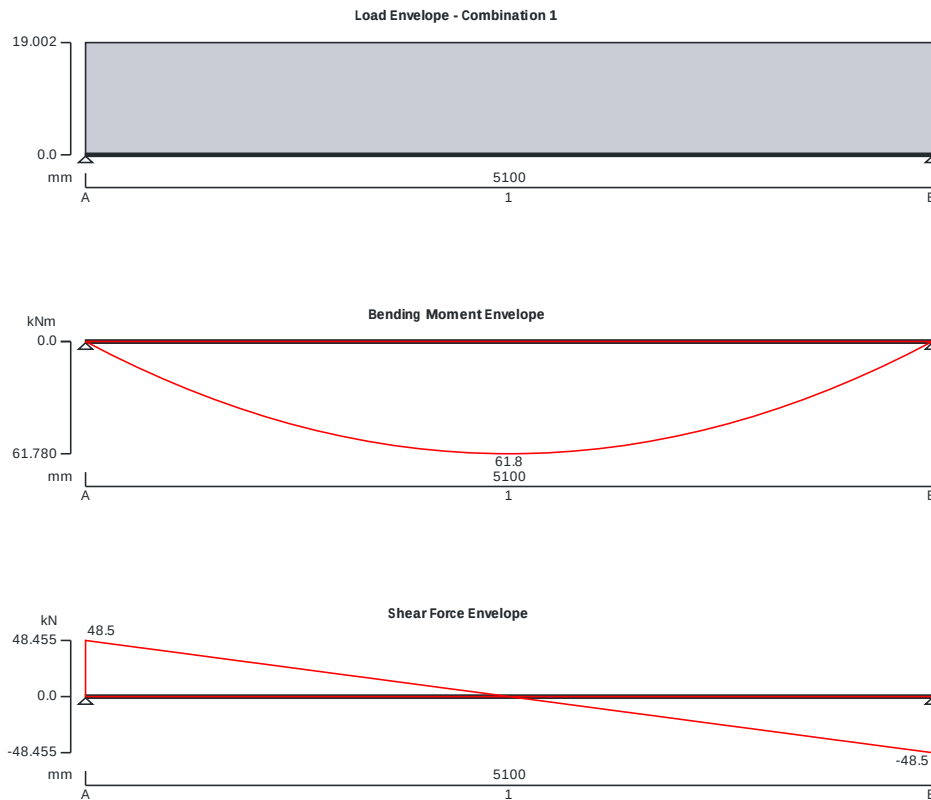


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 6.12 kN/m

Permanent full UDL 7.65 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

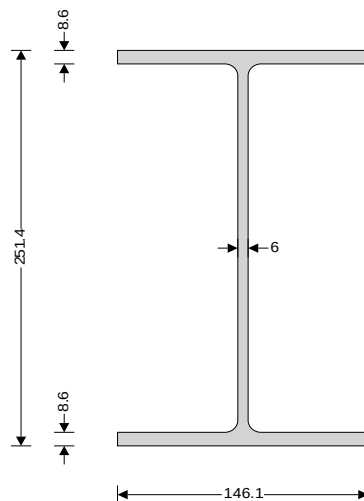
Maximum moment	$M_{max} = 61.8 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 48.5 \text{ kN}$	$V_{min} = -48.5 \text{ kN}$
Deflection	$\delta_{max} = 13.4 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 48.5 \text{ kN}$	$R_{A_{min}} = 48.5 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 35.9 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 48.5 \text{ kN}$	$R_{B_{min}} = 48.5 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 35.9 \text{ kN}$	

Section details

Section type	UB 254x146x31 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 8.6 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT.A} = 1.000$
	$K_{LT.B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 219 \text{ mm}$
 $c / t_w = 39.5 \times \epsilon \leq 72 \times \epsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 62.5 \text{ mm}$
 $c / t_f = 7.9 \times \epsilon \leq 9 \times \epsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 234.2 \text{ mm}$
Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \epsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 48.5 \text{ kN}$
Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1637 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 259.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 61.8 \text{ kNm}$
Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 108.1 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$
 $C_1 = 1 / k_c^2 = 1.132$
Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = 0.948$
Poissons ratio $\nu = 0.3$
Shear modulus $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length $L = 1.0 \times L_{s1} = 5100 \text{ mm}$
Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 78.6 \text{ kNm}$
Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.172$
Limiting slenderness ratio $\bar{\lambda}_{LT,0} = 0.4$
 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b
Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$
Correction factor for rolled sections $\beta = 0.75$
LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.147$
LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.595$
Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.978$
Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.608$
Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 65.8 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads
Limiting deflection $\delta_{lim} = L_{s1} / 360 = 14.2 \text{ mm}$



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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{13.378 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit



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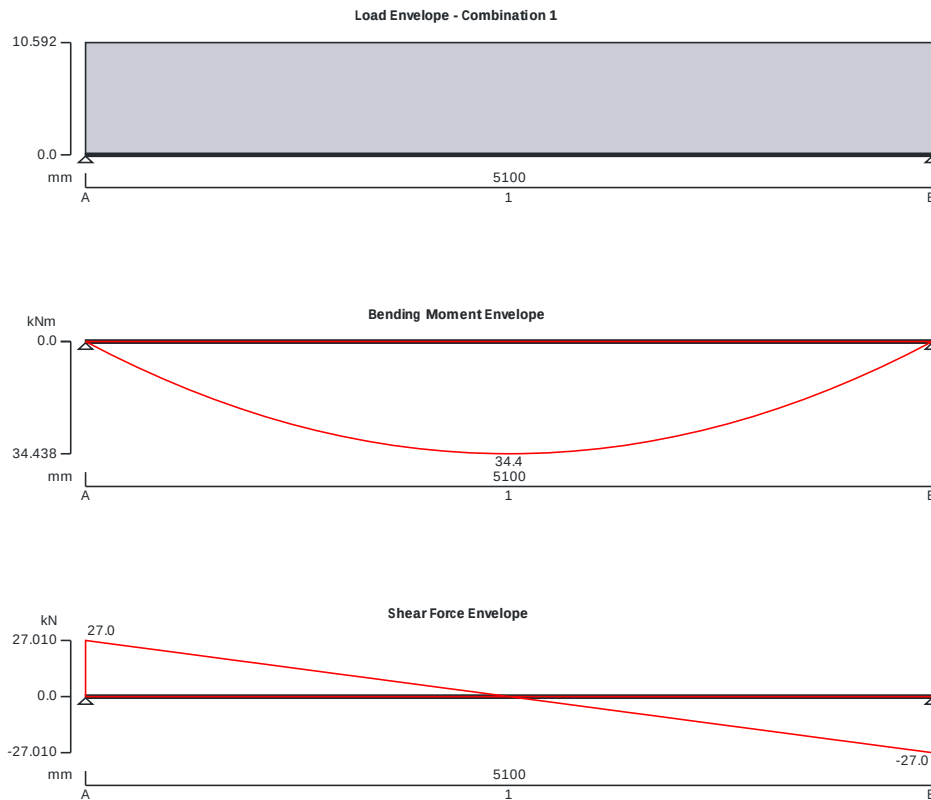
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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 3.4 kN/m

Permanent full UDL 4.2 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

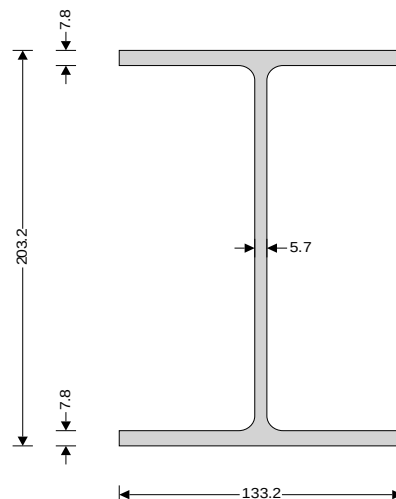
Maximum moment	$M_{\max} = 34.4 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 27 \text{ kN}$	$V_{\min} = -27 \text{ kN}$
Deflection	$\delta_{\max} = 14.1 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 27 \text{ kN}$	$R_{A_{\min}} = 27 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 20 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 27 \text{ kN}$	$R_{B_{\min}} = 27 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 20 \text{ kN}$	

Section details

Section type	UB 203x133x25 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 7.8 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT.A} = 1.000$
	$K_{LT.B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 172.4 \text{ mm}$
 $c / t_w = 32.7 \times \epsilon \leq 72 \times \epsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$
 $c / t_f = 7.8 \times \epsilon \leq 9 \times \epsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 187.6 \text{ mm}$
Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \epsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 27 \text{ kN}$
Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 34.4 \text{ kNm}$
Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$
 $C_1 = 1 / k_c^2 = 1.132$
Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = 0.932$
Poissons ratio $\nu = 0.3$
Shear modulus $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length $L = 1.0 \times L_{s1} = 5100 \text{ mm}$
Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 50.9 \text{ kNm}$
Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.18$
Limiting slenderness ratio $\bar{\lambda}_{LT,0} = 0.4$
 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b
Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$
Correction factor for rolled sections $\beta = 0.75$
LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.155$
LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.591$
Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.979$
Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.603$
Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 42.8 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads
Limiting deflection $\delta_{lim} = L_{s1} / 360 = 14.2 \text{ mm}$



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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{14.064 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

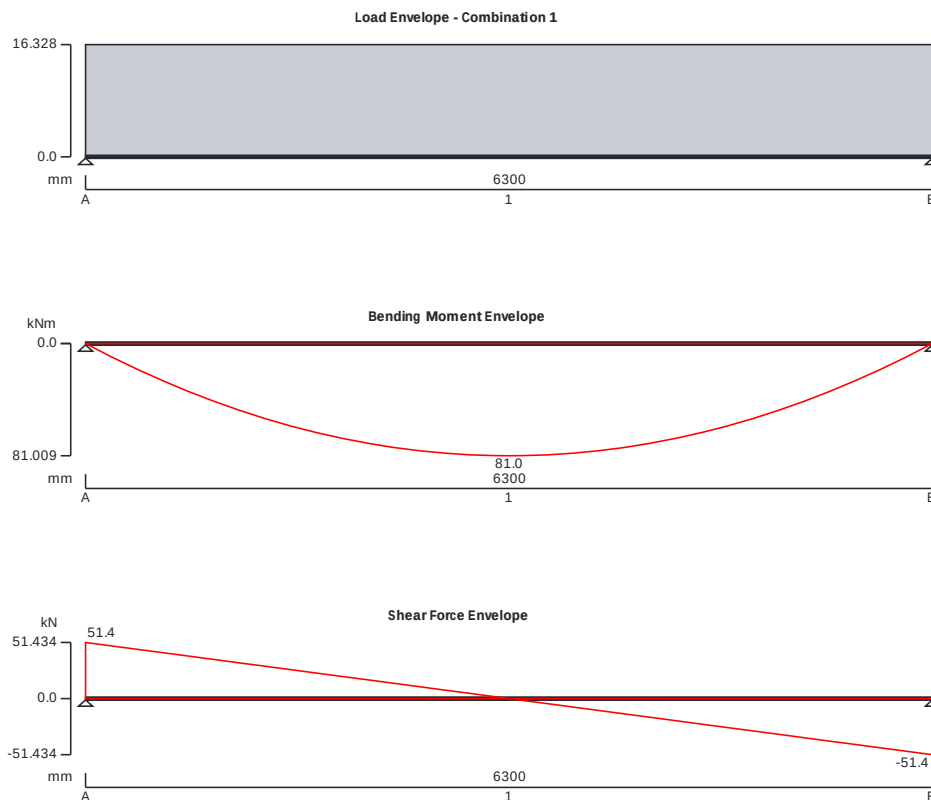


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 5.2 kN/m

Permanent full UDL 6.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

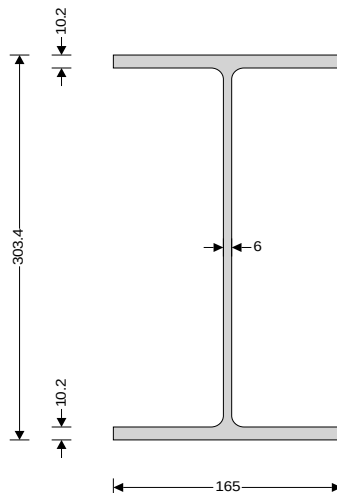
Maximum moment	$M_{max} = 81 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 51.4 \text{ kN}$	$V_{min} = -51.4 \text{ kN}$
Deflection	$\delta_{max} = 13.9 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 51.4 \text{ kN}$	$R_{A_{min}} = 51.4 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 38.1 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 51.4 \text{ kN}$	$R_{B_{min}} = 51.4 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 38.1 \text{ kN}$	

Section details

Section type	UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT.A} = 1.000$
	$K_{LT.B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 265.2 \text{ mm}$
 $c / t_w = 47.8 \times \epsilon \leq 72 \times \epsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 70.6 \text{ mm}$
 $c / t_f = 7.5 \times \epsilon \leq 9 \times \epsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 283 \text{ mm}$

Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \epsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 51.4 \text{ kN}$

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2009 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 318.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 81 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 171.3 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$

$C_1 = 1 / k_c^2 = 1.132$

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = 0.954$

Poissons ratio $\nu = 0.3$

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

Unrestrained length $L = 1.0 \times L_{s1} = 6300 \text{ mm}$

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 107.3 \text{ kNm}$

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.264$

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$

Correction factor for rolled sections $\beta = 0.75$

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.246$

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.543$

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.983$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.553$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 94.7 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = 17.5 \text{ mm}$



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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{13.894 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

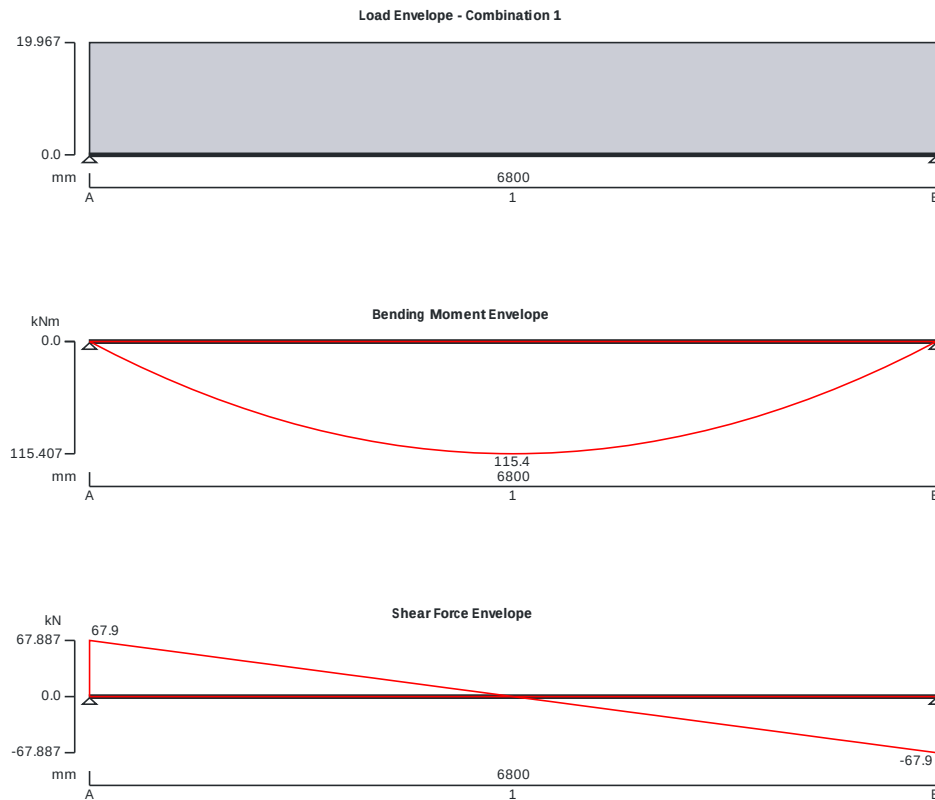


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam $\times$ 1
	Permanent full UDL 7 kN/m
	Permanent full UDL 7 kN/m

Load combinations

Load combination 1	Support A	Permanent $\times$ 1.35
		Variable $\times$ 1.50
		Permanent $\times$ 1.35
		Variable $\times$ 1.50
	Support B	Permanent $\times$ 1.35
		Variable $\times$ 1.50



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Analysis results

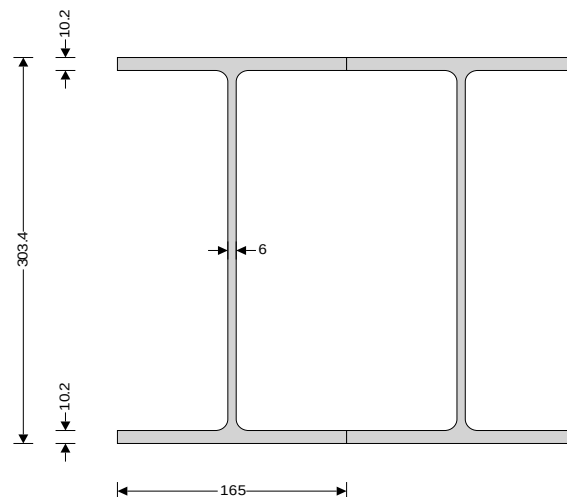
Maximum moment	$M_{max} = 115.4 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 67.9 \text{ kN}$	$V_{min} = -67.9 \text{ kN}$
Deflection	$\delta_{max} = 11.5 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 67.9 \text{ kN}$	$R_{A_min} = 67.9 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 50.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 67.9 \text{ kN}$	$R_{B_min} = 67.9 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 50.3 \text{ kN}$	

Section details

Section type	2 x UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 265.2 \text{ mm}$
 $c / t_w = 47.8 \times \epsilon \leq 72 \times \epsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 70.6 \text{ mm}$
 $c / t_f = 7.5 \times \epsilon \leq 9 \times \epsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 283 \text{ mm}$

Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \epsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 67.9 \text{ kN}$

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2009 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 637.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 115.4 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl,y} \times f_y / \gamma_{M0} = 342.7 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$

$C_1 = 1 / k_c^2 = 1.132$

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = 0.954$

Poissons ratio $\nu = 0.3$

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

Unrestrained length $L = 1.0 \times L_{s1} = 6800 \text{ mm}$

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 96.4 \text{ kNm}$

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.333$

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$

Correction factor for rolled sections $\beta = 0.75$

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.325$

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.506$

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.987$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.513$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl,y} \times f_y / \gamma_{M1} = 175.7 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = 18.9 \text{ mm}$



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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{11.53 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

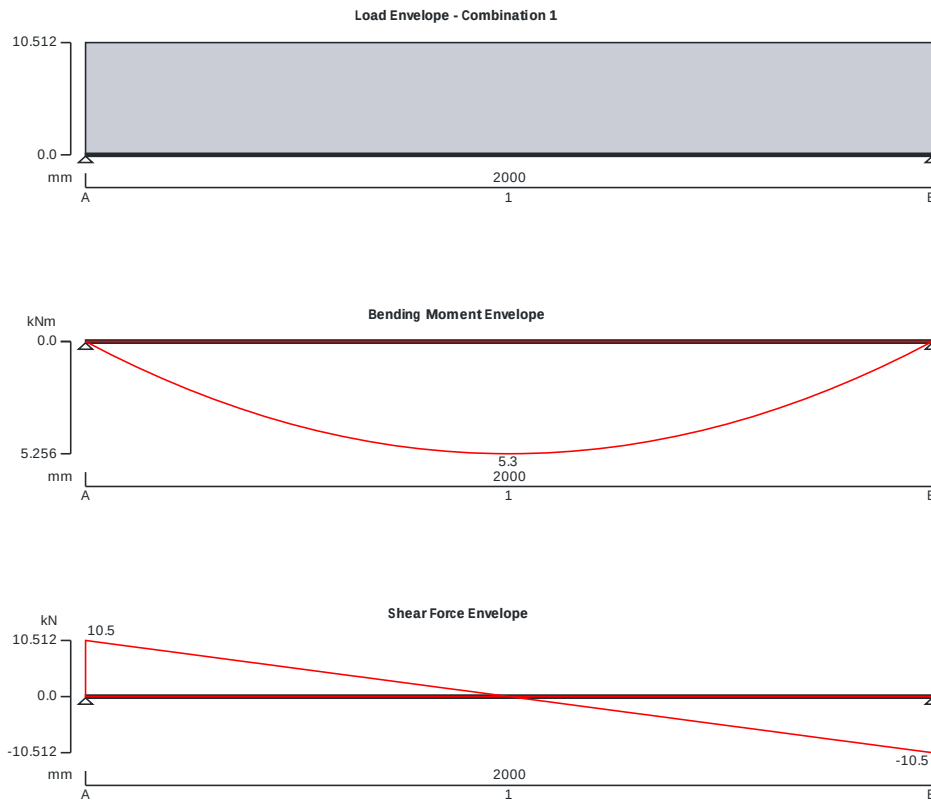


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 3.4 kN/m

Permanent full UDL 4.2 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

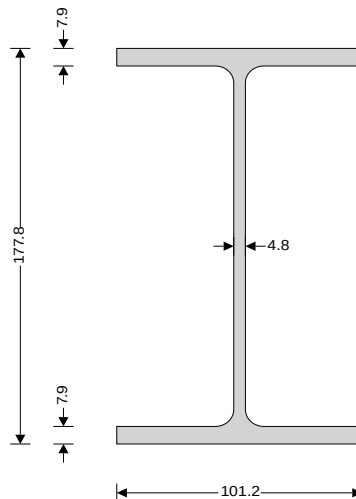
Maximum moment	$M_{\max} = 5.3 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 10.5 \text{ kN}$	$V_{\min} = -10.5 \text{ kN}$
Deflection	$\delta_{\max} = 0.6 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 10.5 \text{ kN}$	$R_{A_{\min}} = 10.5 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 7.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 10.5 \text{ kN}$	$R_{B_{\min}} = 10.5 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 7.8 \text{ kN}$	

Section details

Section type	UB 178x102x19 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 7.9 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT.A} = 1.000$
	$K_{LT.B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 146.8 \text{ mm}$
 $c / t_w = 33.1 \times \varepsilon \leq 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 40.6 \text{ mm}$
 $c / t_f = 5.6 \times \varepsilon \leq 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 162 \text{ mm}$

Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 10.5 \text{ kN}$

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 985 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 156.4 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 5.3 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 47.1 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$

$C_1 = 1 / k_c^2 = 1.132$

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = 0.948$

Poissons ratio $\nu = 0.3$

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

Unrestrained length $L = 1.0 \times L_{s1} = 2000 \text{ mm}$

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 93.5 \text{ kNm}$

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.71$

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$

Correction factor for rolled sections $\beta = 0.75$

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.741$

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.865$

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.970$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.891$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 42 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = 5.6 \text{ mm}$



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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{0.57 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

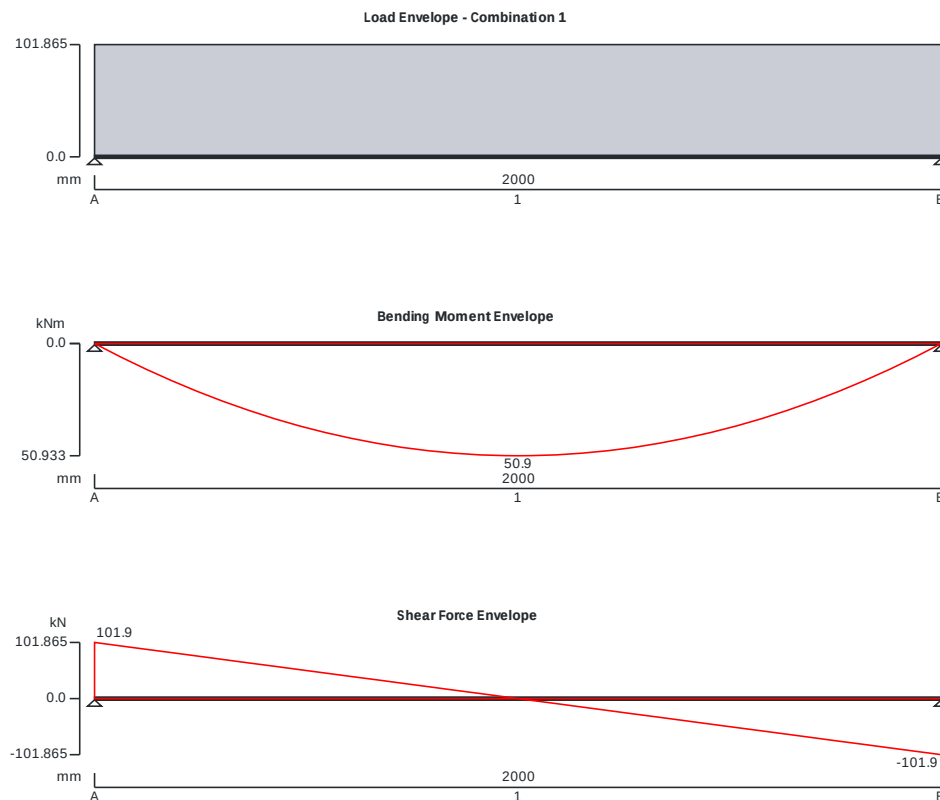


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 61 kN/m

Permanent full UDL 14 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

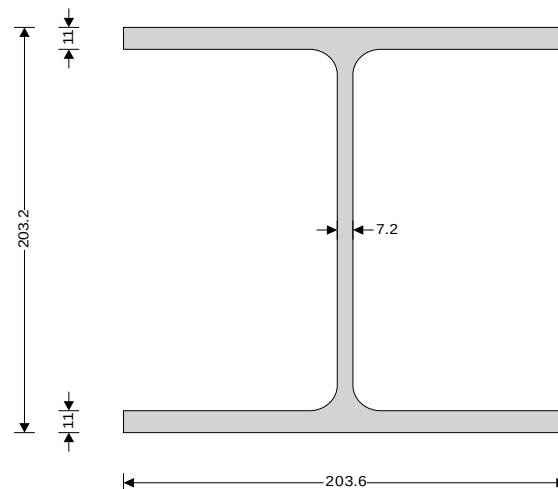
Maximum moment	$M_{max} = 50.9 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 101.9 \text{ kN}$	$V_{min} = -101.9 \text{ kN}$
Deflection	$\delta_{max} = 1.6 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 101.9 \text{ kN}$	$R_{A_min} = 101.9 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 75.5 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 101.9 \text{ kN}$	$R_{B_min} = 101.9 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 75.5 \text{ kN}$	

Section details

Section type	UC 203x203x46 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 11.0 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 155.8 \text{ mm}$
 $c / t_w = 23.4 \times \varepsilon \leq 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 85.5 \text{ mm}$
 $c / t_f = 8.4 \times \varepsilon \leq 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 181.2 \text{ mm}$
Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 101.9 \text{ kN}$
Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1802 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 286.1 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 50.9 \text{ kNm}$
Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 138 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$
 $C_1 = 1 / k_c^2 = 1.132$
Curvature factor $g = \sqrt{1 - (I_z / I_y)} = 0.815$
Poissons ratio $\nu = 0.3$
Shear modulus $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length $L = 1.0 \times L_{s1} = 2000 \text{ mm}$
Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)} = 1199.7 \text{ kNm}$
Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{W_{pl,y} \times f_y / M_{cr}} = 0.339$
Limiting slenderness ratio $\bar{\lambda}_{LT,0} = 0.4$

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads
Limiting deflection $\delta_{lim} = L_{s1} / 360 = 5.6 \text{ mm}$
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 1.626 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

**STEEL BEAM SPLICE DESIGN (EN1993)**

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009

Tedds calculation version 1.0.02

Design summary

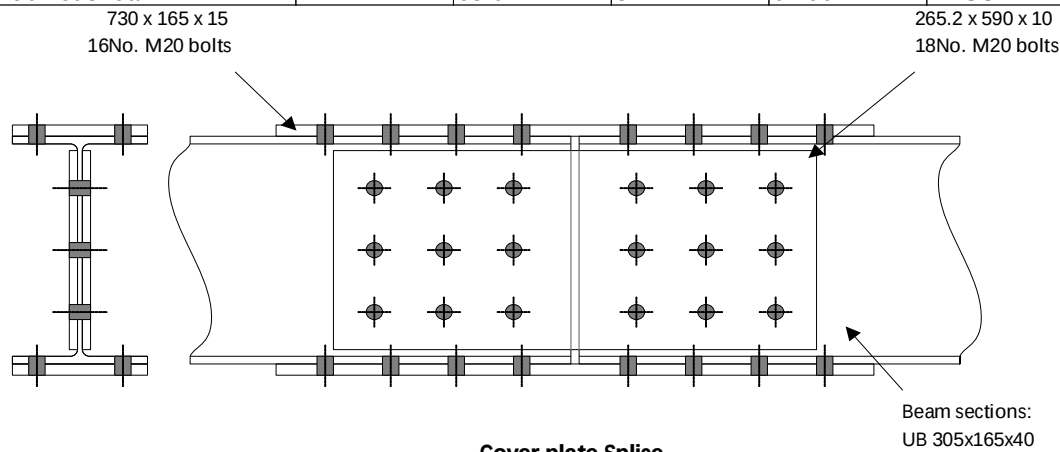
Overall design status

PASS

Overall design utilisation

0.820

Description	Unit	Design	Resistance	Utilisation	Result
Flange bolt group	kN	47.5	94.1	0.504	PASS
Flange bolt slip resist. SLS	kN	41.6	62.4	0.666	PASS
Flange plate compression	kN	379.7	678.0	0.560	PASS
Flange plate tension flange	kN	306.1	608.9	0.503	PASS
Flange plate block tearing	kN	306.1	1274.6	0.240	PASS
Flange compression	kN	379.7	462.8	0.820	PASS
Flange tension	kN	306.1	414.0	0.739	PASS
Web bolt group	kN	45.3	98.4	0.461	PASS
Web bolt slip resist. SLS	kN	43.0	124.7	0.344	PASS
Web plate shear	kN	68.0	663.1	0.103	PASS
Web plate block tearing	kN	68.0	799.4	0.085	PASS
Web plate combined				0.434	PASS
Beam web net shear	kN	68.0	347.1	0.196	PASS

**Design forces**

Design moment (ULS)

 $M_{Ed} = 116.0$ kNm

Design moment (SLS)

 $M_{Ed,ser} = 100.0$ kNm

Design shear force (ULS)

 $V_{Ed} = 68.0$ kN

Design shear force (SLS)

 $V_{Ed,ser} = 70.0$ kN

Design axial force (ULS)

 $N_{Ed} = 110.0$ kN

Design axial force (SLS)

 $N_{Ed,ser} = 110.0$ kN**Partial factors**

Resistance of cross section

 $\gamma_{M0} = 1.00$

Resistance of members to instability

 $\gamma_{M1} = 1.00$

Cross sections in tension to fracture

 $\gamma_{M2} = 1.10$

Resistance of bolts

 $\gamma_{M2,b} = 1.25$

Slip resistance

 $\gamma_{M3} = 1.25$



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Slip resistance, servability

$$\gamma_{M3,ser} = 1.10$$

Structural integrity

$$\gamma_{M,u} = 1.10$$

Beam details

Section name

UB 305x165x40

Steel grade

S275

Yield strength

$$f_{y,bm} = 275 \text{ N/mm}^2$$

Ultimate strength

$$f_{u,bm} = 410 \text{ N/mm}^2$$

Section height

$$h = 303.4 \text{ mm}$$

Section width

$$b = 165.0 \text{ mm}$$

Flange thickness

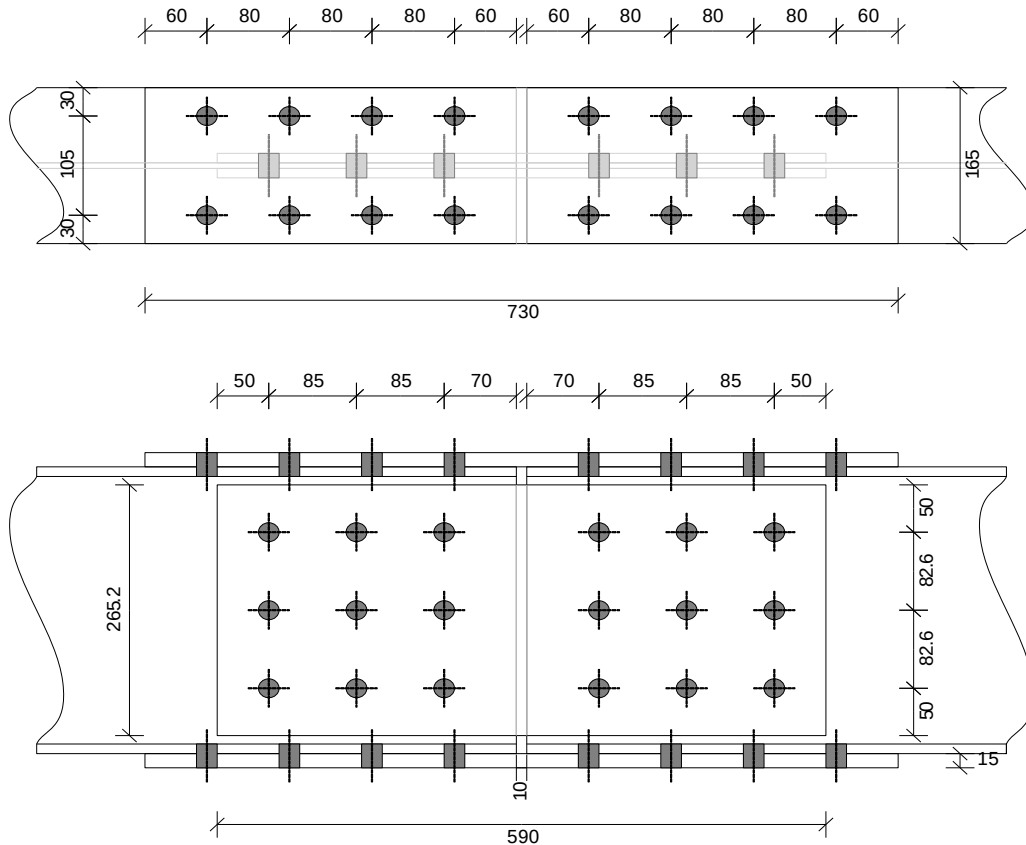
$$t_f = 10.2 \text{ mm}$$

Web thickness

$$t_w = 6.0 \text{ mm}$$

Gap between beams

$$g_v = 10.0 \text{ mm}$$



Flange bolt details

Number of bolt rows

$$n_{1,fp} = 4$$

Bolt size

M20

Bolt grade

8.8

Yield strength

$$f_{yb,fp} = 640 \text{ N/mm}^2$$

Ultimate strength

$$f_{ub,fp} = 800 \text{ N/mm}^2$$

End distance in plate

$$e_{1,fp} = 60 \text{ mm}$$

End distance in beam

$$e_{1,cf} = 60 \text{ mm}$$

Edge distance in plate

$$e_{2,fp} = 30 \text{ mm}$$

Bolt pitch

$$p_{1,fp} = 80 \text{ mm}$$

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Bolt gauge

$$p_{2,fp} = 105 \text{ mm}$$

Flange plate details

Plate width

$$b_{fp,ext} = 2 \times e_{2,fp} + p_{2,fp} = 165 \text{ mm}$$

Plate length

$$l_{fp} = g_v + 2 \times (n_{1,fp} - 1) \times p_{1,fp} + 2 \times (e_{1,cf} + e_{1,fp}) = 730 \text{ mm}$$

Plate thickness

$$t_{fp} = 15 \text{ mm}$$

Plate grade

S275

Yield strength

$$f_{y,fp} = 275 \text{ N/mm}^2$$

Ultimate strength

$$f_{u,fp} = 410 \text{ N/mm}^2$$

Web bolt details

Number of bolt columns

$$n_{1,wp} = 3$$

Number of bolt rows

$$n_{2,wp} = 3$$

Bolt size

M20

Bolt grade

8.8

Yield strength

$$f_{yb,wp} = 640 \text{ N/mm}^2$$

Ultimate strength

$$f_{ub,wp} = 800 \text{ N/mm}^2$$

End distance in plate

$$e_{1,wp} = 50 \text{ mm}$$

End distance in beam

$$e_{1,cw} = 70 \text{ mm}$$

Edge distance in plate

$$e_{2,wp} = 50 \text{ mm}$$

Bolt gauge

$$p_{1,wp} = 85 \text{ mm}$$

Bolt pitch

$$p_{2,wp} = 82.6 \text{ mm}$$

Web plate details

Number of web plates

$$N_{wp} = 2$$

Plate width

$$b_{wp} = g_v + 2 \times (n_{1,wp} - 1) \times p_{1,wp} + 2 \times (e_{1,cw} + e_{1,wp}) = 590 \text{ mm}$$

Plate height

$$h_{wp} = (n_{2,wp} - 1) \times p_{2,wp} + 2 \times e_{2,wp} = 265.2 \text{ mm}$$

Plate thickness

$$t_{wp} = 10 \text{ mm}$$

Plate grade

S275

Yield strength

$$f_{y,wp} = 275 \text{ N/mm}^2$$

Ultimate strength

$$f_{u,wp} = 410 \text{ N/mm}^2$$

Continuity - EN1993-1-8 cl.6.2.7.1(13)

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = W_{pl,y} \times f_{y,bm} / \gamma_{M0} = 171.3 \text{ kNm}$$

Min design bending resist. momnt for continuity

$$M_{Ed,cont} = 0.25 \times M_{c,y,Rd} = 42.8 \text{ kNm}$$

$$M_{Ed,cont} / M_{Ed} = 0.369$$

M_{Ed} is greater or equal to 0.25 × M_{c,y,Rd} so continuity is satisfied

Internal forces - ULS

Second moment of area

$$I_{y,web} = (h - 2 \times t_f)^3 \times t_w / 12 = 1133 \text{ cm}^4$$

Area of web

$$A_w = (h - 2 \times t_f) \times t_w = 1698 \text{ mm}^2$$

Force in each flange due to moment

$$F_{f,M} = (1 - (I_{y,web} / I_y)) \times M_{Ed} / (h - t_f) = 342.9 \text{ kN}$$

Force in each flange due to axial force

$$F_{f,N} = (1 - A_w / A) \times N_{Ed} / 2 = 36.8 \text{ kN}$$

Total force in tension flange

$$F_{tf} = \max(F_{f,M} - F_{f,N}, 0 \text{ kN}) = 306.1 \text{ kN}$$

Total force in compression flange

$$F_{cf} = \max(F_{f,M} + F_{f,N}, 0 \text{ kN}) = 379.7 \text{ kN}$$

Flange bolt forces - ULS

Force in compression flange bolts

$$F_{cf,v} = F_{cf} / (2 \times n_{1,fp}) = 47.5 \text{ kN}$$

Force in tension flange bolts

$$F_{tf,v} = F_{tf} / (2 \times n_{1,fp}) = 38.3 \text{ kN}$$

Maximum design force on flange bolts

$$F_{f,v,Ed} = \max(F_{cf,v}, F_{tf,v}) = 47.5 \text{ kN}$$

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Flange bolt resistance (Table 3.4)

Flange plate bearing strength

$$f_{u,fp} \times t_{fp} = \mathbf{6150 \text{ kN/m}}$$

Beam flange bearing strength

$$f_{u,bm} \times t_f = \mathbf{4182 \text{ kN/m}}$$

Critical section for bearing

Beam flange

Bolt shear area

$$A_{s,fp} = \mathbf{245.0 \text{ mm}^2}$$

Shear area factor

$$\alpha_{v,fp} = \mathbf{0.6}$$

Length of joint

$$L_j = (n_{1,fp} - 1) \times p_{1,fp} = \mathbf{240 \text{ mm}}$$

Reduction factor due to length of joint

$$\beta_{LF,fp} = \mathbf{1.000}$$

Shear resistance of single bolt

$$F_{v,fp,Rd} = \alpha_{v,fp} \times f_{ub,fp} \times A_{s,fp} / \gamma_{M2,b} = \mathbf{94.1 \text{ kN}}$$

Bolt bearing factors

$$k_{1,f} = \min(1.4 \times p_{2,fp} / d_{0,fp} - 1.7, 2.5) = \mathbf{2.50}$$

$$\alpha_{b,f} = \min(e_{1,cf} / (3 \times d_{0,fp}), p_{1,fp} / (3 \times d_{0,fp}) - 1/4, f_{ub,fp} / f_{u,bm}, 1.0) = \mathbf{0.91}$$

Bearing resistance of bolt

$$F_{b,f,Rd} = k_{1,f} \times \alpha_{b,f} \times f_{u,bm} \times d_{b,fp} \times t_f / \gamma_{M2,b} = \mathbf{152.1 \text{ kN}}$$

Design resistance of bolt

$$F_{t,Rd} = \min(F_{v,fp,Rd}, F_{b,f,Rd}) = \mathbf{94.1 \text{ kN}}$$

$$F_{t,v,Ed} / F_{t,Rd} = \mathbf{0.504}$$

PASS - Bolt resistance exceeds applied force on bolt

Internal forces - SLS

Force in each flange due to moment

$$F_{t,M,ser} = (1 - (I_{y,web} / I_y)) \times M_{Ed,ser} / (h - t_f) = \mathbf{295.6 \text{ kN}}$$

Force in each flange due to axial force

$$F_{t,N,ser} = (1 - A_w / A) \times N_{Ed,ser} / 2 = \mathbf{36.8 \text{ kN}}$$

Total force in tension flange

$$F_{tf,ser} = \max(F_{t,M,ser} - F_{t,N,ser}, 0 \text{ kN}) = \mathbf{258.8 \text{ kN}}$$

Total force in compression flange

$$F_{cf,ser} = \max(F_{t,M,ser} + F_{t,N,ser}, 0 \text{ kN}) = \mathbf{332.4 \text{ kN}}$$

Flange bolt forces - SLS

Force in compression flange bolts

$$F_{cf,v,ser} = F_{cf,ser} / (2 \times n_{1,fp}) = \mathbf{41.6 \text{ kN}}$$

Force in tension flange bolts

$$F_{tf,v,ser} = F_{tf,ser} / (2 \times n_{1,fp}) = \mathbf{32.4 \text{ kN}}$$

Maximum design force on flange bolts

$$F_{t,v,Ed,ser} = \max(F_{cf,v,ser}, F_{tf,v,ser}) = \mathbf{41.6 \text{ kN}}$$

Flange bolt resistance SLS

Pre-loading force

$$F_{p,C,fp,ser} = 0.7 \times f_{ub,fp} \times A_{s,fp} = \mathbf{137.2 \text{ kN}}$$

Bolt type factor

$$k_s = \mathbf{1.00}$$

Slip factor

$$\mu = \mathbf{0.50}$$

Slip resistance of single bolt

$$F_{s,fp,Rd,ser} = k_s \times \mu \times F_{p,C,fp,ser} / \gamma_{M3,ser} = \mathbf{62.4 \text{ kN}}$$

$$F_{t,v,Ed,ser} / F_{s,fp,Rd,ser} = \mathbf{0.666}$$

PASS - Bolt resistance exceeds applied force on bolt

Flange cover plates - resistance of plates in compression

Gross area of plate to 1 flange

$$A_{fp} = b_{fp,ext} \times t_{fp} = \mathbf{2475 \text{ mm}^2}$$

Pitch between the bolts rows of the splice

$$p_{1,j} = \max(2 \times e_{1,cf} + g_v, p_{1,fp}) = \mathbf{130 \text{ mm}}$$

Critical buckling length

$$L_{cr} = 0.6 \times p_{1,j} = \mathbf{78 \text{ mm}}$$

Steel strength factor (cl.6.3.1.3)

$$\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_{y,fp})} = \mathbf{0.92}$$

Radius of gyration of plate

$$i_{z,fp} = t_{fp} / \sqrt{(12)} = \mathbf{4.3 \text{ mm}}$$

Non-dimensional slenderness factor (cl.6.3.1.3)

$$\lambda_1 = 93.9 \times \varepsilon = \mathbf{86.80}$$

$$\bar{\lambda} = L_{cr} / i_{z,fp} \times 1 / \lambda_1 = \mathbf{0.21}$$

Buckling curve - solid section (Table 6.2)

$$c$$

Imperfection factor (Table 6.1)

$$\alpha = \mathbf{0.49}$$

Buckling reduction factor (cl.6.3.1.2)

$$\Phi = 0.5 \times (1 + \alpha \times (\bar{\lambda} - 0.2) + \bar{\lambda}^2) = \mathbf{0.52}$$

$$\chi_{fp} = 1 / (\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}) = \mathbf{1.00}$$

Design compressive resistance (eqn. 6.47)

$$N_{b,fp,Rd} = \chi_{fp} \times A_{fp} \times f_{y,fp} / \gamma_{M1} = \mathbf{678.01 \text{ kN}}$$

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Utilisation

$$F_{cf} / N_{b,fp,Rd} = \mathbf{0.560}$$

PASS - Flange cover plate compressive resistance greater than design compressive force

Flange cover plates - resistance of plates in tension

Net area of plate to 1 flange

$$A_{fp,net} = A_{fp} - 2 \times t_{fp} \times d_{0,fp} = \mathbf{1815 \text{ mm}^2}$$

Tension resistance of gross area (eqn.6.6)

$$N_{pl,fp,Rd} = A_{fp} \times f_{y,fp} / \gamma_{M0} = \mathbf{680.63 \text{ kN}}$$

Tension resistance of net area (eqn.6.6)

$$N_{u,fp,Rd} = 0.9 \times A_{fp,net} \times f_{u,fp} / \gamma_{M2} = \mathbf{608.85 \text{ kN}}$$

Design tension resistance

$$N_{t,fp,Rd} = \min(N_{pl,fp,Rd}, N_{u,fp,Rd}) = \mathbf{608.85 \text{ kN}}$$

Utilisation

$$F_{tf} / N_{t,fp,Rd} = \mathbf{0.503}$$

PASS - Flange cover plate tensile resistance greater than design tensile force

Flange cover plates - block tearing

Net area of plate to subject to shear

$$A_{fp,nv} = 2 \times t_{fp} \times (e_{1,fp} + (n_{1,fp} - 1) \times p_{1,fp} - (n_{1,fp} - 0.5) \times d_{0,fp}) = \mathbf{6690 \text{ mm}^2}$$

Net area of plate to subject to tension

$$A_{fp,nt} = t_{fp} \times (2 \times e_{2,fp} - d_{0,fp}) = \mathbf{570 \text{ mm}^2}$$

Block tearing resistance (EN1993-1-8 eqn.3.9)

$$N_{bt,Rd} = f_{u,fp} \times A_{fp,nt} / \gamma_{M2} + f_{y,fp} \times A_{fp,nv} / (\sqrt{3} \times \gamma_{M0}) = \mathbf{1274.63 \text{ kN}}$$

Utilisation

$$F_{tf} / N_{bt,Rd} = \mathbf{0.240}$$

PASS - Flange cover plate block tearing resistance greater than design tensile force

Beam flange - resistance in compression

Gross area of flange

$$A_f = b \times t_f = \mathbf{1683 \text{ mm}^2}$$

Assume χ is 1.0 due to presence of beam web

$$\chi = \mathbf{1.00}$$

Design compressive resistance (eqn. 6.47)

$$N_{b,Rd} = \chi \times A_f \times f_{y,bm} / \gamma_{M1} = \mathbf{462.82 \text{ kN}}$$

Utilisation

$$F_{cf} / N_{b,Rd} = \mathbf{0.820}$$

PASS - Beam flange compressive resistance greater than design compressive force

Beam flange - resistance in tension

Net area of flange

$$A_{f,net} = A_f - 2 \times t_f \times d_{0,fp} = \mathbf{1234.2 \text{ mm}^2}$$

Tension resistance of gross area (eqn.6.6)

$$N_{pl,Rd} = A_f \times f_{y,bm} / \gamma_{M0} = \mathbf{462.82 \text{ kN}}$$

Tension resistance of net area (eqn.6.6)

$$N_{u,Rd} = 0.9 \times A_{f,net} \times f_{u,bm} / \gamma_{M2} = \mathbf{414.02 \text{ kN}}$$

Design tension resistance

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = \mathbf{414.02 \text{ kN}}$$

Utilisation

$$F_{tf} / N_{t,Rd} = \mathbf{0.739}$$

PASS - Beam flange tensile resistance greater than design tensile force

Internal forces - ULS

Moment in the web (at the CL of the splice)

$$M_w = M_{Ed} \times l_{y,web} / l_y = \mathbf{15.5 \text{ kNm}}$$

Force in web due to axial force

$$F_{w,N} = N_{Ed} \times A_w / A = \mathbf{36.4 \text{ kN}}$$

Force in web due to vertical shear

$$F_{w,V} = V_{Ed} = \mathbf{68.0 \text{ kN}}$$

Web bolt forces - ULS

Web bolt group inertia

$$I_{b,wp} = \mathbf{84287 \text{ mm}^2}$$

Extreme horiz. bolt position from group centroid

$$x_{max} = (p_{1,wp} \times (n_{1,wp} - 1)) / 2 = \mathbf{85.0 \text{ mm}}$$

Extreme vert. bolt position from group centroid

$$z_{max} = (p_{2,wp} \times (n_{2,wp} - 1)) / 2 = \mathbf{82.6 \text{ mm}}$$

Number of bolts in bolt group

$$n_{wp} = n_{1,wp} \times n_{2,wp} = \mathbf{9}$$

Add. moment due to eccentricity of bolt group

$$M_{add} = V_{Ed} \times (g_v / 2 + e_{1,cw} + (p_{1,wp} \times (n_{1,wp} - 1)) / 2) = \mathbf{10.9 \text{ kNm}}$$

Horizontal component of force on extreme bolt

$$F_{x,M} = (M_w + M_{add}) \times z_{max} / I_{b,wp} = \mathbf{25.8 \text{ kN}}$$

Vertical component of force on extreme bolt

$$F_{z,M} = (M_w + M_{add}) \times x_{max} / I_{b,wp} = \mathbf{26.6 \text{ kN}}$$

Resultant force on extreme bolt

$$F_{w,v,Ed} = ((F_{z,M} + F_{w,V} / n_{wp})^2 + (F_{x,M} + F_{w,N} / n_{wp})^2)^{0.5} = \mathbf{45.3 \text{ kN}}$$

Web bolt resistance (Table 3.4)

Web plate bearing strength

$$f_{u,wp} \times N_{wp} \times t_{wp} = \mathbf{8200 \text{ kN/m}}$$

Beam web bearing strength

$$f_{u,bm} \times t_w = \mathbf{2460 \text{ kN/m}}$$

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Critical section for bearing

Bolt shear area

Shear area factor

Shear resistance of single bolt

Bolt bearing factors

Bearing resistance of bolt

Design resistance of bolt

Beam web

$$A_{s,wp} = 245.0 \text{ mm}^2$$

$$\alpha_{v,wp} = 0.6$$

$$F_{v,wp,Rd} = N_{wp} \times \alpha_{v,wp} \times f_{ub,wp} \times A_{s,wp} / \gamma_{M2,b} = 188.2 \text{ kN}$$

$$k_{1,w} = \min(1.4 \times p_{2,wp} / d_{0,wp} - 1.7, 2.5) = 2.50$$

$$\alpha_{b,w} = \min(e_{1,cw} / (3 \times d_{0,wp}), p_{1,wp} / (3 \times d_{0,wp}) - 1/4, f_{ub,wp} / f_{u,bm}, 1.0) = 1.00$$

$$F_{b,w,Rd} = k_{1,w} \times \alpha_{b,w} \times f_{u,bm} \times d_{b,wp} \times t_w / \gamma_{M2,b} = 98.4 \text{ kN}$$

$$F_{w,Rd} = \min(F_{v,wp,Rd}, F_{b,w,Rd}) = 98.4 \text{ kN}$$

$$F_{w,v,Ed} / F_{w,Rd} = 0.461$$

PASS - Bolt resistance exceeds applied force on bolt

Internal forces - SLS

Moment in the web (at the CL of the splice)

$$M_{w,ser} = M_{Ed,ser} \times l_{y,web} / l_y = 13.3 \text{ kNm}$$

Force in web due to axial force

$$F_{w,N,ser} = N_{Ed,ser} \times A_w / A = 36.4 \text{ kN}$$

Force in web due to vertical shear

$$F_{w,V,ser} = V_{Ed,ser} = 70.0 \text{ kN}$$

Web bolt forces - SLS

Add. moment due to eccentricity of bolt group

$$M_{add,ser} = V_{Ed,ser} \times (g_v / 2 + e_{1,cw} + (p_{1,wp} \times (n_{1,wp} - 1)) / 2) = 11.2 \text{ kNm}$$

Horizontal component of force on extreme bolt

$$F_{x,M,ser} = (M_{w,ser} + M_{add,ser}) \times z_{max} / l_{b,wp} = 24.0 \text{ kN}$$

Vertical component of force on extreme bolt

$$F_{z,M,ser} = (M_{w,ser} + M_{add,ser}) \times x_{max} / l_{b,wp} = 24.7 \text{ kN}$$

Resultant force on extreme bolt

$$F_{w,v,Ed,ser} = ((F_{z,M,ser} + F_{w,V,ser} / n_{wp})^2 + (F_{x,M,ser} + F_{w,N,ser} / n_{wp})^2)^{0.5} = 43.0 \text{ kN}$$

Web bolt resistance SLS

Pre-loading force

$$F_{p,C,wp,ser} = 0.7 \times f_{ub,wp} \times A_{s,wp} = 137.2 \text{ kN}$$

Slip resistance of single bolt

$$F_{s,wp,Rd,ser} = k_s \times N_{wp} \times \mu \times F_{p,C,wp,ser} / \gamma_{M3,ser} = 124.7 \text{ kN}$$

$$F_{w,v,Ed,ser} / F_{s,wp,Rd,ser} = 0.344$$

PASS - Bolt resistance exceeds applied force on bolt

Resistance of web cover plate in shear

$$\text{Resist. of gross shear area of web plate (Eqn.6.18)} \quad V_{wp,g,Rd} = 2 \times h_{wp} \times t_{wp} \times f_{y,wp} / (1.27 \times \sqrt{3} \times \gamma_{M0}) = 663.09 \text{ kN}$$

Net area of web plate

$$A_{v,wp,net} = (h_{wp} - n_{2,wp} \times d_{0,wp}) \times t_{wp} = 1992 \text{ mm}^2$$

Resist. of net shear area of web plate (Eqn.6.18)

$$V_{wp,net,Rd} = 2 \times A_{v,wp,net} \times f_{u,wp} / (\sqrt{3} \times \gamma_{M2}) = 857.33 \text{ kN}$$

Plastic resistance of web cover plate

$$V_{pl,wp,Rd} = \min(V_{wp,g,Rd}, V_{wp,net,Rd}) = 663.09 \text{ kN}$$

$$V_{Ed} / V_{pl,wp,Rd} = 0.103$$

PASS - Shear resistance of web plate exceeds applied shear

Block tearing of web cover plate

Area subject to tension

$$A_{nt} = t_{wp} \times (e_{1,wp} - d_{0,wp} / 2) = 390 \text{ mm}^2$$

Area subject to shear

$$A_{nv} = t_{wp} \times (h_{wp} - e_{2,wp} - (n_{2,wp} - 0.5) \times d_{0,wp}) = 1602 \text{ mm}^2$$

Block tearing resistance

$$V_{b,Rd} = 2 \times (f_{u,wp} \times A_{nt} / \gamma_{M2} + f_{y,wp} \times A_{nv} / (\sqrt{3} \times \gamma_{M0})) = 799.43 \text{ kN}$$

$$V_{Ed} / V_{b,Rd} = 0.085$$

PASS - Block tearing resistance of web plate exceeds applied shear

Resistance of beam web shear accounting for holes

Height of web

$$h_w = h - 2 \times t_f = 283.0 \text{ mm}$$

$$\eta = 1.000$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2009 \text{ mm}^2$$

Net shear area

$$A_{v,net} = A_v - (n_{2,wp} \times d_{0,wp} \times t_w) = 1613 \text{ mm}^2$$



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Design shear net resistance - cl 6.2.6(2)

$$V_{nw,Rd} = A_{v,net} \times f_{u,bm} / (\sqrt{3} \times \gamma_{M2}) = \mathbf{347.1 \text{ kN}}$$

$$V_{Ed} / V_{nw,Rd} = \mathbf{0.196}$$

PASS - Net shear resistance of beam web exceeds applied shear force

Resistance of web cover plate to combined bending, shear and axial

Elastic modulus of web cover plate

$$W_{wp} = t_{wp} \times h_{wp}^2 / 6 = \mathbf{117218 \text{ mm}^3}$$

Bending resistance of web cover plate

$$M_{c,wp,Rd} = 2 \times W_{wp} \times f_{y,wp} / \gamma_{M0} = \mathbf{64.5 \text{ kNm}}$$

Axial resistance of web cover plate

$$N_{wp,Rd} = 2 \times t_{wp} \times h_{wp} \times f_{y,wp} = \mathbf{1458.6 \text{ kN}}$$

Applied moment

$$M_{wp,Ed} = M_w + M_{add} = \mathbf{26.3 \text{ kNm}}$$

Applied axial force

$$N_{wp,Ed} = F_{w,N} = \mathbf{36.4 \text{ kN}}$$

Interaction formula

$$N_{wp,Ed} / N_{wp,Rd} + M_{wp,Ed} / M_{c,wp,Rd} = \mathbf{0.434}$$

PASS - Combined bending, shear and axial check satisfied



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STEEL BEAM SPLICE DESIGN (EN1993)

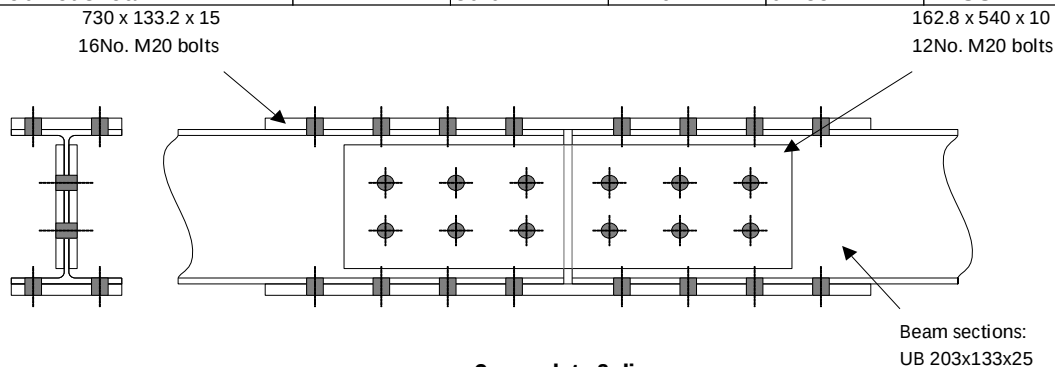
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009

Tedds calculation version 1.0.02

Design summary

Overall design status **PASS**
Overall design utilisation **0.679**

Description	Unit	Design	Resistance	Utilisation	Result
Flange bolt group	kN	24.2	94.1	0.258	PASS
Flange bolt slip resist. SLS	kN	19.8	62.4	0.318	PASS
Flange plate compression	kN	193.9	547.3	0.354	PASS
Flange plate tension flange	kN	160.6	448.8	0.358	PASS
Flange plate block tearing	kN	160.6	1234.4	0.130	PASS
Flange compression	kN	193.9	285.7	0.679	PASS
Flange tension	kN	160.6	233.4	0.688	PASS
Web bolt group	kN	30.2	58.5	0.515	PASS
Web bolt slip resist. SLS	kN	27.4	124.7	0.219	PASS
Web plate shear	kN	30.0	407.1	0.074	PASS
Web plate block tearing	kN	30.0	544.1	0.055	PASS
Web plate combined				0.406	PASS
Beam web net shear	kN	30.0	221.9	0.135	PASS



Design forces

Design moment (ULS) $M_{Ed} = 40.0$ kNm
 Design moment (SLS) $M_{Ed,ser} = 32.0$ kNm
 Design shear force (ULS) $V_{Ed} = 30.0$ kN
 Design shear force (SLS) $V_{Ed,ser} = 30.0$ kN
 Design axial force (ULS) $N_{Ed} = 50.0$ kN
 Design axial force (SLS) $N_{Ed,ser} = 50.0$ kN

Partial factors

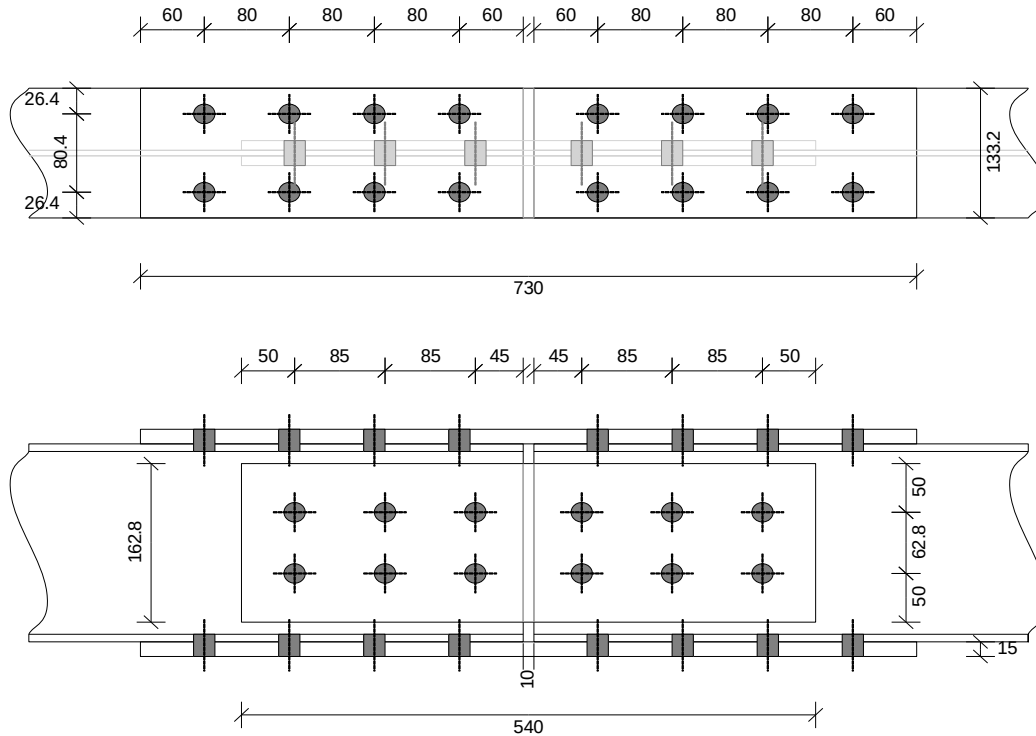
Resistance of cross section $\gamma_{M0} = 1.00$
 Resistance of members to instability $\gamma_{M1} = 1.00$
 Cross sections in tension to fracture $\gamma_{M2} = 1.10$
 Resistance of bolts $\gamma_{M2,b} = 1.25$
 Slip resistance $\gamma_{M3} = 1.25$
 Slip resistance, servability $\gamma_{M3,ser} = 1.10$
 Structural integrity $\gamma_{M,u} = 1.10$



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Beam details

Section name	UB 203x133x25
Steel grade	S275
Yield strength	$f_{y,bm} = 275 \text{ N/mm}^2$
Ultimate strength	$f_{u,bm} = 410 \text{ N/mm}^2$
Section height	$h = 203.2 \text{ mm}$
Section width	$b = 133.2 \text{ mm}$
Flange thickness	$t_f = 7.8 \text{ mm}$
Web thickness	$t_w = 5.7 \text{ mm}$
Gap between beams	$g_v = 10.0 \text{ mm}$



Flange bolt details

Number of bolt rows	$n_{1,fp} = 4$
Bolt size	M20
Bolt grade	8.8
Yield strength	$f_{yb,fp} = 640 \text{ N/mm}^2$
Ultimate strength	$f_{ub,fp} = 800 \text{ N/mm}^2$
End distance in plate	$e_{1,fp} = 60 \text{ mm}$
End distance in beam	$e_{1,cf} = 60 \text{ mm}$
Edge distance in plate	$e_{2,fp} = 26.4 \text{ mm}$
Bolt pitch	$p_{1,fp} = 80 \text{ mm}$
Bolt gauge	$p_{2,fp} = 80.4 \text{ mm}$

Flange plate details

Plate width	$b_{fp,ext} = 2 \times e_{2,fp} + p_{2,fp} = 133.2 \text{ mm}$
Plate length	$l_{fp} = g_v + 2 \times (n_{1,fp} - 1) \times p_{1,fp} + 2 \times (e_{1,cf} + e_{1,fp}) = 730 \text{ mm}$
Plate thickness	$t_{fp} = 15 \text{ mm}$

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Plate grade **S275**
 Yield strength $f_{y,fp} = 275 \text{ N/mm}^2$
 Ultimate strength $f_{u,fp} = 410 \text{ N/mm}^2$

Web bolt details

Number of bolt columns $n_{1,wp} = 3$
 Number of bolt rows $n_{2,wp} = 2$
 Bolt size **M20**
 Bolt grade **8.8**
 Yield strength $f_{yb,wp} = 640 \text{ N/mm}^2$
 Ultimate strength $f_{ub,wp} = 800 \text{ N/mm}^2$
 End distance in plate $e_{1,wp} = 50 \text{ mm}$
 End distance in beam $e_{1,cw} = 45 \text{ mm}$
 Edge distance in plate $e_{2,wp} = 50 \text{ mm}$
 Bolt gauge $p_{1,wp} = 85 \text{ mm}$
 Bolt pitch $p_{2,wp} = 62.8 \text{ mm}$

Web plate details

Number of web plates $N_{wp} = 2$
 Plate width $b_{wp} = g_v + 2 \times (n_{1,wp} - 1) \times p_{1,wp} + 2 \times (e_{1,cw} + e_{1,wp}) = 540 \text{ mm}$
 Plate height $h_{wp} = (n_{2,wp} - 1) \times p_{2,wp} + 2 \times e_{2,wp} = 162.8 \text{ mm}$
 Plate thickness $t_{wp} = 10 \text{ mm}$
 Plate grade **S275**
 Yield strength $f_{y,wp} = 275 \text{ N/mm}^2$
 Ultimate strength $f_{u,wp} = 410 \text{ N/mm}^2$

Continuity - EN1993-1-8 cl.6.2.7.1(13)

Design bending resistance moment - eq 6.13
 Min design bending resist. momnt for continuity

$$M_{c,y,Rd} = W_{ply} \times f_{y,bm} / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{Ed,cont} = 0.25 \times M_{c,y,Rd} = 17.7 \text{ kNm}$$

$$M_{Ed,cont} / M_{Ed} = 0.443$$

M_{Ed} is greater or equal to $0.25 \times M_{c,y,Rd}$ so continuity is satisfied

Internal forces - ULS

Second moment of area $I_{y,web} = (h - 2 \times t_f)^3 \times t_w / 12 = 314 \text{ cm}^4$
 Area of web $A_w = (h - 2 \times t_f) \times t_w = 1069 \text{ mm}^2$
 Force in each flange due to moment $F_{f,M} = (1 - (I_{y,web} / I_y)) \times M_{Ed} / (h - t_f) = 177.3 \text{ kN}$
 Force in each flange due to axial force $F_{f,N} = (1 - A_w / A) \times N_{Ed} / 2 = 16.6 \text{ kN}$
 Total force in tension flange $F_{tf} = \max(F_{f,M} - F_{f,N}, 0 \text{ kN}) = 160.6 \text{ kN}$
 Total force in compression flange $F_{cf} = \max(F_{f,M} + F_{f,N}, 0 \text{ kN}) = 193.9 \text{ kN}$

Flange bolt forces - ULS

Force in compression flange bolts $F_{cf,v} = F_{cf} / (2 \times n_{1,fp}) = 24.2 \text{ kN}$
 Force in tension flange bolts $F_{tf,v} = F_{tf} / (2 \times n_{1,fp}) = 20.1 \text{ kN}$
 Maximum design force on flange bolts $F_{f,v,Ed} = \max(F_{cf,v}, F_{tf,v}) = 24.2 \text{ kN}$

Flange bolt resistance (Table 3.4)

Flange plate bearing strength $f_{u,fp} \times t_{fp} = 6150 \text{ kN/m}$
 Beam flange bearing strength $f_{u,bm} \times t_f = 3198 \text{ kN/m}$
 Critical section for bearing Beam flange
 Bolt shear area $A_{s,fp} = 245.0 \text{ mm}^2$
 Shear area factor $\alpha_{v,fp} = 0.6$

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Length of joint	$L_j = (n_{1,fp} - 1) \times p_{1,fp} = 240 \text{ mm}$
Reduction factor due to length of joint	$\beta_{LF,fp} = 1.000$
Shear resistance of single bolt	$F_{v,fp,Rd} = \alpha_{v,fp} \times f_{ub,fp} \times A_{s,fp} / \gamma_{M2,b} = 94.1 \text{ kN}$
Bolt bearing factors	$k_{1,f} = \min(1.4 \times p_{2,fp} / d_{0,fp} - 1.7, 2.5) = 2.50$
	$\alpha_{b,f} = \min(e_{1,cf} / (3 \times d_{0,fp}), p_{1,fp} / (3 \times d_{0,fp}) - 1/4, f_{ub,fp} / f_{u,bm}, 1.0) = 0.91$
Bearing resistance of bolt	$F_{b,f,Rd} = k_{1,f} \times \alpha_{b,f} \times f_{u,bm} \times d_{b,fp} \times t_f / \gamma_{M2,b} = 116.3 \text{ kN}$
Design resistance of bolt	$F_{t,Rd} = \min(F_{v,fp,Rd}, F_{b,f,Rd}) = 94.1 \text{ kN}$
	$F_{t,v,Ed} / F_{t,Rd} = 0.258$
PASS - Bolt resistance exceeds applied force on bolt	

Internal forces - SLS

Force in each flange due to moment	$F_{t,M,ser} = (1 - (I_{y,web} / I_y)) \times M_{Ed,ser} / (h - t_f) = 141.8 \text{ kN}$
Force in each flange due to axial force	$F_{t,N,ser} = (1 - A_w / A) \times N_{Ed,ser} / 2 = 16.6 \text{ kN}$
Total force in tension flange	$F_{tf,ser} = \max(F_{t,M,ser} - F_{t,N,ser}, 0 \text{ kN}) = 125.2 \text{ kN}$
Total force in compression flange	$F_{cf,ser} = \max(F_{t,M,ser} + F_{t,N,ser}, 0 \text{ kN}) = 158.5 \text{ kN}$

Flange bolt forces - SLS

Force in compression flange bolts	$F_{cf,v,ser} = F_{cf,ser} / (2 \times n_{1,fp}) = 19.8 \text{ kN}$
Force in tension flange bolts	$F_{tf,v,ser} = F_{tf,ser} / (2 \times n_{1,fp}) = 15.6 \text{ kN}$
Maximum design force on flange bolts	$F_{t,v,Ed,ser} = \max(F_{cf,v,ser}, F_{tf,v,ser}) = 19.8 \text{ kN}$

Flange bolt resistance SLS

Pre-loading force	$F_{p,C,fp,ser} = 0.7 \times f_{ub,fp} \times A_{s,fp} = 137.2 \text{ kN}$
Bolt type factor	$k_s = 1.00$
Slip factor	$\mu = 0.50$
Slip resistance of single bolt	$F_{s,fp,Rd,ser} = k_s \times \mu \times F_{p,C,fp,ser} / \gamma_{M3,ser} = 62.4 \text{ kN}$
	$F_{t,v,Ed,ser} / F_{s,fp,Rd,ser} = 0.318$

PASS - Bolt resistance exceeds applied force on bolt

Flange cover plates - resistance of plates in compression

Gross area of plate to 1 flange	$A_{fp} = b_{fp,ext} \times t_{fp} = 1998 \text{ mm}^2$
Pitch between the bolts rows of the splice	$p_{1,j} = \max(2 \times e_{1,cf} + g_v, p_{1,fp}) = 130 \text{ mm}$
Critical buckling length	$L_{cr} = 0.6 \times p_{1,j} = 78 \text{ mm}$
Steel strength factor (cl.6.3.1.3)	$\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_{y,fp})} = 0.92$
Radius of gyration of plate	$i_{z,fp} = t_{fp} / \sqrt{12} = 4.3 \text{ mm}$
Non-dimensional slenderness factor (cl.6.3.1.3)	$\lambda_1 = 93.9 \times \varepsilon = 86.80$
	$\bar{\lambda} = L_{cr} / i_{z,fp} \times 1 / \lambda_1 = 0.21$
Buckling curve - solid section (Table 6.2)	c
Imperfection factor (Table 6.1)	$\alpha = 0.49$
Buckling reduction factor (cl.6.3.1.2)	$\Phi = 0.5 \times (1 + \alpha \times (\bar{\lambda} - 0.2) + \bar{\lambda}^2) = 0.52$
	$\chi_{fp} = 1 / (\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}) = 1.00$
Design compressive resistance (eqn. 6.47)	$N_{b,fp,Rd} = \chi_{fp} \times A_{fp} \times f_{y,fp} / \gamma_{M1} = 547.34 \text{ kN}$
Utilisation	$F_{cf} / N_{b,fp,Rd} = 0.354$

PASS - Flange cover plate compressive resistance greater than design compressive force

Flange cover plates - resistance of plates in tension

Net area of plate to 1 flange	$A_{fp,net} = A_{fp} - 2 \times t_{fp} \times d_{0,fp} = 1338 \text{ mm}^2$
Tension resistance of gross area (eqn.6.6)	$N_{pl,fp,Rd} = A_{fp} \times f_{y,fp} / \gamma_{M0} = 549.45 \text{ kN}$
Tension resistance of net area (eqn.6.6)	$N_{u,fp,Rd} = 0.9 \times A_{fp,net} \times f_{u,fp} / \gamma_{M2} = 448.84 \text{ kN}$

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Design tension resistance

$$N_{t,fp,Rd} = \min(N_{pl,fp,Rd}, N_{u,fp,Rd}) = \mathbf{448.84 \text{ kN}}$$

Utilisation

$$F_{tf} / N_{t,fp,Rd} = \mathbf{0.358}$$

PASS - Flange cover plate tensile resistance greater than design tensile force

Flange cover plates - block tearing

Net area of plate to subject to shear

$$A_{fp,nv} = 2 \times t_{fp} \times (e_{1,fp} + (n_{1,fp} - 1) \times p_{1,fp} - (n_{1,fp} - 0.5) \times d_{0,fp}) = \mathbf{6690 \text{ mm}^2}$$

Net area of plate to subject to tension

$$A_{fp,nt} = t_{fp} \times (2 \times e_{2,fp} - d_{0,fp}) = \mathbf{462 \text{ mm}^2}$$

Block tearing resistance (EN1993-1-8 eqn.3.9)

$$N_{bt,Rd} = f_{u,fp} \times A_{fp,nt} / \gamma_{M2} + f_{y,fp} \times A_{fp,nv} / (\sqrt{3} \times \gamma_{M0}) = \mathbf{1234.38 \text{ kN}}$$

Utilisation

$$F_{tf} / N_{bt,Rd} = \mathbf{0.130}$$

PASS - Flange cover plate block tearing resistance greater than design tensile force

Beam flange - resistance in compression

Gross area of flange

$$A_f = b \times t_f = \mathbf{1039 \text{ mm}^2}$$

Assume χ is 1.0 due to presence of beam web

$$\chi = \mathbf{1.00}$$

Design compressive resistance (eqn. 6.47)

$$N_{b,Rd} = \chi \times A_f \times f_{y,bm} / \gamma_{M1} = \mathbf{285.71 \text{ kN}}$$

Utilisation

$$F_{cf} / N_{b,Rd} = \mathbf{0.679}$$

PASS - Beam flange compressive resistance greater than design compressive force

Beam flange - resistance in tension

Net area of flange

$$A_{f,net} = A_f - 2 \times t_f \times d_{0,fp} = \mathbf{695.8 \text{ mm}^2}$$

Tension resistance of gross area (eqn.6.6)

$$N_{pl,Rd} = A_f \times f_{y,bm} / \gamma_{M0} = \mathbf{285.71 \text{ kN}}$$

Tension resistance of net area (eqn.6.6)

$$N_{u,Rd} = 0.9 \times A_{f,net} \times f_{u,bm} / \gamma_{M2} = \mathbf{233.4 \text{ kN}}$$

Design tension resistance

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = \mathbf{233.4 \text{ kN}}$$

Utilisation

$$F_{tf} / N_{t,Rd} = \mathbf{0.688}$$

PASS - Beam flange tensile resistance greater than design tensile force

Internal forces - ULS

Moment in the web (at the CL of the splice)

$$M_w = M_{Ed} \times I_{y,web} / I_y = \mathbf{5.4 \text{ kNm}}$$

Force in web due to axial force

$$F_{w,N} = N_{Ed} \times A_w / A = \mathbf{16.7 \text{ kN}}$$

Force in web due to vertical shear

$$F_{w,V} = V_{Ed} = \mathbf{30.0 \text{ kN}}$$

Web bolt forces - ULS

Web bolt group inertia

$$I_{b,wp} = \mathbf{34816 \text{ mm}^2}$$

Extreme horiz. bolt position from group centroid

$$x_{max} = (p_{1,wp} \times (n_{1,wp} - 1)) / 2 = \mathbf{85.0 \text{ mm}}$$

Extreme vert. bolt position from group centroid

$$z_{max} = (p_{2,wp} \times (n_{2,wp} - 1)) / 2 = \mathbf{31.4 \text{ mm}}$$

Number of bolts in bolt group

$$n_{wp} = n_{1,wp} \times n_{2,wp} = \mathbf{6}$$

Add. moment due to eccentricity of bolt group

$$M_{add} = V_{Ed} \times (g_v / 2 + e_{1,cw} + (p_{1,wp} \times (n_{1,wp} - 1)) / 2) = \mathbf{4.1 \text{ kNm}}$$

Horizontal component of force on extreme bolt

$$F_{x,M} = (M_w + M_{add}) \times z_{max} / I_{b,wp} = \mathbf{8.5 \text{ kN}}$$

Vertical component of force on extreme bolt

$$F_{z,M} = (M_w + M_{add}) \times x_{max} / I_{b,wp} = \mathbf{23.0 \text{ kN}}$$

Resultant force on extreme bolt

$$F_{w,v,Ed} = ((F_{z,M} + F_{w,V} / n_{wp})^2 + (F_{x,M} + F_{w,N} / n_{wp})^2)^{0.5} = \mathbf{30.2 \text{ kN}}$$

Web bolt resistance (Table 3.4)

Web plate bearing strength

$$f_{u,wp} \times N_{wp} \times t_{wp} = \mathbf{8200 \text{ kN/m}}$$

Beam web bearing strength

$$f_{u,bm} \times t_w = \mathbf{2337 \text{ kN/m}}$$

Critical section for bearing

Beam web

Bolt shear area

$$A_{s,wp} = \mathbf{245.0 \text{ mm}^2}$$

Shear area factor

$$\alpha_{v,wp} = \mathbf{0.6}$$

Shear resistance of single bolt

$$F_{v,wp,Rd} = N_{wp} \times \alpha_{v,wp} \times f_{ub,wp} \times A_{s,wp} / \gamma_{M2,b} = \mathbf{188.2 \text{ kN}}$$

Bolt bearing factors

$$k_{1,w} = \min(1.4 \times p_{2,wp} / d_{0,wp} - 1.7, 2.5) = \mathbf{2.30}$$

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Bearing resistance of bolt

Design resistance of bolt

$$\alpha_{b,w} = \min(e_{1,cw} / (3 \times d_{0,wp}), p_{1,wp} / (3 \times d_{0,wp}) - 1/4, f_{ub,wp} / f_{u,bm}, 1.0) = \mathbf{0.68}$$

$$F_{b,w,Rd} = k_{1,w} \times \alpha_{b,w} \times f_{u,bm} \times d_{b,wp} \times t_w / \gamma_{M2,b} = \mathbf{58.5 \text{ kN}}$$

$$F_{w,Rd} = \min(F_{v,wp,Rd}, F_{b,w,Rd}) = \mathbf{58.5 \text{ kN}}$$

$$F_{w,v,Ed} / F_{w,Rd} = \mathbf{0.515}$$

PASS - Bolt resistance exceeds applied force on bolt

Internal forces - SLS

Moment in the web (at the CL of the splice)

Force in web due to axial force

Force in web due to vertical shear

$$M_{w,ser} = M_{Ed,ser} \times l_{y,web} / l_y = \mathbf{4.3 \text{ kNm}}$$

$$F_{w,N,ser} = N_{Ed,ser} \times A_w / A = \mathbf{16.7 \text{ kN}}$$

$$F_{w,V,ser} = V_{Ed,ser} = \mathbf{30.0 \text{ kN}}$$

Web bolt forces - SLS

Add. moment due to eccentricity of bolt group

Horizontal component of force on extreme bolt

Vertical component of force on extreme bolt

Resultant force on extreme bolt

$$M_{add,ser} = V_{Ed,ser} \times (g_v / 2 + e_{1,cw} + (p_{1,wp} \times (n_{1,wp} - 1)) / 2) = \mathbf{4.1 \text{ kNm}}$$

$$F_{x,M,ser} = (M_{w,ser} + M_{add,ser}) \times Z_{max} / l_{b,wp} = \mathbf{7.5 \text{ kN}}$$

$$F_{z,M,ser} = (M_{w,ser} + M_{add,ser}) \times x_{max} / l_{b,wp} = \mathbf{20.4 \text{ kN}}$$

$$F_{w,v,Ed,ser} = ((F_{z,M,ser} + F_{w,V,ser} / n_{wp})^2 + (F_{x,M,ser} + F_{w,N,ser} / n_{wp})^2)^{0.5} = \mathbf{27.4 \text{ kN}}$$

Web bolt resistance SLS

Pre-loading force

Slip resistance of single bolt

$$F_{p,C,wp,ser} = 0.7 \times f_{ub,wp} \times A_{s,wp} = \mathbf{137.2 \text{ kN}}$$

$$F_{s,wp,Rd,ser} = k_s \times N_{wp} \times \mu \times F_{p,C,wp,ser} / \gamma_{M3,ser} = \mathbf{124.7 \text{ kN}}$$

$$F_{w,v,Ed,ser} / F_{s,wp,Rd,ser} = \mathbf{0.219}$$

PASS - Bolt resistance exceeds applied force on bolt

Resistance of web cover plate in shear

Resist. of gross shear area of web plate (Eqn.6.18) $V_{wp,g,Rd} = 2 \times h_{wp} \times t_{wp} \times f_{y,wp} / (1.27 \times \sqrt{3} \times \gamma_{M0}) = \mathbf{407.05 \text{ kN}}$

Net area of web plate

$$A_{v,wp,net} = (h_{wp} - n_{2,wp} \times d_{0,wp}) \times t_{wp} = \mathbf{1188 \text{ mm}^2}$$

Resist. of net shear area of web plate (Eqn.6.18)

$$V_{wp,net,Rd} = 2 \times A_{v,wp,net} \times f_{u,wp} / (\sqrt{3} \times \gamma_{M2}) = \mathbf{511.30 \text{ kN}}$$

Plastic resistance of web cover plate

$$V_{pl,wp,Rd} = \min(V_{wp,g,Rd}, V_{wp,net,Rd}) = \mathbf{407.05 \text{ kN}}$$

$$V_{Ed} / V_{pl,wp,Rd} = \mathbf{0.074}$$

PASS - Shear resistance of web plate exceeds applied shear

Block tearing of web cover plate

Area subject to tension

$$A_{nt} = t_{wp} \times (e_{1,wp} - d_{0,wp} / 2) = \mathbf{390 \text{ mm}^2}$$

Area subject to shear

$$A_{nv} = t_{wp} \times (h_{wp} - e_{2,wp} - (n_{2,wp} - 0.5) \times d_{0,wp}) = \mathbf{798 \text{ mm}^2}$$

Block tearing resistance

$$V_{b,Rd} = 2 \times (f_{u,wp} \times A_{nt} / \gamma_{M2} + f_{y,wp} \times A_{nv} / (\sqrt{3} \times \gamma_{M0})) = \mathbf{544.13 \text{ kN}}$$

$$V_{Ed} / V_{b,Rd} = \mathbf{0.055}$$

PASS - Block tearing resistance of web plate exceeds applied shear

Resistance of beam web shear accounting for holes

Height of web

$$h_w = h - 2 \times t_f = \mathbf{187.6 \text{ mm}}$$

$$\eta = \mathbf{1.000}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1282 \text{ mm}^2}$$

Net shear area

$$A_{v,net} = A_v - (n_{2,wp} \times d_{0,wp} \times t_w) = \mathbf{1031 \text{ mm}^2}$$

Design shear net resistance - cl 6.2.6(2)

$$V_{nw,Rd} = A_{v,net} \times f_{u,bm} / (\sqrt{3} \times \gamma_{M2}) = \mathbf{221.9 \text{ kN}}$$

$$V_{Ed} / V_{nw,Rd} = \mathbf{0.135}$$

PASS - Net shear resistance of beam web exceeds applied shear force

Resistance of web cover plate to combined bending, shear and axial

Elastic modulus of web cover plate

$$W_{wp} = t_{wp} \times h_{wp}^2 / 6 = \mathbf{44173 \text{ mm}^3}$$



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Bending resistance of web cover plate

$$M_{c,wp,Rd} = 2 \times W_{wp} \times f_{y,wp} / \gamma_{M0} = \mathbf{24.3 \text{ kNm}}$$

Axial resistance of web cover plate

$$N_{wp,Rd} = 2 \times t_{wp} \times h_{wp} \times f_{y,wp} = \mathbf{895.4 \text{ kN}}$$

Applied moment

$$M_{wp,Ed} = M_w + M_{add} = \mathbf{9.4 \text{ kNm}}$$

Applied axial force

$$N_{wp,Ed} = F_{w,N} = \mathbf{16.7 \text{ kN}}$$

Interaction formula

$$N_{wp,Ed} / N_{wp,Rd} + M_{wp,Ed} / M_{c,wp,Rd} = \mathbf{0.406}$$

PASS - Combined bending, shear and axial check satisfied

Top Of Second Floor

Load Bearing Structural Elements Check



GENERAL ARRANGEMENT
TOP OF SECOND FLOOR

Floor Joists

Max. Length $\leq$ 4700mm

Floor Loads:

Permenant: 1.2 kN/m²

Live: 1.5 kN/m²

Existing Joist C16+ 47/275 SUFFICIENT

Beams Check

Existing Steel Beam 1 Check

Loads

Max. Span 6300mm

Max. Loads:

Permenant: 6 kN/m

Live: 7.5 kN/m

Min. Required Section UB 356x171x45 S275

(See Calculations Below)

Existing Steel Beam 4 Check

Loads

Max. Span 6500mm

Max. Loads:

Permenant: 3 kN/m

Live: 4 kN/m

Min. Required Section 2 No UB 305x127x37 S275

(See Calculations Below)

New Beam 1

Loads

Max. Span 5000mm

Max. Loads:

Permenant: 6 kN/m

Live: 7.5 kN/m

Min. Required Section UB 254x146x31 S355

(See Calculations Below)

New Beam 2

Loads

Max. Span 5000mm

Max. Loads:

Permenant: 11.5 kN/m

Live: 10 kN/m

Min. Required Section UB 305x165x40 S355

(See Calculations Below)

New Beam 3

Loads

Max. Span 7200mm

Max. Loads:

Permenant: 4.5 kN/m

Live: 5.5 kN/m

Min. Required Section 2 No UB 305x165x40 S355

(See Calculations Below)

Existing Timber Beam 2 Check

Loads

Max. Span 1800mm

Max. Loads:

Permenant: 4.6 kN/m

Live: 5.7 kN/m

Min. Required Section C16 100x300

(See Calculations Below)



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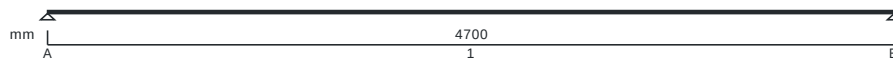
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TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

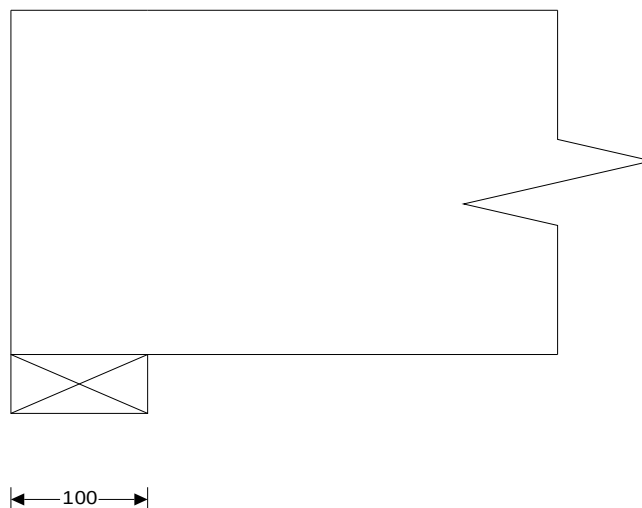
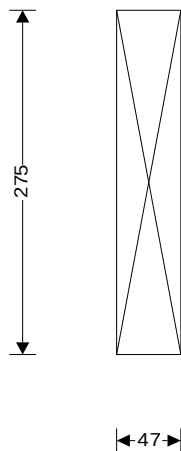
Joist details

Joist breadth $b = 47 \text{ mm}$
 Joist depth $h = 275 \text{ mm}$
 Joist spacing $s = 400 \text{ mm}$
 Timber strength class **C24**
 Service class of timber **1**



Span details

Number of spans $N_{\text{span}} = 1$
 Length of bearing $L_b = 100 \text{ mm}$
 Effective length of span $L_{s1} = 4700 \text{ mm}$



Section properties

Second moment of area $I = b \times h^3 / 12 = 81454427 \text{ mm}^4$
 Section modulus $Z = b \times h^2 / 6 = 592396 \text{ mm}^3$

Loading details

Joist self weight $F_{\text{swt}} = b \times h \times \rho_{\text{char}} \times g_{\text{acc}} = 0.04 \text{ kN/m}$
 Dead load $F_{\text{d_udl}} = 1.20 \text{ kN/m}^2$
 Imposed UDL(Long term) $F_{\text{i_udl}} = 1.50 \text{ kN/m}^2$
 Imposed point load (Medium term) $F_{\text{i_pt}} = 1.40 \text{ kN}$

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Modification factors

Service class for bending parallel to grain	$K_{2m} = 1.00$
Service class for compression	$K_{2c} = 1.00$
Service class for shear parallel to grain	$K_{2s} = 1.00$
Service class for modulus of elasticity	$K_{2e} = 1.00$
Section depth factor	$K_7 = 1.01$
Load sharing factor	$K_8 = 1.10$

Consider long term loads

Load duration factor	$K_3 = 1.00$
Maximum bending moment	$M = 3.105 \text{ kNm}$
Maximum shear force	$V = 2.642 \text{ kN}$
Maximum support reaction	$R = 2.642 \text{ kN}$
Maximum deflection	$\delta = 8.548 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 7.500 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 8.329 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 5.241 \text{ N/mm}^2$
PASS - Applied bending stress within permissible limits	

Check shear stress

Shear stress	$\tau = 0.710 \text{ N/mm}^2$
Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.781 \text{ N/mm}^2$
Applied shear stress	$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.307 \text{ N/mm}^2$
PASS - Applied shear stress within permissible limits	

Check bearing stress

Compression perpendicular to grain (no wane)	$\sigma_{cp1} = 2.400 \text{ N/mm}^2$
Permissible bearing stress	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.640 \text{ N/mm}^2$
Applied bearing stress	$\sigma_{c_max} = R / (b \times L_b) = 0.562 \text{ N/mm}^2$
PASS - Applied bearing stress within permissible limits	

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$
Bending deflection (based on E_{mean})	$\delta_{bending} = 8.121 \text{ mm}$
Shear deflection	$\delta_{shear} = 0.427 \text{ mm}$
Total deflection	$\delta = \delta_{bending} + \delta_{shear} = 8.548 \text{ mm}$
PASS - Actual deflection within permissible limits	

Consider medium term loads

Load duration factor	$K_3 = 1.25$
Maximum bending moment	$M = 3.093 \text{ kNm}$
Maximum shear force	$V = 2.632 \text{ kN}$
Maximum support reaction	$R = 2.632 \text{ kN}$
Maximum deflection	$\delta = 7.655 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 7.500 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 10.412 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 5.221 \text{ N/mm}^2$



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PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress

$$\tau = 0.710 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976 \text{ N/mm}^2$$

Applied shear stress

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.305 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = 2.400 \text{ N/mm}^2$$

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.300 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = 0.560 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = 7.229 \text{ mm}$$

Shear deflection

$$\delta_{shear} = 0.425 \text{ mm}$$

Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = 7.655 \text{ mm}$$

PASS - Actual deflection within permissible limits



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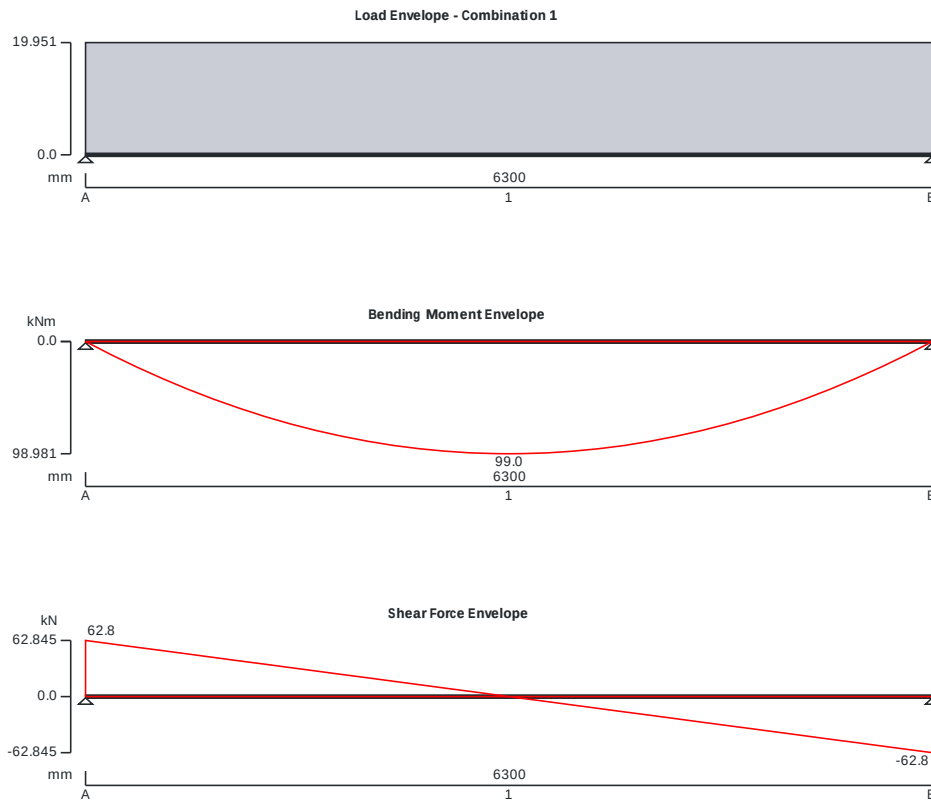
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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 6 kN/m

Variable full UDL 7.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



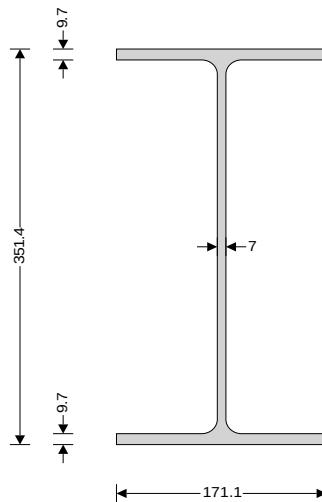
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Analysis results

Maximum moment	$M_{max} = 99 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 62.8 \text{ kN}$	$V_{min} = -62.8 \text{ kN}$
Deflection	$\delta_{max} = 11.2 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 62.8 \text{ kN}$	$R_{A_{min}} = 62.8 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 20.3 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{Variable}} = 23.6 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 62.8 \text{ kN}$	$R_{B_{min}} = 62.8 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 20.3 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{Variable}} = 23.6 \text{ kN}$	

Section details

Section type	UB 356x171x45 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_r, t_w) = 9.7 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = \mathbf{306.6 \text{ mm}}$$

$$c / t_w = 47.4 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{69.4 \text{ mm}}$$

$$c / t_f = 7.7 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{332 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{62.8 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2777 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{440.9 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{99 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{215.2 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.966}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{6300 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{123 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{1.323}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$c$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.49}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.383}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.464}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.986}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.470}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{101.2 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment



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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 17.5 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 11.169 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

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SIMPLY SUPPORTED TIMBER BEAM

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.23

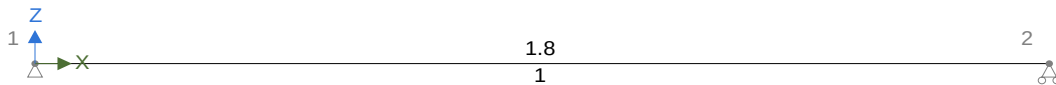
Beam results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	2.0	1.0	0.504	PASS
Bending stress	N/mm ²	14.9	4.0	0.270	PASS
Shear stress	N/mm ²	3.0	1.0	0.336	PASS
Deflection	mm	7.2	2.3	0.326	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - C16 (EC5) - 100x300



Span	Length (m)	Section	Start Support	End Support
1	1.8	100x300	Pinned	Roller Pin X

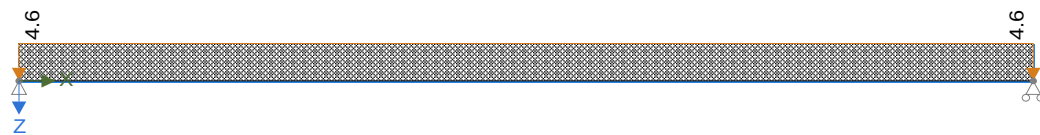
100x300: Area 300 cm², Inertia Major 22500 cm⁴, Inertia Minor 2500 cm⁴, Shear area parallel to Minor 250 cm², Shear area parallel to Major = 250 cm²

C16 (EC5): Density 370 kg/m³, Youngs 8 kN/mm², Shear 0.5 kN/mm², Thermal 0 °C⁻¹

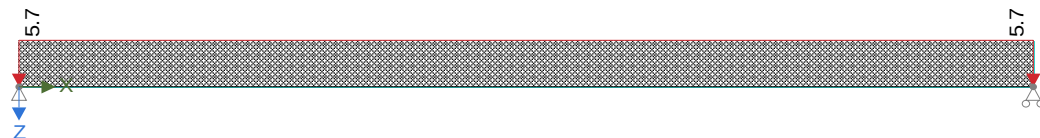
Loading

Self weight included

Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



Load combination factors

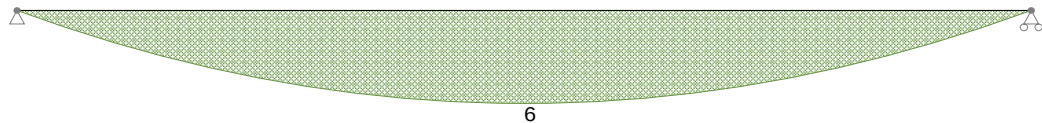
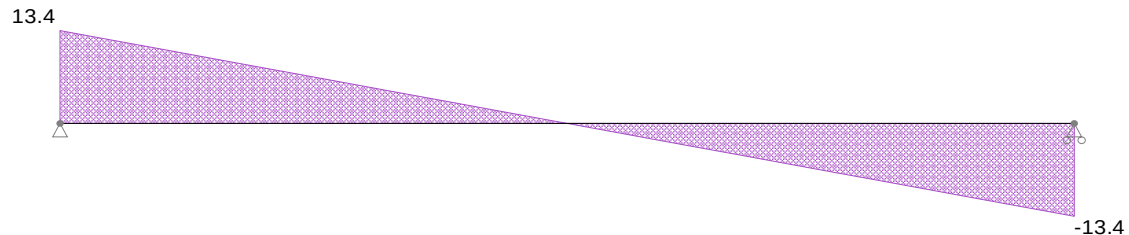
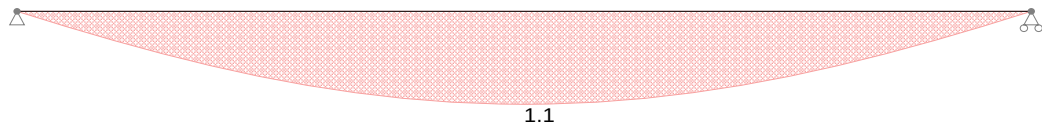
Load combination	Self Weight	Permanent	Imposed
Default (Strength)	1.35	1.35	1.50
Default (Service)	1.00	1.00	1.00



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Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	4.6 kN/m
Beam	Imposed	UDL	GlobalZ	5.7 kN/m

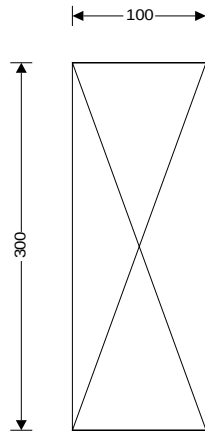
Results**Forces****Strength combinations - Moment envelope (kNm)****Strength combinations - Shear envelope (kN)****Service combinations - Deflection envelope (mm)****Beam - Span 1****Partial factor for material properties and resistances**Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$ **Member details**

Load duration - cl.2.3.1.2 Instantaneous

Service class - cl.2.3.1.3 1

Timber section detailsNumber of timber sections in member $N = 1$ Breadth of sections $b = 100$ mmDepth of sections $h = 300$ mmTimber strength class - EN 338:2016 Table 1 **C16**

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100x300 timber section

Cross-sectional area, A , 30000 mm²
 Section modulus, W_y , 1500000 mm³
 Section modulus, W_z , 500000 mm³
 Second moment of area, I_y , 225000000 mm⁴
 Second moment of area, I_z , 25000000 mm⁴
 Radius of gyration, i_y , 86.6 mm
 Radius of gyration, i_z , 28.9 mm
Timber strength class C16
 Characteristic bending strength, $f_{m,k}$, 16 N/mm²
 Characteristic shear strength, $f_{v,k}$, 3.2 N/mm²
 Characteristic compression strength parallel to grain, $f_{t,0,k}$, 17 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{t,90,k}$, 2.2 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 8.5 N/mm²
 Mean modulus of elasticity, $E_{0,mean}$, 8000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0.05}$, 5400 N/mm²
 Shear modulus of elasticity, G_{mean} , 500 N/mm²
 Characteristic density, ρ_k , 310 kg/m³
 Mean density, ρ_{mean} , 370 kg/m³

Span details

Bearing length $L_b = 100$ mm

Consider Combination 1 - Default (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 1.1$
 Deformation factor - Table 3.2 $k_{def} = 0.6$
 Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$
 Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$
 System strength factor - cl.6.6 $k_{sys} = 1.1$

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 13.416$ kN
 Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm
 Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 1.032$ N/mm²
 Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 2.048$ N/mm²
 $\sigma_{c,y,90,d} / (K_{c,90} \times f_{c,y,90,d}) = 0.504$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 13.416$ kN
 Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 1.001$ N/mm²
 Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.978$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.336$

PASS - Design shear strength exceeds design shear stress

Check design 900 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 6.037$ kNm
 Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 4.025$ N/mm²
 Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 14.892$ N/mm²
 $\sigma_{m,y,d} / f_{m,y,d} = 0.27$

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PASS - Design bending strength exceeds design bending stress

Check design at end of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 13.416 \text{ kN}$
Effective contact length $L_{b,ef} = \min(L_b, 30 \text{ mm}) + L_b = 130 \text{ mm}$
Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 1.032 \text{ N/mm}^2$
Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 2.048 \text{ N/mm}^2$
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.504$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 13.416 \text{ kN}$
Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 1.001 \text{ N/mm}^2$
Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.978 \text{ N/mm}^2$
 $\tau_{y,d} / f_{v,y,d} = 0.336$

PASS - Design shear strength exceeds design shear stress

Check design 900 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y = 1.6 \text{ mm}$
Quasi-permanent variable load factor $\psi_2 = 0.3$
Final deflection with creep $\delta_{y,Final} = 0.65 \times \delta_y \times (1 + k_{def}) + 0.35 \times \delta_y \times (1 + \psi_2 \times k_{def}) = 2.3 \text{ mm}$
Allowable deflection $\delta_{y,Allowable} = L_{m1_s1} / 250 = 7.2 \text{ mm}$
 $\delta_{y,Final} / \delta_{y,Allowable} = 0.326$

PASS - Allowable deflection exceeds final deflection

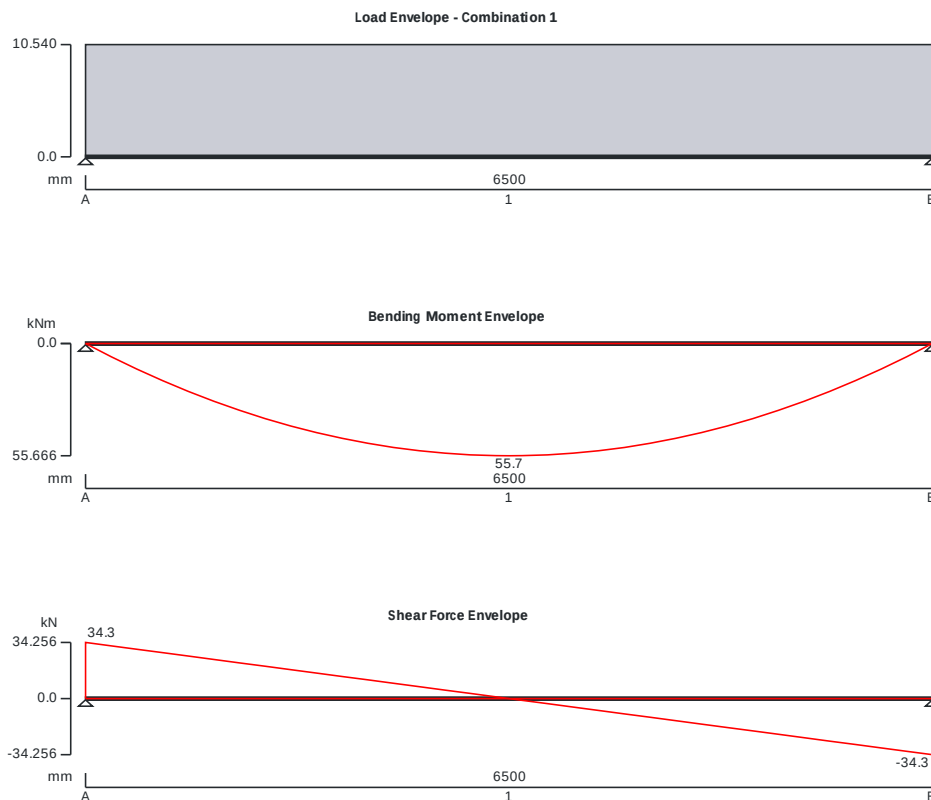


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 3 kN/m

Variable full UDL 4 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

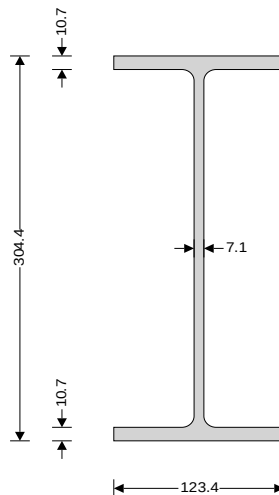
Maximum moment	$M_{max} = 55.7 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum moment span 1 segment 1	$M_{s1_seg1_max} = 55.7 \text{ kNm}$	$M_{s1_seg1_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 2	$M_{s1_seg2_max} = 55.7 \text{ kNm}$	$M_{s1_seg2_min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 34.3 \text{ kN}$	$V_{min} = -34.3 \text{ kN}$
Maximum shear span 1 segment 1	$V_{s1_seg1_max} = 34.3 \text{ kN}$	$V_{s1_seg1_min} = 0 \text{ kN}$
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 0 \text{ kN}$	$V_{s1_seg2_min} = -34.3 \text{ kN}$
Deflection segment 3	$\delta_{max} = 11.4 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 34.3 \text{ kN}$	$R_{A_min} = 34.3 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 10.9 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 13 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 34.3 \text{ kN}$	$R_{B_min} = 34.3 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 10.9 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 13 \text{ kN}$	

Section details

Section type	UB 305x127x37 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_r, t_w) = 10.7 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports plus midspan

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$

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Effective length factor for torsion

$$K_{LT,A} = 1.000$$

$$K_{LT,B} = 1.000$$

Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = 265.2 \text{ mm}$$

$$c / t_w = 40.4 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 49.3 \text{ mm}$$

$$c / t_f = 5.0 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 283 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 34.3 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2344 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 372.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 1 major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_seg1_max}), \text{abs}(M_{s1_seg1_min})) = 55.7 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 148.3 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.873$$

$$C_1 = 1 / k_c^2 = 1.311$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = 0.976$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained length

$$L = 1.0 \times L_{s1_seg1} = 3250 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 176.4 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.917$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$c$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.49$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.942$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.690$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.938$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.736$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 109.1 \text{ kNm}$$



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PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{18.1 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{11.365 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

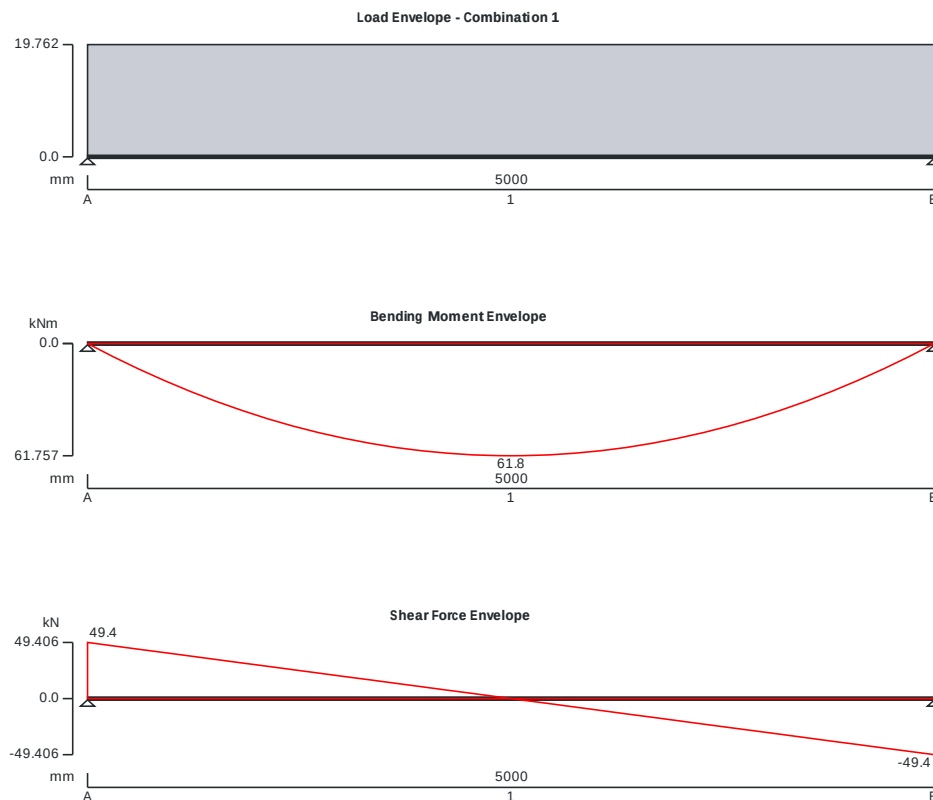


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Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 6 kN/m

Variable full UDL 7.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



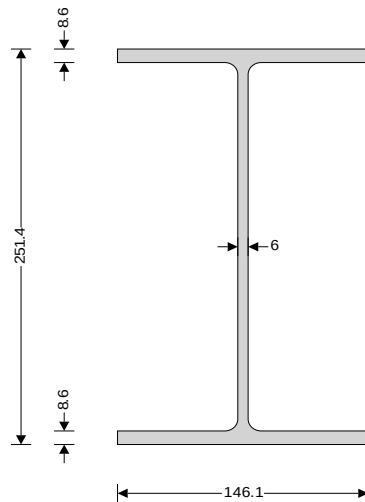
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Analysis results

Maximum moment	$M_{max} = 61.8 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 49.4 \text{ kN}$	$V_{min} = -49.4 \text{ kN}$
Deflection	$\delta_{max} = 12.1 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 49.4 \text{ kN}$	$R_{A_min} = 49.4 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 15.8 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 18.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 49.4 \text{ kN}$	$R_{B_min} = 49.4 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 15.8 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 18.8 \text{ kN}$	

Section details

Section type	UB 254x146x31 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_r, t_w) = 8.6 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = \mathbf{219 \text{ mm}}$$

$$c / t_w = 39.5 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{62.5 \text{ mm}}$$

$$c / t_f = 7.9 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{234.2 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{49.4 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1637 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{259.9 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{61.8 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{108.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.948}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{5000 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{80.9 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{1.156}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.130}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.605}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.978}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.619}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{66.9 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment

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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{13.9 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{12.122 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

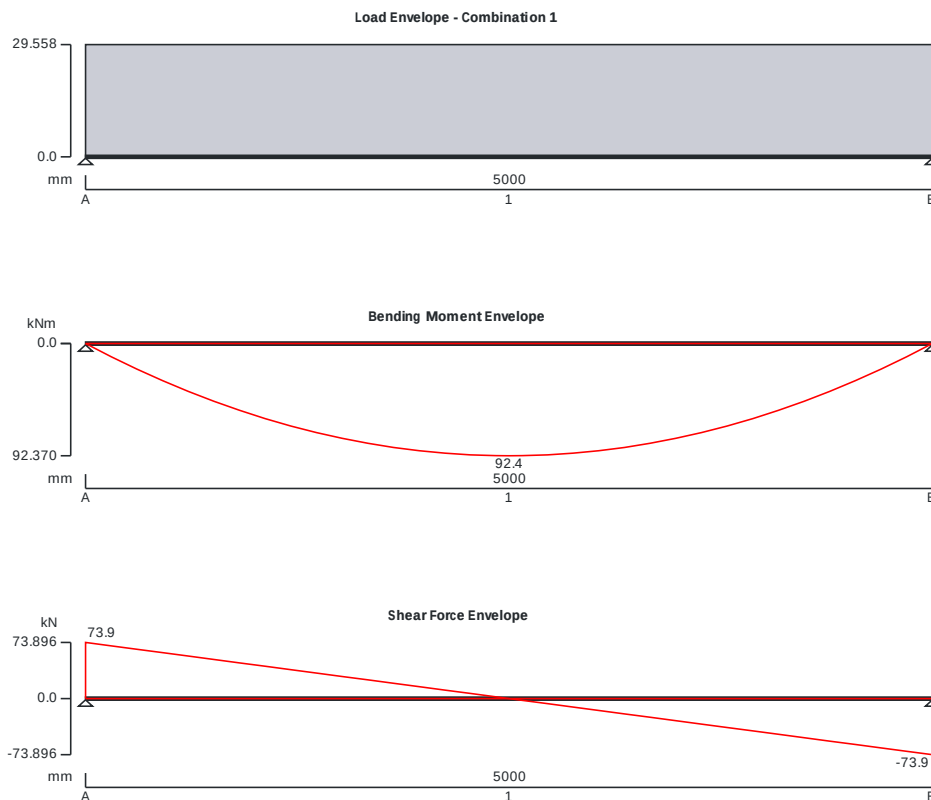


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Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 11.5 kN/m

Permanent full UDL 10 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

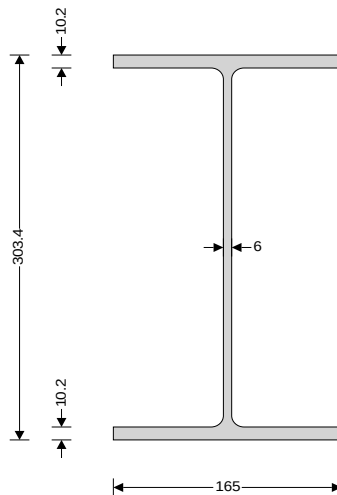
Maximum moment	$M_{max} = 92.4 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 73.9 \text{ kN}$	$V_{min} = -73.9 \text{ kN}$
Deflection	$\delta_{max} = 10 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 73.9 \text{ kN}$	$R_{A_{min}} = 73.9 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 54.7 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 73.9 \text{ kN}$	$R_{B_{min}} = 73.9 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 54.7 \text{ kN}$	

Section details

Section type	UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT.A} = 1.000$
	$K_{LT.B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 265.2 \text{ mm}$
 $c / t_w = 47.8 \times \epsilon \leq 72 \times \epsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 70.6 \text{ mm}$
 $c / t_f = 7.5 \times \epsilon \leq 9 \times \epsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 283 \text{ mm}$
Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \epsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 73.9 \text{ kN}$
Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2009 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 318.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 92.4 \text{ kNm}$
Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 171.3 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$
 $C_1 = 1 / k_c^2 = 1.132$
Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = 0.954$
Poissons ratio $\nu = 0.3$
Shear modulus $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length $L = 1.0 \times L_{s1} = 5000 \text{ mm}$
Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 150.9 \text{ kNm}$
Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.066$
Limiting slenderness ratio $\bar{\lambda}_{LT,0} = 0.4$
 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b
Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$
Correction factor for rolled sections $\beta = 0.75$
LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.039$
LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.659$
Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.974$
Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.677$
Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 116 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads
Limiting deflection $\delta_{lim} = L_{s1} / 360 = 13.9 \text{ mm}$



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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{9.979 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

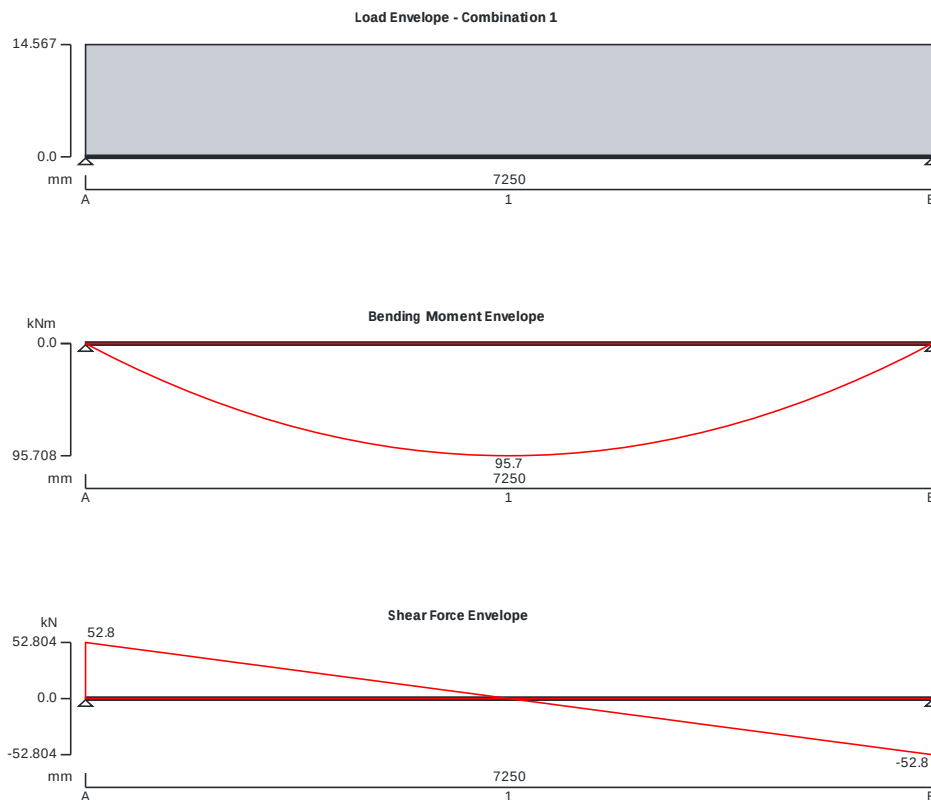


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Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 4.5 kN/m

Permanent full UDL 5.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

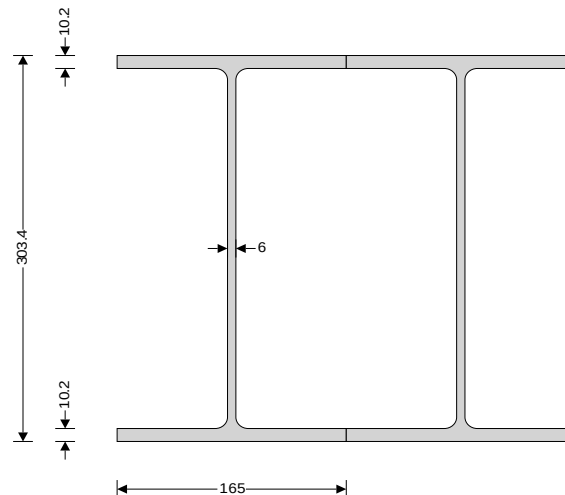
Maximum moment	$M_{max} = 95.7 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 52.8 \text{ kN}$	$V_{min} = -52.8 \text{ kN}$
Deflection	$\delta_{max} = 10.9 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 52.8 \text{ kN}$	$R_{A_{min}} = 52.8 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 39.1 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 52.8 \text{ kN}$	$R_{B_{min}} = 52.8 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 39.1 \text{ kN}$	

Section details

Section type	2 x UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_f, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = 265.2 \text{ mm}$
 $c / t_w = 47.8 \times \varepsilon \leq 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 70.6 \text{ mm}$
 $c / t_f = 7.5 \times \varepsilon \leq 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 283 \text{ mm}$

Shear area factor $\eta = 1.000$
 $h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 52.8 \text{ kN}$

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2009 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 637.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 95.7 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl,y} \times f_y / \gamma_{M0} = 342.7 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$

$C_1 = 1 / k_c^2 = 1.132$

Curvature factor $g = \sqrt{1 - (I_z / I_y)} = 0.954$

Poissons ratio $\nu = 0.3$

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

Unrestrained length $L = 1.0 \times L_{s1} = 7250 \text{ mm}$

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)} = 88.3 \text{ kNm}$

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.393$

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT} = 0.34$

Correction factor for rolled sections $\beta = 0.75$

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.397$

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.476$

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.991$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.480$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl,y} \times f_y / \gamma_{M1} = 164.6 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = 20.1 \text{ mm}$



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Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{10.87 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

Top Of Third Floor

Load Bearing Structural Elements Check

Floor Joists

Max. Length $\leq$ 4700mm

Floor Loads:

Permenant: 1.2 kN/m²

Live: 1.5 kN/m²

Adopt C24 47/275 300mm Spacing

New Beam 1

Loads

Max. Span 6200mm

Max. Loads:

Permenant: 6 kN/m

Live: 7.5 kN/m

Min. Required Section UB 305x165x40 S355

(See Calculations Below)

New Beam 2

Loads

Max. Span 1800mm

Max. Loads:

Permenant: 6 kN/m

Live: 7.5 kN/m

Min. Required Section UB 178x102x19 S355

(See Calculations Below)



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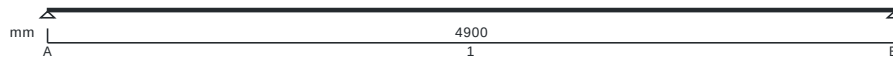
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TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

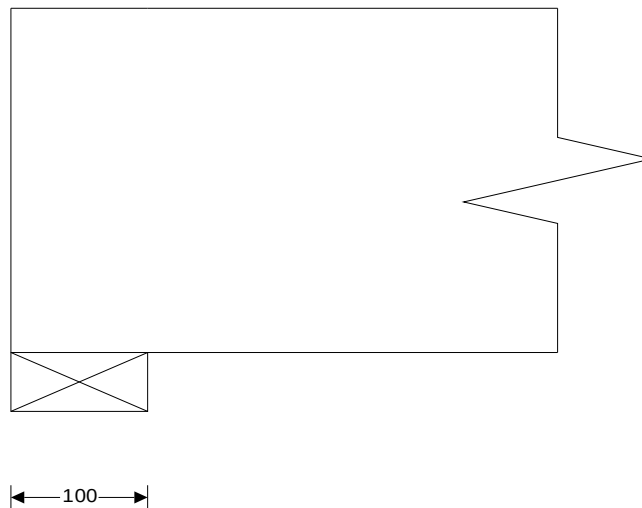
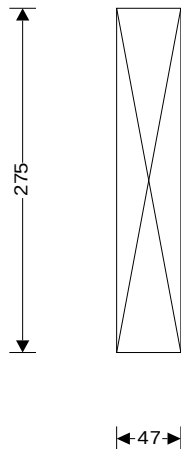
Joist details

Joist breadth $b = 47 \text{ mm}$
 Joist depth $h = 275 \text{ mm}$
 Joist spacing $s = 300 \text{ mm}$
 Timber strength class **C24**
 Service class of timber **1**



Span details

Number of spans $N_{\text{span}} = 1$
 Length of bearing $L_b = 100 \text{ mm}$
 Effective length of span $L_{s1} = 4900 \text{ mm}$



Section properties

Second moment of area $I = b \times h^3 / 12 = 81454427 \text{ mm}^4$
 Section modulus $Z = b \times h^2 / 6 = 592396 \text{ mm}^3$

Loading details

Joist self weight $F_{\text{swt}} = b \times h \times \rho_{\text{char}} \times g_{\text{acc}} = 0.04 \text{ kN/m}$
 Dead load $F_{\text{d_udl}} = 1.20 \text{ kN/m}^2$
 Imposed UDL(Long term) $F_{\text{i_udl}} = 1.50 \text{ kN/m}^2$
 Imposed point load (Medium term) $F_{\text{i_pt}} = 1.40 \text{ kN}$

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Modification factors

Service class for bending parallel to grain	$K_{2m} = 1.00$
Service class for compression	$K_{2c} = 1.00$
Service class for shear parallel to grain	$K_{2s} = 1.00$
Service class for modulus of elasticity	$K_{2e} = 1.00$
Section depth factor	$K_7 = 1.01$
Load sharing factor	$K_8 = 1.10$

Consider long term loads

Load duration factor	$K_3 = 1.00$
Maximum bending moment	$M = 2.564 \text{ kNm}$
Maximum shear force	$V = 2.093 \text{ kN}$
Maximum support reaction	$R = 2.093 \text{ kN}$
Maximum deflection	$\delta = 7.643 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 7.500 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 8.329 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 4.328 \text{ N/mm}^2$
PASS - Applied bending stress within permissible limits	

Check shear stress

Shear stress	$\tau = 0.710 \text{ N/mm}^2$
Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.781 \text{ N/mm}^2$
Applied shear stress	$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.243 \text{ N/mm}^2$
PASS - Applied shear stress within permissible limits	

Check bearing stress

Compression perpendicular to grain (no wane)	$\sigma_{cp1} = 2.400 \text{ N/mm}^2$
Permissible bearing stress	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.640 \text{ N/mm}^2$
Applied bearing stress	$\sigma_{c_max} = R / (b \times L_b) = 0.445 \text{ N/mm}^2$
PASS - Applied bearing stress within permissible limits	

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$
Bending deflection (based on E_{mean})	$\delta_{bending} = 7.290 \text{ mm}$
Shear deflection	$\delta_{shear} = 0.353 \text{ mm}$
Total deflection	$\delta = \delta_{bending} + \delta_{shear} = 7.643 \text{ mm}$
PASS - Actual deflection within permissible limits	

Consider medium term loads

Load duration factor	$K_3 = 1.25$
Maximum bending moment	$M = 2.929 \text{ kNm}$
Maximum shear force	$V = 2.391 \text{ kN}$
Maximum support reaction	$R = 2.391 \text{ kN}$
Maximum deflection	$\delta = 7.754 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 7.500 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 10.412 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 4.944 \text{ N/mm}^2$



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PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress

$$\tau = 0.710 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976 \text{ N/mm}^2$$

Applied shear stress

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.277 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = 2.400 \text{ N/mm}^2$$

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.300 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = 0.509 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = 7.351 \text{ mm}$$

Shear deflection

$$\delta_{shear} = 0.403 \text{ mm}$$

Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = 7.754 \text{ mm}$$

PASS - Actual deflection within permissible limits

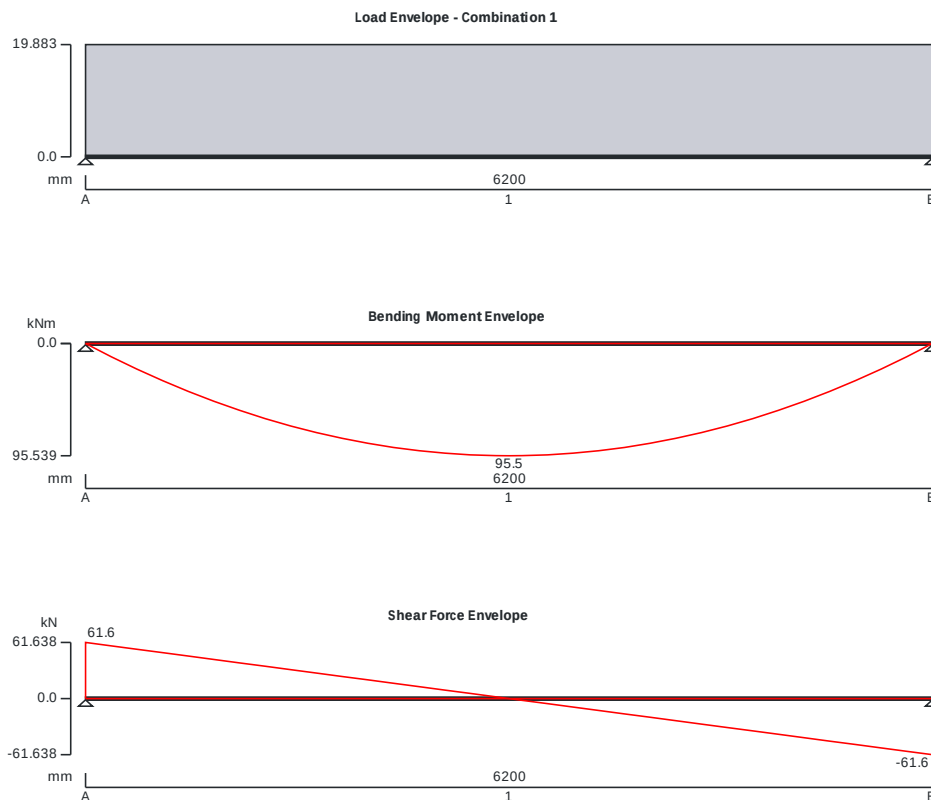


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 6 kN/m

Variable full UDL 7.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

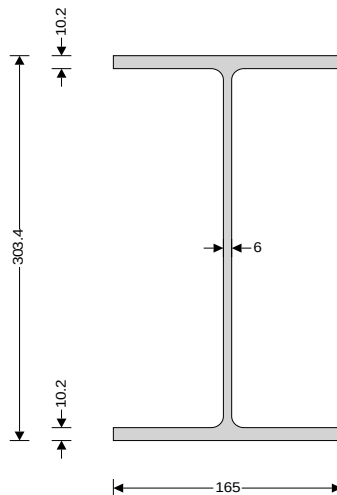
Maximum moment	$M_{max} = 95.5 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 61.6 \text{ kN}$	$V_{min} = -61.6 \text{ kN}$
Deflection	$\delta_{max} = 15 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 61.6 \text{ kN}$	$R_{A_{min}} = 61.6 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 19.8 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{Variable}} = 23.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 61.6 \text{ kN}$	$R_{B_{min}} = 61.6 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 19.8 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{Variable}} = 23.2 \text{ kN}$	

Section details

Section type	UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_r, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT.A} = 1.000$
	$K_{LT.B} = 1.000$

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Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = \mathbf{265.2 \text{ mm}}$$

$$c / t_w = 47.8 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{70.6 \text{ mm}}$$

$$c / t_f = 7.5 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{283 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{61.6 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2009 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{318.9 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{95.5 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{171.3 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.954}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{6200 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{109.7 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{1.25}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.230}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.551}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.982}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.561}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{96.1 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment



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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 17.2 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 14.973 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

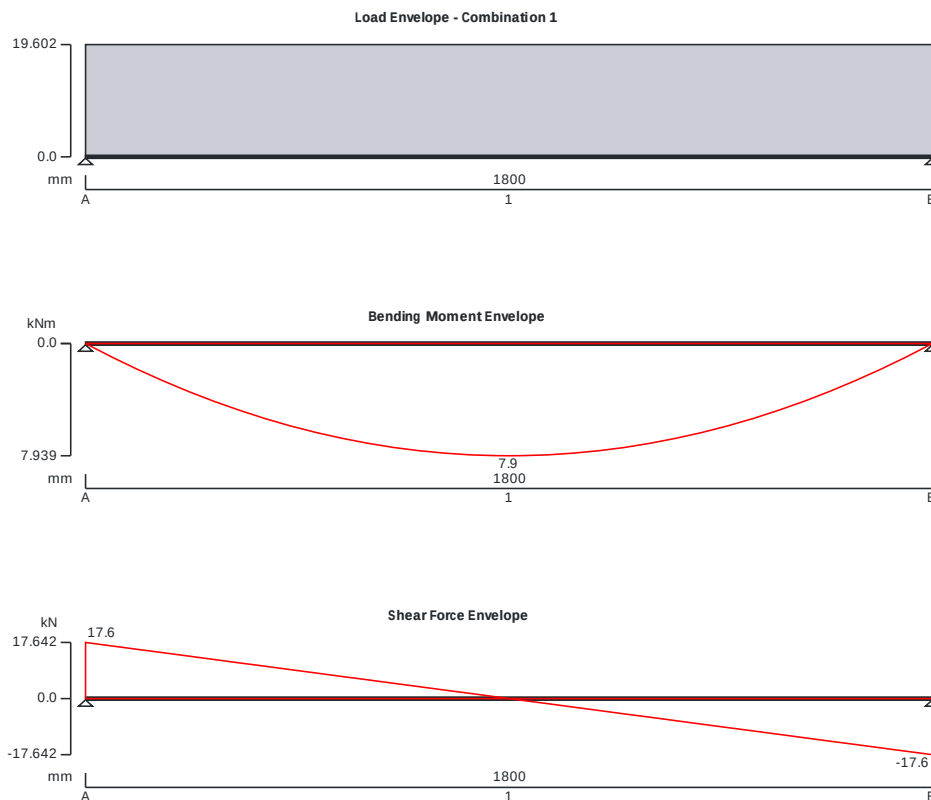


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 6 kN/m

Variable full UDL 7.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

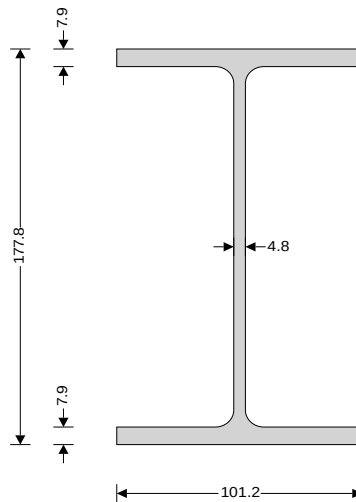
Maximum moment	$M_{max} = 7.9 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 17.6 \text{ kN}$	$V_{min} = -17.6 \text{ kN}$
Deflection	$\delta_{max} = 0.7 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 17.6 \text{ kN}$	$R_{A_min} = 17.6 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 5.6 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 6.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 17.6 \text{ kN}$	$R_{B_min} = 17.6 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 5.6 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 6.8 \text{ kN}$	

Section details

Section type	UB 178x102x19 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_r, t_w) = 7.9 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = \mathbf{146.8 \text{ mm}}$$

$$c / t_w = 33.1 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{40.6 \text{ mm}}$$

$$c / t_f = 5.6 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{162 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{17.6 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{985 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{156.4 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{7.9 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{47.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.948}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{1800 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{110.9 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{0.652}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.702}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.893}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.971}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.920}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{43.3 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment



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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 5 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 0.657 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

Roof Structure

Ridge Beam

Span 4800mm

Loads:

LOADING			
☒ Self Weight			
TITLE Pitched Roof			
	DEAD kN / m	9.68	LIVE kN / m
		5.25	

ADOPT UB 305x165x40 S355

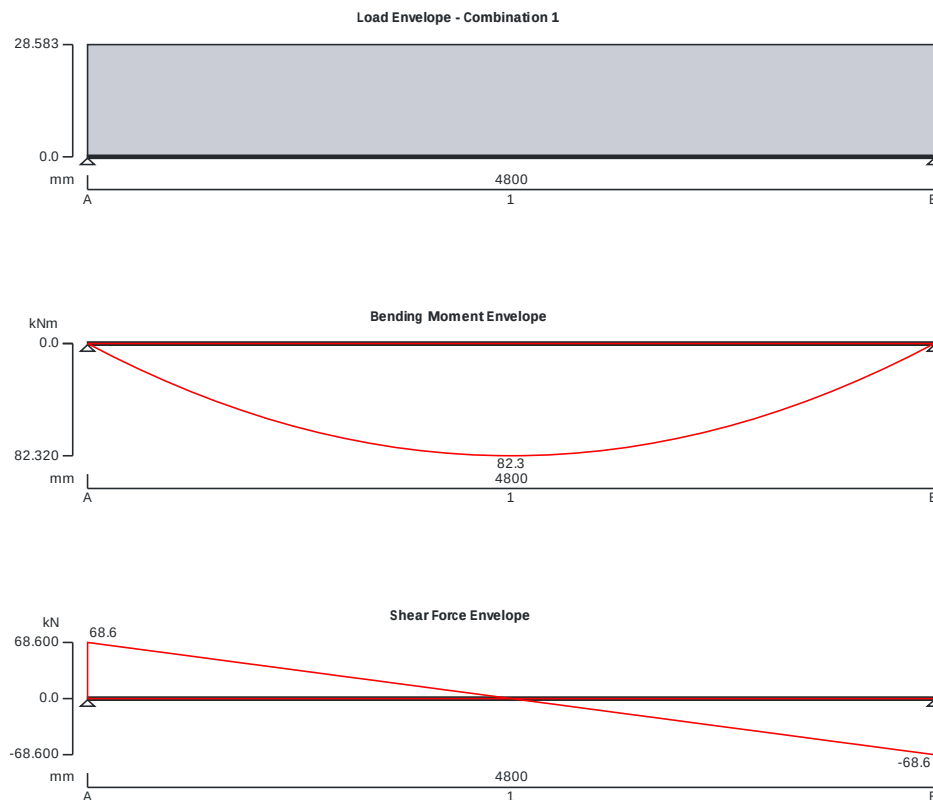


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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 13 kN/m

Variable full UDL 7 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Analysis results

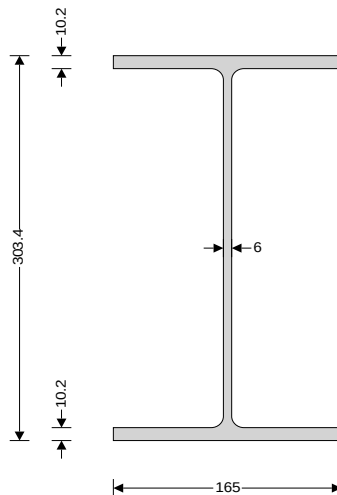
Maximum moment	$M_{\max} = 82.3 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 68.6 \text{ kN}$	$V_{\min} = -68.6 \text{ kN}$
Deflection	$\delta_{\max} = 7.9 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 68.6 \text{ kN}$	$R_{A_{\min}} = 68.6 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 32.1 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 16.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 68.6 \text{ kN}$	$R_{B_{\min}} = 68.6 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 32.1 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 16.8 \text{ kN}$	

Section details

Section type	UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275

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

Nominal thickness of element	$t = \max(t_r, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = \mathbf{265.2 \text{ mm}}$$

$$c / t_w = 47.8 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{70.6 \text{ mm}}$$

$$c / t_f = 7.5 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{283 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{68.6 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2009 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{318.9 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{82.3 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{171.3 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.954}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{4800 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{160.7 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{1.033}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.007}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.680}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.973}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.698}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{119.7 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment



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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{13.3 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{7.895 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

**TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)****In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex**

Tedds calculation version 1.0.09

Rafter details

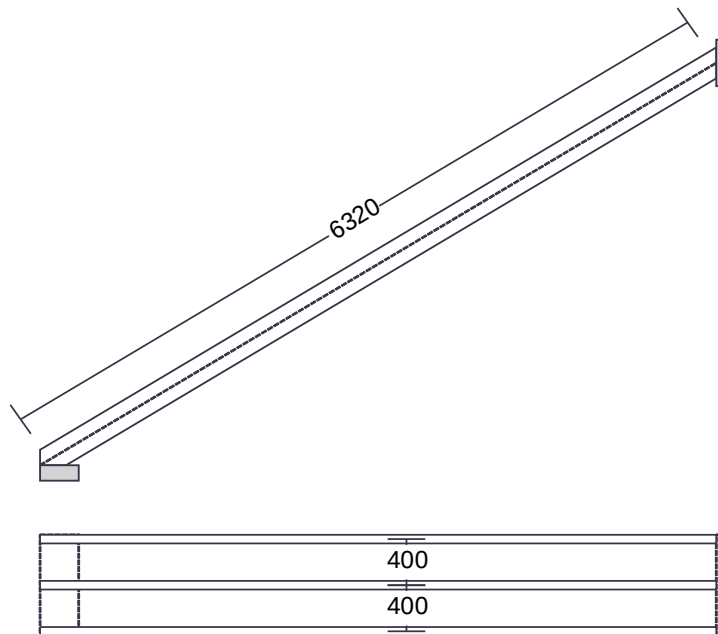
Description

75 x 225 C24 timber rafters

Rafter spacing

 $S_{\text{Rafter}} = 400 \text{ mm}$

Rafter inclination

 $\theta_{\text{Rafter}} = 33 \text{ deg}$ **Forces input on Rafter**

Permanent load on slope

 $F_{G_Rafter} = 0.75 \text{ kN/m}^2$

Imposed load on plan

 $F_{Q_Rafter} = 0.30 \text{ kN/m}^2$ **Rafter loading details****Distributed loads**

Permanent load on slope

 $p_G = F_{G_Rafter} \times S_{\text{Rafter}} = 0.30 \text{ kN/m}$

Imposed load on slope

 $p_Q = F_{Q_Rafter} \times S_{\text{Rafter}} \times \cos(\theta_{\text{Rafter}}) = 0.10 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	14.2	0.1	0.009	PASS
Bearing stress	N/mm ²	1.7	0.3	0.166	PASS
Bending stress	N/mm ²	16.2	4.3	0.265	PASS
Shear stress	N/mm ²	2.7	0.2	0.084	PASS
Bending and axial force				0.265	PASS
Column stability check				0.289	PASS
Beam stability check				0.079	PASS
Deflection	mm	31.6	26.5	0.839	PASS



ANALYSIS

Tedds calculation version 1.0.38

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

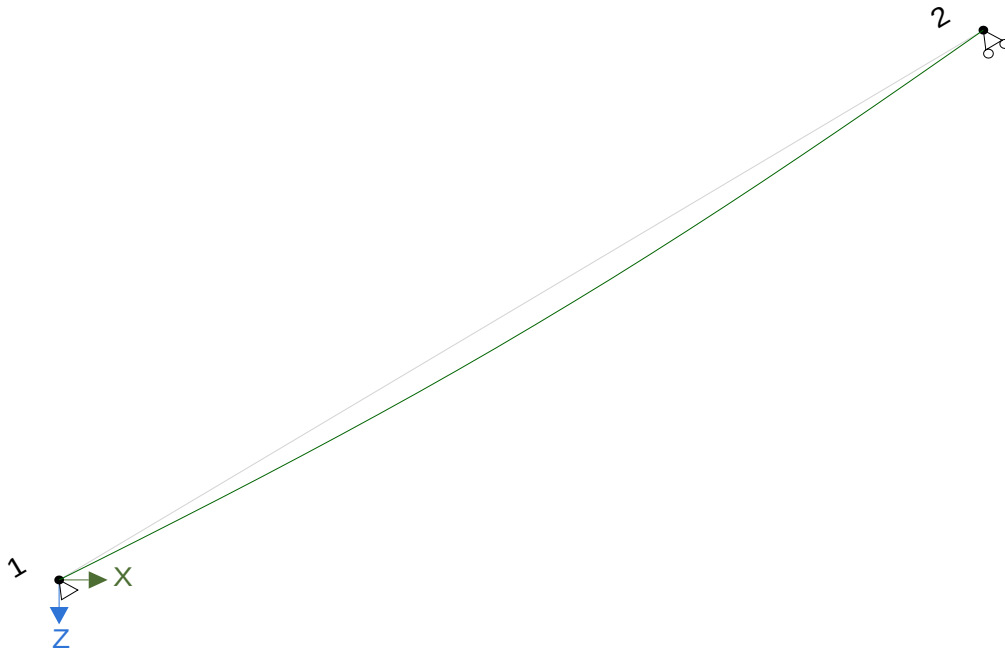
Member Loads

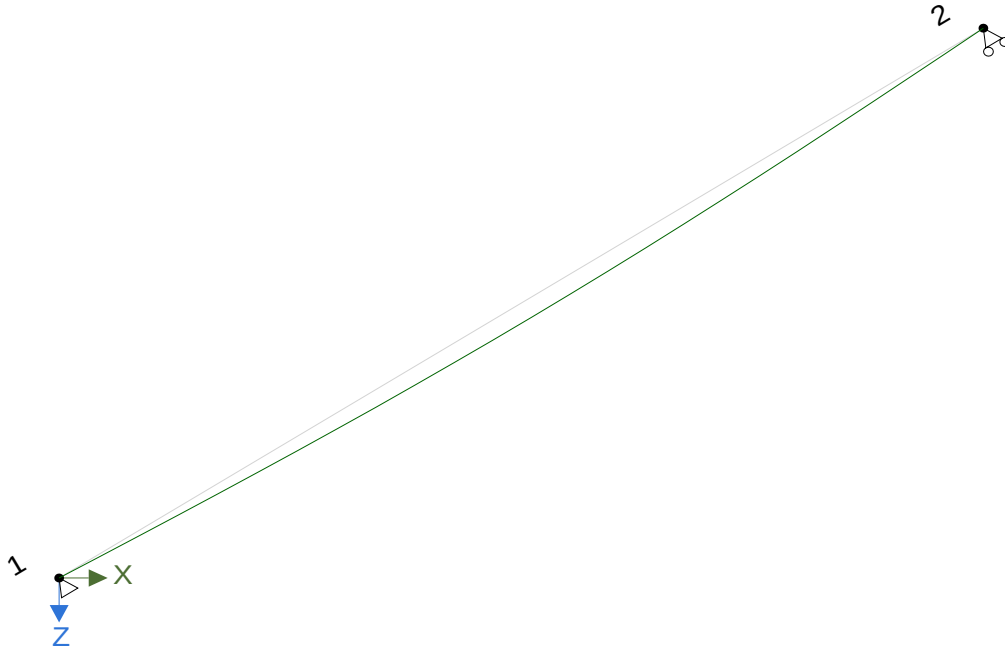
Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.3 kN/m at 0 m to 6.32 m
Member	Imposed	UDL	GlobalZ	0.1 kN/m at 0 m to 6.32 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



**1.00G + 1.00Q (Service) - Total deflection****Node deflections****Load combination: 1.35G + 1.50Q (Strength)**

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.41938	Member
2	0	0	-0.41938	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.30344	Member
2	0	0	-0.30344	Member

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	4.1
1.00G + 1.00Q (Service)	0	3



Project

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Element end forces

Load combination: 1.35G + 1.50Q (Strength)

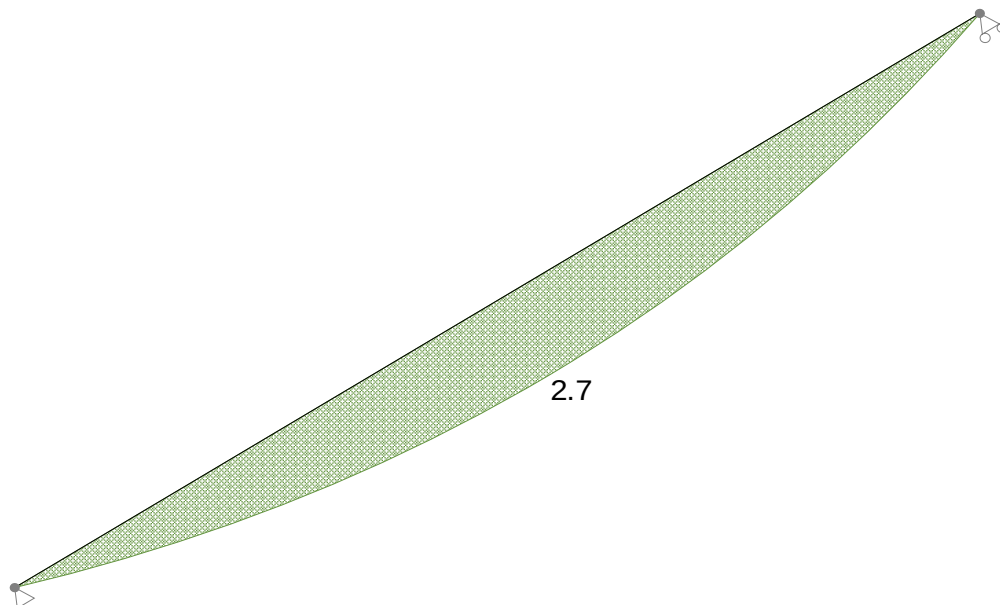
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	6.32	1	-2.2	-1.7	0
		2	0	-1.7	0

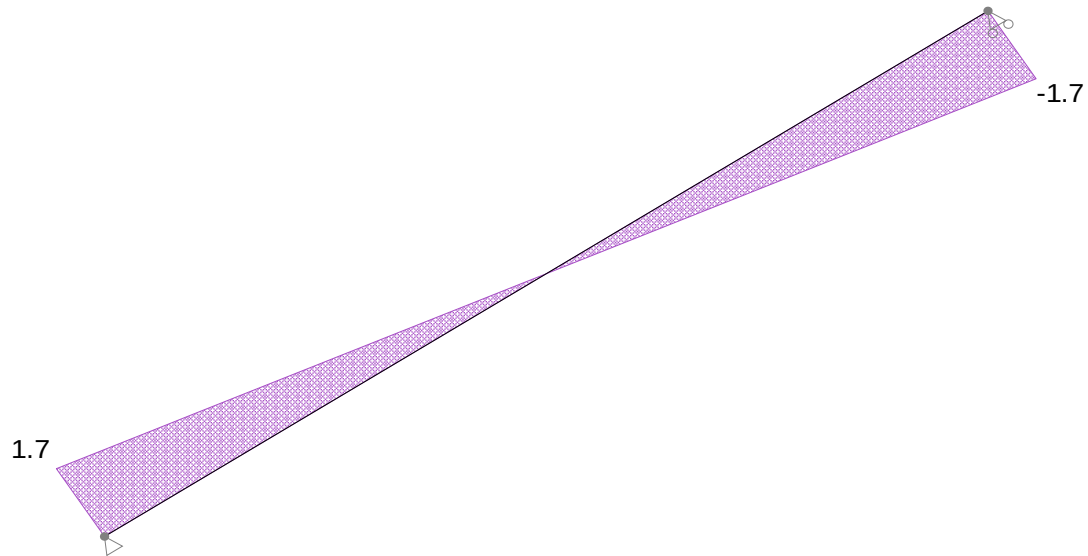
Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	6.32	1	-1.6	-1.2	0
		2	0	-1.2	0

Forces

Strength combinations - Moment envelope (kNm)



**Strength combinations - Shear envelope (kN)****Member results****Envelope - Strength combinations**

Member	Position (m)	Shear force (kN)	Moment (kNm)
Member	0	1.7	0 (min)
	3.16	0	2.7 (max)
	6.32	-1.7 (max abs)	0 (min)

Tedds calculation version 2.2.23

Member - Span 1**Partial factor for material properties and resistances**Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$ **Member details**

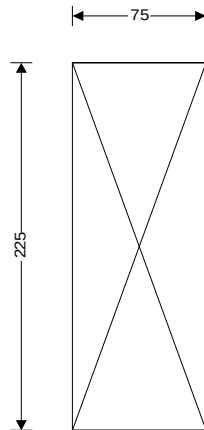
Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 2

Timber section detailsNumber of timber sections in member $N = 1$ Breadth of sections $b = 75$ mmDepth of sections $h = 225$ mmTimber strength class - EN 338:2016 Table 1 **C24**



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75x225 timber section

Cross-sectional area, A , 16875 mm²
 Section modulus, W_y , 632812.5 mm³
 Section modulus, W_z , 210937 mm³
 Second moment of area, I_y , 71191406 mm⁴
 Second moment of area, I_z , 7910156 mm⁴
 Radius of gyration, i_y , 65 mm
 Radius of gyration, i_z , 21.7 mm
Timber strength class C24
 Characteristic bending strength, $f_{m,k}$, 24 N/mm²
 Characteristic shear strength, $f_{v,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²
 Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm²
 Shear modulus of elasticity, G_{mean} , 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length $L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.8$
 Deformation factor - Table 3.2 $k_{def} = 0.8$
 Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$
 Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$
 System strength factor - cl.6.6 $k_{sys} = 1.1$

Check compression parallel to the grain - cl.6.1.4

Design axial compression $P_d = 2.237$ kN
 Design compressive stress $\sigma_{c,0,d} = P_d / A = 0.133$ N/mm²
 Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = 14.215$ N/mm²
 $\sigma_{c,0,d} / f_{c,0,d} = 0.009$

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 2.744$ kN
 Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm
 Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.281$ N/mm²
 Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.692$ N/mm²
 $\sigma_{c,y,90,d} / (K_{c,90} \times f_{c,y,90,d}) = 0.166$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.722$ kN
 Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.228$ N/mm²
 Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.708$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.084$

PASS - Design shear strength exceeds design shear stress

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Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending	$L_{e,y} = 0.9 \times 6320 \text{ mm} = 5688 \text{ mm}$
Slenderness ratio	$\lambda_y = L_{e,y} / i_y = 87.572$
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 1.485$
Effective length for z-axis bending	$L_{e,z} = 0 \text{ mm}$
Slenderness ratio	$\lambda_z = L_{e,z} / i_z = 0$
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$
	$\lambda_{rel,y} > 0.3$ column stability check is required
Straightness factor	$\beta_c = 0.2$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.721$
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.386$
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.024$
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.009$
	PASS - Column stability is acceptable

Check design 3160 mm along span

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d} = 2.721 \text{ kNm}$
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y = 4.3 \text{ N/mm}^2$
Design bending strength	$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 16.246 \text{ N/mm}^2$
	$\sigma_{m,y,d} / f_{m,y,d} = 0.265$
	PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20	$(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} = 0.265$
	$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.185$
	PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending	$L_{e,y} = 0.9 \times 6320 \text{ mm} = 5688 \text{ mm}$
Slenderness ratio	$\lambda_y = L_{e,y} / i_y = 87.572$
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 1.485$
Effective length for z-axis bending	$L_{e,z} = 0 \text{ mm}$
Slenderness ratio	$\lambda_z = L_{e,z} / i_z = 0$
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$
	$\lambda_{rel,y} > 0.3$ column stability check is required
Straightness factor	$\beta_c = 0.2$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.721$
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.386$
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = 0.289$
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.194$
	PASS - Column stability is acceptable



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Project 31-34 Commercial St, Newport NP20 1HJ				Job no.	
Calcs for RAFTERS				Start page no./Revision 8	
Calcs by TL	Calcs date 24/02/2025	Checked by	Checked date	Approved by	Approved date

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34

$$k_{crit} = 1.000$$

Beam stability check - exp.6.35

$$(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.079$$

PASS - Beam stability is acceptable

Check design 3160 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 14.7 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{det}) = 26.5 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1_s1} / 200 = 31.6 \text{ mm}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = 0.839$$

PASS - Allowable deflection exceeds final deflection