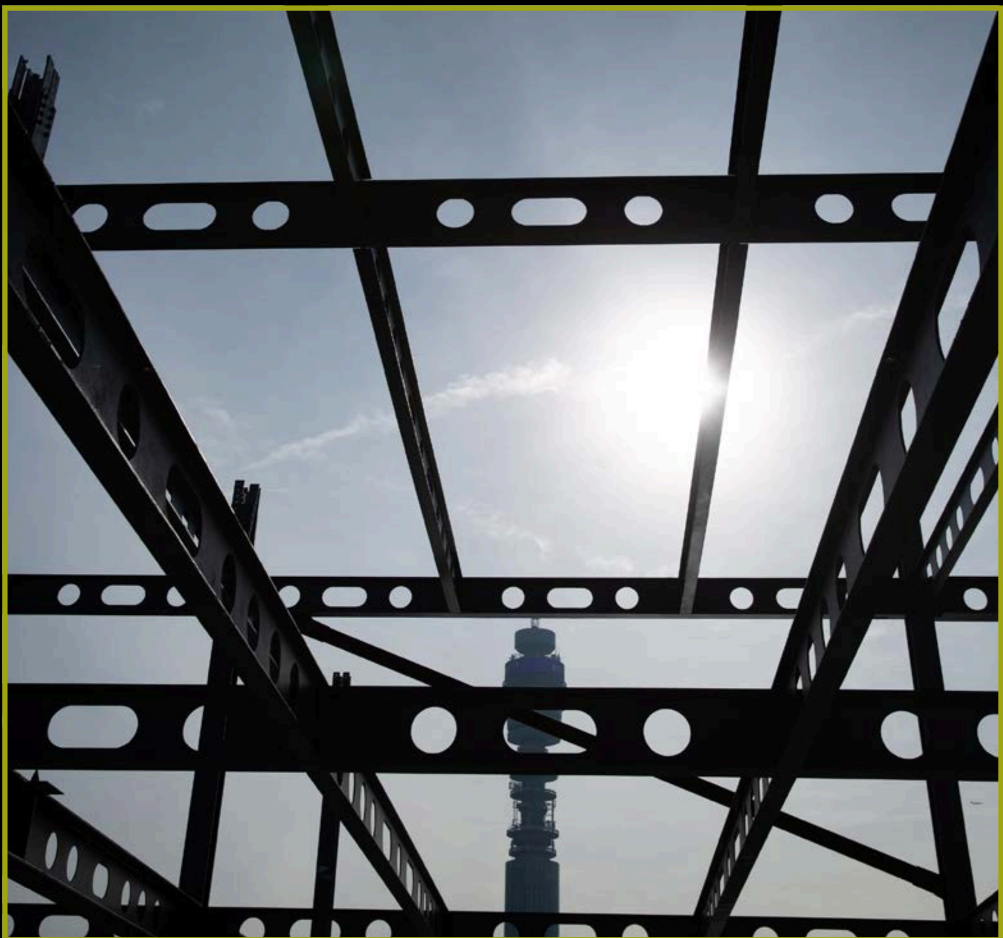
BEST PRACTICE FOR DESIGNING LOW EMBODIED CARBON STEEL BUILDINGS



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BEST PRACTICE FOR DESIGNING LOW EMBODIED CARBON STEEL BUILDINGS

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SUMMARY

This publication recommends ways in which structural engineers should strive to reduce embodied carbon in their steelwork designs. The recommendations are presented in the context of multi-storey buildings but are equally applicable to all structural types.

Detailed advice is given in subsequent sections of this publication. The following summary presents the designer's responsibility to prepare a design which is as efficient as possible.

- Challenge the brief – adapt, extend, refurbish instead of new construction.
- Design clever.
  - Don't overdesign.
  - Don't allow for undefined future use – it may never happen.
  - Use the codified imposed loading.
  - Have utilisation factors for resistance as close to 1.0 as possible.
  - Don't specify unnecessarily restrictive deflection or vibration criteria.
  - Do use the codified reduction factors for area and numbers of storeys.
  - Don't use typical composite arrangements, when shallower slabs or longer spans would be adequate.
- Engage early with steelwork contractors to develop carbon-efficient solutions.
- Do use material efficiently.
  - Consider S460 for column sections.
  - Consider cellular beams, fabricated profiles and asymmetric profiles.
  - Utilise the benefits of semi-continuity.
  - Use recovered steelwork.
- Consider alternative floor slab solutions, such as mass timber.
- Utilise the benefits of lightweight long spans.
- Provide sufficient information for others to specify effective fire protection.
- Recognise that specifying electric arc furnace (EAF) steel exclusively will reduce the embodied carbon of an individual project, but not impact the global problem – good design to reduce the weight of steel is a better approach.

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INTRODUCTION

Of the many sustainability issues relating to buildings and the construction industry, the climate emergency and action to reduce carbon emissions has become a priority for construction clients alongside cost.

Decarbonising the structural steelwork supply chain requires action by all parties; structural engineers/designers, steel producers, stockholders and steelwork contractors and, at the end-of-life of buildings, demolition contractors, scrap processors and refurbishment and reuse experts. Reflecting this, the British Constructional Steelwork Association (BCSA) decarbonisation roadmap (see Section 1.2) sets out how the sector can decarbonise by 2050.

Stopping or reducing the construction of new buildings is one way to reduce embodied carbon however, to ensure the UK remains globally competitive, to upgrade inefficient building stock and to provide a future low-carbon infrastructure, new buildings and infrastructure will be needed.

In addition, extending buildings vertically and horizontally, refurbishment of the existing building stock and reusing reclaimed construction elements will have a key role to play in reducing embodied carbon.

While new, low-carbon technologies are developed and commercialised for mainstream construction materials like steel and cement, attention should also focus on demand-side reduction measures. This is where design teams can employ their expertise and experience.

This publication provides guidance on sustainable structural steelwork design for buildings. The focus is on embodied carbon emissions and what structural engineers can do to reduce carbon emissions, but the wider context is also covered in terms of broader sustainability considerations.

This guide is for structural engineers to help them design steel structures more efficiently to reduce demand for steel without compromising safety and creativity and, by doing so, reduce embodied carbon emissions. The guidance will help to deliver Lever 1 in the BCSA 2050 decarbonisation roadmap – see Section 1.2.

The guidance is structured around the Net Zero Design hierarchy developed by the IStructE[1] which is based on PAS 2080[2] principles, as shown in Figure 1.1.

The focus of the guidance is structural steelwork used as the primary structure in multi-storey buildings. It includes steelwork fabricated from open and closed steel sections and sections fabricated from steel plate and, where relevant, steel decking, concrete and mass timber.

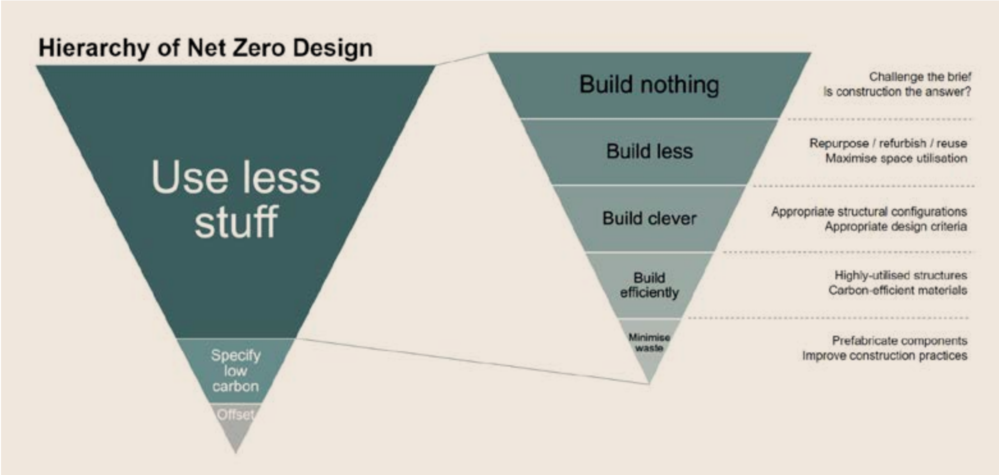


Figure 1.1 Hierarchy of design decisions

1.1 Sustainability and steel construction

First introduced by the United Nations in 1987, the term sustainability or sustainable development has a very broad definition that simultaneously addresses the environmental, social and economic aspects of development. The environmental dimension of sustainability has generally taken precedence and, more recently, the urgency of climate change and the need to reduce greenhouse gas (GHG) emissions, has meant that 'low carbon' has become the priority and, for many, is today synonymous with the term sustainability.

While climate change and reducing GHG are clearly priorities and the focus of this guidance, the benefits of steel, structural steelwork and steel-framed buildings within a broader sustainable context should not be forgotten. These include:

- Steel structures are lightweight and structurally efficient;
- Steelwork is efficiently fabricated offsite offering quality assured, fully tested and traceable products;
- Steelwork design and fabrication is BIM-led providing a digital-twin, enabling future reuse;
- On-site construction is safe and fast with minimal local adverse environmental impacts;
- Structural steel is fully recyclable and many structural elements are reusable;
- Steel-framed buildings are flexible to change of use and steel structures are easily adapted, potentially extending the life of the building.

Designers should strive to deliver lean and carbon-efficient designs of steel buildings that offer long-term sustainability benefits. As construction materials are decarbonised, the focus should turn to conserving finite resources and delivering adaptable, long-life and truly circular buildings.

Steel in general, and structural steel specifically, is truly compatible with circular economic models. To drive optimum resource decision-making, it is important that the whole-life impacts and the whole-life benefits of our buildings and of the components used to construct them are properly considered.

1.2 Steel decarbonisation roadmap

In response to the climate emergency and particularly the international 2015 Paris Agreement and the national 2050 UK net zero targets and commitments, several steel decarbonisation strategies and roadmaps have been published.

BCSA published its 2050 decarbonisation roadmap in 2021[3]. The scope of this roadmap is the UK structural steelwork sector and it sets out how the sector can decarbonise to meet the UK net-zero carbon target by 2050.

The BCSA roadmap is based on six decarbonisation strategies or 'levers' that the sector is concurrently developing and deploying. The roadmap is graphically illustrated in Figure 1.2.

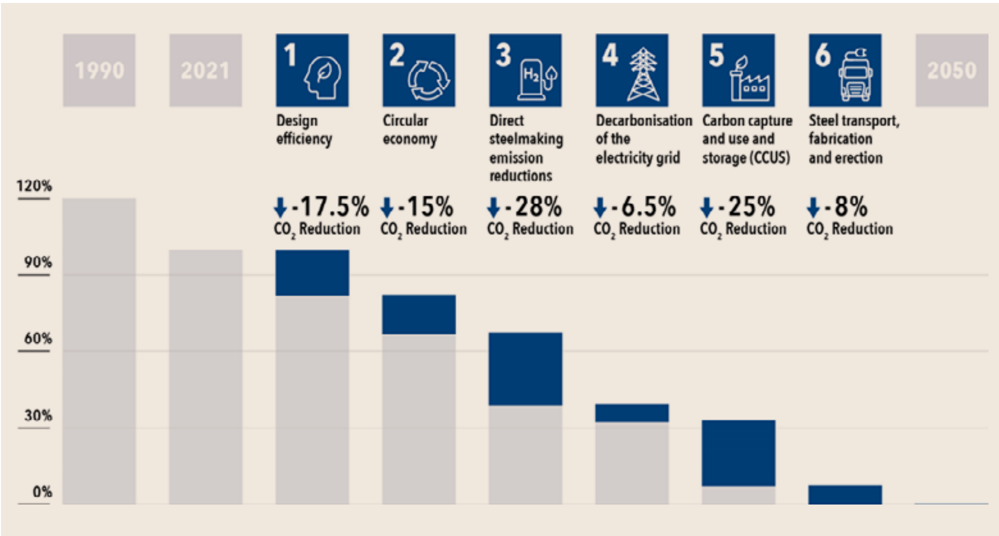


Figure 1.2 BCSA 2050 decarbonisation roadmap

Although the largest contributions to decarbonisation (levers 3 and 5) are the responsibility of the steelmakers, the opportunities to reduce embodied carbon through design (lever 1) are significant and readily achievable today.

The major steelmakers who supply the UK structural steelwork sector have published their own decarbonisation roadmaps and committed to decarbonise in line with the 2050 UK and other national and EU targets including adopting Science Based Targets. In addition, many steelmakers have achieved or are in the process of obtaining, certification to the ResponsibleSteel standard[4].

1.2.1 Whole life carbon

It is important to understand the definitions of embodied, operational and whole life carbon as they relate to buildings. The modular approach adopted by CEN TC350 (the European committee responsible for assessment of the sustainability aspects of construction works), to assess the sustainability impacts of construction, is useful to define, understand and report impacts of buildings in a consistent and transparent way.

The following definitions are based on BS EN 15804[5] and BS EN 15978[6], the European standards used to define the rules for producing environmental product declarations (EPD, see Section 1.4) for construction products and for assessing the environmental performance of buildings, respectively.

The **embodied carbon** emissions of a building are the total GHG emissions associated with materials and construction processes throughout the whole life cycle of a built asset (Modules A0-A5, B1-B5, C1-C4). In the context of structural steel, this includes:

- Mining iron ore and coal and the preparation of scrap (A1)
- Transport (A1)
- Steel production (A1)
- Transport to the steelwork contractor (A2)
- Fabrication of structural elements (A3)
- Transport to the construction site (A4)
- Erection on site including site clearance (A5)
- Maintenance and repair (B2, B3)
- Replacing components (B4)
- Refurbishment (B5)
- End-of-life impacts of deconstruction and preparation for reuse or recycling (C).

In addition, BS EN 15978-1[7] includes a pre-construction module (A0) covering non-physical activities, for example, design, preliminary studies, etc.

It should be noted that definitions of Modules A1-A3 within steel EPD, can be confusing. Steel production EPD generally define Module A1 as raw material extraction and processing and steelmaking and Module A3 as hot roll-forming of the steel to produce sections. However, this leaves no module for the fabrication stage. Rather than aggregate fabrication and roll-forming processes together in Module A3, in this guide Module A1 is taken to include all steelmaking processes including roll-forming, A2 is the transport from the steel mill to the fabrication facility and A3 is the fabrication impacts.

Upfront embodied carbon emissions are GHG emissions associated with materials and construction processes up to practical completion (modules A0–A5). Upfront carbon excludes the biogenic carbon sequestered in the installed products at practical completion.

Operational Carbon emissions (Module B6) are the GHG emissions arising from all energy consumed by building asset in-use. This includes carbon emissions associated with heating, hot water, cooling, ventilation and lighting systems, as well as those associated with cooking, equipment, and lifts, etc. i.e. both regulated and unregulated energy uses as defined in the UK Building Regulations.

Whole Life Carbon emissions are the sum-total of all asset-related GHG emissions, both operational and embodied over the life cycle of an asset including its disposal.

Overall Whole Life Carbon asset performance includes separately reporting the potential benefit from future energy recovery, reuse, and recycling (Module D). Reporting of Module D is mandatory under BS EN 15804[5] and BS EN 15978-1 and is recommended best practice in guidance published by authorities such as the Royal Institute of Chartered Surveyors, The Mayor of London and the Institution of Structural Engineers. Embodied carbon is currently (2024) not regulated in the UK, although there are calls for this to happen. Appendix A provides more information on Module D.

Despite the focus on reducing operational carbon over recent years, UK Green Building Council (UKGBC) data[9] shows that operational carbon emissions from UK buildings still exceed embodied carbon emissions by a factor of around three and consequently operational carbon reductions remains the priority for new and existing buildings.

1.3 Measuring embodied carbon

Embodied carbon assessment is a subset of a broader discipline called Life Cycle Assessment (LCA) which covers the quantification of a range of different environmental impacts. As such, many of the principles and standards applicable to LCA are also applicable to embodied carbon assessments. Within an LCA study, embodied carbon is one of the core environmental indicators and is called global warming potential or GWP and is expressed in $kgCO_2e$. The lower case ‘e’ stands for ‘equivalent’, so that the GWP of different greenhouse gases can be aggregated into a single metric, i.e. carbon dioxide.

LCA is the methodology that is used to develop Environmental Product Declarations (EPD) which are a standardised set of environmental information discussed in Section 1.4.

Construction LCA and embodied carbon assessments are conducted by following standards. The most relevant in the UK are the EN standards:

- At the product level – BS EN 15804 which gives core rules for producing EPDs for construction products
- At the building level – BS EN 15978 which provides the calculation method for assessing the environmental performance of buildings.

Detailed guidance on the calculation of embodied carbon of structures has been published by the Institution of Structural Engineers[10],[11]. Guidance on whole life carbon assessment of buildings has been published by the Royal Institution of Chartered Surveyors (RICS)[12].

1.4 Environmental product declarations

Environmental Product Declarations (EPD), are used to provide environmental information derived from life cycle assessment (LCA) studies in a common format, based on a common set of rules, known as Product Category Rules (PCR). The construction industry has widely adopted EPD as the means of reporting and communicating environmental information on construction products.

EPD can be used externally for marketing materials and internally for the improvement of product manufacture, or process efficiency. They are also used within whole building assessment schemes, other comparative assessment tools (particularly embodied carbon assessment tools) and building information modelling (BIM) CAD software.

BS EN ISO 14025[13] sets out principles and procedures for developing Type III environmental declarations, such as EPD. This standard also draws on the key LCA standards BS EN ISO 14040[14] and BS EN ISO 14044[15].

For construction in Europe, BS EN 15804 is the key standard which provides the core product category rules for producing EPD of construction products.

To be comparable, EPD must have been developed using the same PCR, to ensure the scope, methodology, data quality and impact indicators are the same. EPD can only be compared when the same PCR have been used and all the relevant life cycle stages or modules have been included.

In addition, so-called complementary product category rules (c-PCR) have been developed for some product groups. These provide additional, product-specific information for developing EPD. The c-PCR standard for steel and aluminium structural products is prEN 17662[8] developed by CEN Technical Committee TC 135.

Before publication, an EPD needs to be verified by an independent third party reviewer. This ensures accuracy, reliability and ensures that the EPD conforms to the requirements of the relevant PCR.

EPD provide quantification of a range of environmental impacts including ozone depletion, acidification, eutrophication and abiotic depletion in addition to the most widely used global warming potential indicator.

Most steel producers supplying the UK market have published EPD for their products including open and close hot-rolled sections and plate. These are available directly from the producers and are also generally available within LCA and embodied carbon assessment software.

Downstream EPD covering steelwork fabrication and Modules A4 and A5 are generally not available because of the bespoke nature of individual fabricated products and projects.

1.5 Embodied carbon targets

In the absence of UK regulation of embodied carbon and definitive targets or benchmarks, several organisations have proposed embodied carbon targets that are voluntarily adopted on some projects. These include whole building embodied carbon targets, in terms of $kgCO_2e/m^2$ of gross internal floor area, by RIBA[16], the Low Energy Transformation Initiative[17] (LETI) and the Mayor of London[18] and 'structure only' targets (SCORS) proposed by the IStructE[19]. The SCORS (Structural Carbon Rating Scheme) targets are derived using the LETI targets. Some organisations have also established their own internal embodied carbon targets. Targets are generally limited to 'upfront' embodied carbon, i.e. Modules A1-A3 or A1-A5.

Embodied carbon targets are generally based on understanding of current and best practice although it is recognised that currently (2024), there is not a good dataset of robust, embodied carbon benchmarks. Embodied carbon targets are proposed typically for three different building types, residential, education and commercial. However, there is significant variation in embodied carbon benchmarks for different building types that can vary dramatically in form and scale. Further granularity of embodied carbon targets, both by building type and building element, is required before robust targets can be developed.

LETI proposes baseline, 2020 and 2030 best practice targets for 'upfront' embodied carbon (Modules A1-A5 only). Although LETI does not include Modules C and D within the scope of these targets, they do propose targets both for incorporating reclaimed construction products in new buildings and designing new buildings for deconstruction and reuse.

It is noted that the LETI and RIBA embodied targets vary both in terms of the scope of assessment and that the LETI targets are 'design targets' whereas the RIBA targets are 'built targets'. There is also a lack of transparency and justification on how their future embodied carbon targets have been derived. Current targets for commercial office buildings, based on gross internal floor area, are summarised in Table 1.1.

Table 1.1 Upfront embodied carbon targets for office buildings

Organisation	2020 target ($kgCO_2e/m^2$)	2030 target ($kgCO_2e/m^2$)	Scope/Notes
LETI[17]	600	350	Module A1-A5 Design target, excluding sequestration 40% 2020 reduction relative to the 1000 $kgCO_2e/m^2$ benchmark 65% 2023 reduction relative to the 1000 $kgCO_2e/m^2$ benchmark
RIBA 2021 V2[16]	<970 (2025 target)	<750	Modules A1-A5, B1-B5, C1-C4 including sequestration Built target Benchmark 1400 CO_2e/m^2
Mayor of London[18]	950 (benchmark)	600 (2030 aspirational target)	Modules A1-A5 excluding sequestration Aspirational target is based on a 40% reduction

Work is underway to develop a UK Net Zero Carbon building standard which is expected to derive new embodied and operational carbon benchmarks and targets that will be based both on current practice and on current and future UK carbon budgets. These targets are likely to supersede those shown in Table 1.1.

1.5.1 Embodied carbon by building element

Both LETI and the Mayor of London provide similar elemental breakdowns of embodied carbon for office buildings as shown in Figure 1.3.

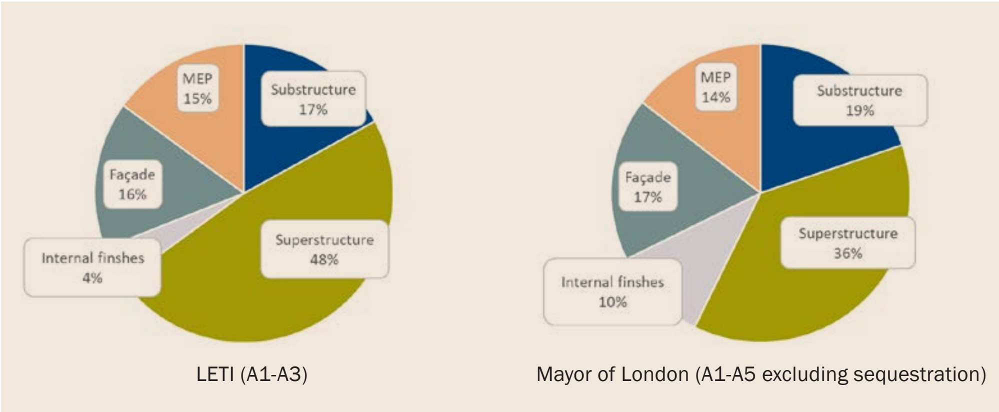


Figure 1.3 Elemental breakdown of embodied carbon for office buildings

Details of how the breakdowns in Figure 1.3 have been derived is lacking. It is understood that these elemental splits only apply to today's embodied carbon benchmarks and cannot be applied to future embodied carbon targets since the opportunity to decarbonise different elements of buildings varies

LETI does not define 'superstructure' but the Mayor of London defines superstructure to include:

- Frame
- Upper floors incl. balconies
- Roof
- Stairs and ramps
- External walls
- Windows and external doors
- Internal walls and partitions
- Internal doors.

The IStructE SCORS uses the LETI elemental breakdown (Figure 1.3 above) to derive structure-only embodied carbon targets for the sub and superstructural building elements.

When considering the frame only, and setting an embodied carbon target just for the frame, there is a lack of granularity of the breakdown of the structural elements.

Furthermore, there are significant project variations that will influence the embodied carbon of the frame and the superstructure including, but not limited to:

- the type and height of the building,
- the floor system,
- the structural grids,
- whether BF-BOF or EAF structural steel is used.

1.6 Embodied carbon from steelmaking

Today, steelmaking is dominated (99.5%) by two production processes:

1. Blast furnace-basic oxygen furnace (BF-BOF) involving the reduction of iron ore in a blast furnace (BF) using coke. The liquid iron is then converted into steel in the basic oxygen furnace (BOF). Steel scrap is added to the BOF, typically 15-25%.
2. Electric arc furnace (EAF) production uses an electric arc to melt materials charged to the furnace. Most (≈80%) EAF production today uses scrap to produce secondary steel, but direct reduced iron (DRI) is also used either on its own or mixed with scrap and alloys.

DRI is made by reducing iron ore at a temperature lower than the melting point of iron. Reducing gases used in DRI production are currently either derived from coal or more commonly (85% globally) natural gas. DRI is used in both the BF, to optimise the mix, and more commonly, in EAF production generally, in addition to scrap, to maintain steel quality. While global DRI production is only around 100 Mt today (2024), it has great potential as a low-carbon technology using hydrogen as the reductant.

Globally BF-BOF currently accounts for 71% of all steelmaking and EAF 29% [3]. The dominance of BF-BOF production is driven mainly by the finite supply of scrap and growing global demand for steel particularly in developing economies, which currently exceeds scrap supply by a factor of around 3 – see Section 1.10.

1.7 Steel construction products

All steel construction products can be produced using either of the two principal steelmaking routes, i.e. primary Blast Furnace-Basic Oxygen Furnace (BF-BOF) or the secondary, Electric Arc Furnace (EAF) route. However, for various technical and economic reasons, some products are currently preferentially produced by one or other of these routes as indicated in Table 1.2.

Table 1.2 Manufacturing routes for steel products

Product category	Manufacturing route	Products
Long products	Blast furnace route and electric arc furnace route	Sections; Rebar; Wire rod
Flat products	Blast furnace route	Plate; Hot-rolled coil; Cold-rolled coil; Hot-dip galvanised; Organic coated flats

In EAF steelmaking in the UK and mainland Europe, the primary input is scrap steel and the type of the steel produced is heavily influenced by the blend and quality control of the input scrap. Globally, around 80% of EAF production is directly from scrap steel (the remainder being from DRI-derived steel).

Traditionally, it has been more difficult to make relatively thin, flat steel products (including plate and hollow sections) from scrap steel due to the variable nature of the scrap input and therefore today, the majority of EAF steel is used for long products such as hot-rolled sections and wire rod. Hot-rolled sections are produced, equally efficiently, via either production route, while plate and hollow sections used in the UK today tend to be produced via the BF-BOF route.

1.8 Embodied carbon in the structural steel supply chain

All organisations in the structural steel supply chain add to the total or whole life carbon footprint of erected steelwork. Although actual impacts are both product and project-specific, for example the complexity of the fabricated product, any protective coatings applied, the location of the construction site relative to the steelwork contractor and the assumed end-of-life scenario, Table 1.3 gives indicative ranges of the embodied carbon emissions associated with structural steelwork fabricated and erected the UK.

BS EN 15978 and RICS whole life carbon assessment guidance, require all life cycle stages or modules to be assessed, including Module D. If embodied carbon comparisons are to made between steel and other structural materials, it is important that the equivalent assessment scope is used for all materials and that, if known, any project specific factors are accounted for.

Activity	CEN TC 350 Module	Indicative carbon emission (tCO_2e per tonne of fabricated steelwork)	Notes	Reference
Steel production including upstream mining, etc	A1-A3	0.047	Reclaimed and reusable steel sections	Reusable steel EPD from EMR
	A1	0.524	100% scrap-based EAF production of hot-rolled sections	Histar structural steel sections EPD
	A1	0.33	100% scrap-based EAF production of hot-rolled sections using renewable energy	Xcarb structural steel sections and merchant bars
	A1	2.45	BF-BOF production of hot rolled sections	British Steel steel rails and sections EPD

Table 1.3 (continues)

Activity	CEN TC 350 Module	Indicative carbon emission (tCO_2e per tonne of fabricated steelwork)	Notes	Reference
	A1	1.64	UK consumption based average of hot rolled sections (2019-22)	BCSA[20]
Transport from steel mill to fabricator	A2	0.024	Emissions are highly dependent upon the location of the steel mill or stockholder relative to the steelwork fabricator	Value is based on an average of UK steelwork contractors
Average fabrication impact	A3	0.08-0.1	Emissions depend on the complexity of fabrication. Any protective coatings are not included	Value is based on an average of UK steelwork contractors
Transport from fabricator to site	A4	0.013	Emissions are highly dependent upon the location of the steelwork contractor relative to the construction site	Value is based on an average of UK steelwork contractors
Steel erection	A5	0.02	Erection emissions are highly dependent on the type and height of the building	Value is based on an average of UK steelwork contractors
Building use stage	B	-	In-use impacts from repair, replacement, etc. are not relevant to the the steel structure	
Building end-of-life	C	-	End-of-life impacts are scenario-specific and therefore should be estimated based on project-specific	PE International - Average end-of-life data for steel sections used in a building in the UK. End-of-life assumptions: 86% recycling 13% reuse 1% landfill
End-of-life deconstruction/ demolition	C1	0.02	Average demolition of a steel structure	
End-of-life transport	C2	0.04	Average impacts of transporting scrap to UK steel mills and exporting scrap	
End-of-life waste processing	C3	-	Scrapped steel sections are assumed to reach 'end-of-waste' state during demolition (C1) and therefore no further processing is required	
End-of-life disposal	C4	0	Environmental impact of inert steel waste in a typical European municipal waste landfill	

Table 1.3 (continues)

Activity	CEN TC 350 Module	Indicative carbon emission (tCO_2e per tonne of fabricated steelwork)	Notes	Reference
Reuse, recovery and recycling potential	D1		Module D1 impacts depend both on the Module A1 and Module C assumptions therefore the D1 value should be used together with the relevant A1 value	
	D1	0		Reusable steel EPD from EMR
	D1	0.087	88% recycling 11% reuse	Histar structural steel sections EPD
	D1	0.214	88% recycling 11% reuse	Xcarb structural steel sections and merchant bars
	D1	-1.6	92% recycling 7% reuse	British Steel steel rails and sections EPD
Flat products	D1	-0.914	UK consumption based average of hot rolled sections (2019-22)	BCSA

Table 1.3 cont... Indicative embodied carbon emissions for UK structural steelwork

1.9 Influence of the structure on operational carbon emissions

The orientation and structural layout of buildings can significantly impact operational carbon performance, for example:

- Using glazing, rooflights, etc. to allow solar gain and natural lighting;
- Providing solar shading to prevent overheating and glare;
- Using natural or mixed-mode ventilation wherever possible rather than full mechanical ventilation.

Many of these aspects are complex and strongly interrelated and consequently, require expertise including detailed dynamic thermal modelling, to optimise design solutions. As such, they are not within the scope of this design guide.

The impact of the structural system on the operational carbon emissions from non-domestic buildings is generally small. Detailed, dynamic thermal modelling undertaken by AECOM as part of the Target Zero research programme[21], found that for a range non-domestic buildings and structural floor systems, the difference in operational carbon emissions varied by less than 1% per year for all building typologies.

Two important issues relating to the operational carbon emissions of steel-framed structures, are thermal mass and thermal bridging, discussed in the following sections.

1.9.1 Thermal mass

Thermal mass, or fabric energy storage, is the ability of a material to absorb and store heat. It is important in construction because, utilised effectively, it can act as a thermal 'flywheel', smoothing out temperature variations within a building leading to reductions in operational carbon emissions particularly reduced cooling loads in commercial office buildings.

To take advantage of thermal mass, it was considered that buildings had to be structurally 'heavy' to provide sufficient physical mass to absorb heat during the day. However, research and dynamic thermal modelling has demonstrated that this is not the case and it is the exposure of the building elements, particularly the upper floor soffits, and providing good night time cooling that are the key factors.

Systems for mobilising thermal mass are generally described as either passive or active.

Passive systems rely on natural ventilation to disperse the heat absorbed by the upper floor slabs.

In active systems, the heat exchange with the structure is enhanced by mechanical ventilation, either within the core of the slab or over its surface. In practice, the methods used to mobilise thermal mass in the UK are usually mixed-mode systems combining natural and mechanical ventilation, with natural ventilation being the default mode to minimise energy consumption. Active systems are generally air-based or liquid-based systems and include:

- Under floor ventilation with exposed soffits;
- Exposed hollowcore slabs with mechanical ventilation;
- Water cooled slabs;
- The use of phase-change materials in floor slabs, semi-permeable ceilings and in ventilation systems.

1.9.2 Thermal bridging

Thermal bridges occur where the building envelope is penetrated by a material with a significantly higher thermal conductivity than the surrounding materials, and at interfaces between building elements where there is a discontinuity in the insulation. Thermal bridges result in local heat losses, consequently more energy is required to maintain the internal temperature of the building and lower internal surface temperatures can be found around the thermal bridge. Cold internal surface temperatures can cause condensation which may lead to mould growth.

Steel has a high thermal conductivity compared with many other construction materials, meaning that in steel construction systems, both the structural frame and cladding, must be carefully designed to minimise thermal bridging.

Thermal bridges in building envelopes may be caused by:

- Geometry, e.g. at corners which provide additional heat flow paths;
- Building envelope interfaces, e.g. window sills, jambs and headers;
- Structural interfaces, e.g. floor to wall junctions, eaves;
- Penetration of the building envelope, e.g. balcony supports, fixings and structural elements;
- Structural considerations, e.g. lintels, cladding supports;
- Poor construction practice, e.g. gaps in insulation, debris in wall cavity.

For some buildings there may be situations where structural steel elements penetrate the insulated envelope, e.g. canopies and roof members, or where they are fixed to other steel components, such as balcony brackets and brick support units. These areas require careful consideration to minimise thermal bridging. Further information and guidance is provided in SCI P380[22].

There are three ways of reducing thermal bridging in steel construction:

- Eliminate the thermal bridge by keeping the steelwork within the insulated envelope;
- Locally insulate any steelwork that penetrates the envelope;
- Reduce the thermal transmittance of the thermal bridge by using thermal breaks, changing the detailing or by including alternative materials.

Where structural forces are transferred through steel elements that pass through the insulated building envelope, such as in balcony connections, brickwork support systems and roof structures, the form of break must be considered carefully. It is vital to ensure that the structural performance remains acceptable. Materials used for thermal breaks will generally be more compressible than steel. Therefore, deflections, as well as strength, should be checked when thermal breaks are used.

1.10 Measuring and setting embodied carbon targets for steel structures

Hot-rolled structural steel sections, of the same grade and quality, are produced both by the BF-BOF and the EAF route. Their ‘upfront’ (Module A1) embodied carbon however, varies markedly with EAF sections typically having only 20% of the embodied carbon of BF-BOF sections.

To meet project or building level embodied carbon targets, such as LETI or RIBA, an obvious response is to specify EAF sections for the steel structure. Unfortunately, in a global context, this decision will not reduce carbon emissions from steelmaking. In fact, additional transport impacts from importing EAF sections can increase overall embodied carbon emissions.

As shown in Figure 1.4, global demand for new steel currently exceeds the supply of scrap steel by a factor of nearly three and therefore, to meet this growing demand (expected to grow by a further 30% by 2050), new steel has to be produced from primary sources, today largely via the BF-BOF production route. Supplies of ferrous scrap will increase over time but are finite and exclusively specifying 100% recycled content steel, to meet building-level targets will not reduce global carbon emissions whilst global demand exceeds scrap supply.

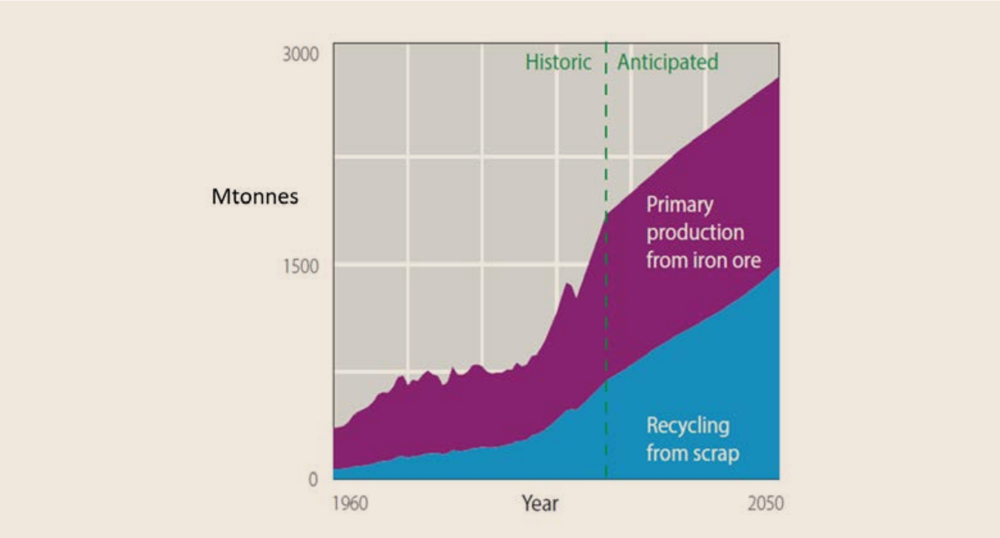


Figure 1.4 Global demand for steel and anticipated production processes (from Reference [23])

Substituting structural steel with 'lower-carbon' materials is similarly impacted by constrained supplies of finite resource, for example, GGBS (ground granulated blast furnace slag, a by-product of BF-BOF steelmaking) as replacement for clinker in concrete and the availability of land and time to grow sufficient sustainably sourced timber. This has been recognised in the IStructE-led position paper on the efficient use of GGBS[24] which concluded that GGBS is a limited and constrained resource that is almost fully utilised globally and therefore locally increasing the amount of clinker substituted with imported GGBS is unlikely to decrease global GHG emissions.

Instead, structural engineers are encouraged to use their expertise, experience and influence to reduce demand for construction materials by following the IStructE Net Zero Design hierarchy and the guidance in this document. By doing so, total demand for new steel can be reduced and therefore the amount of steel required to be produced by the primary (BF-BOF) production route can be reduced.

The variation in the embodied carbon of structural steel, makes setting absolute embodied carbon targets for steel-framed buildings difficult. For example, a very efficient structural design using BF-BOF steel sections will have a higher embodied carbon footprint than an inefficient design using EAF sections.

1.11 Specifying low carbon steel

Figure 1.5 shows the distribution of GHG emissions intensity of crude steel production as a function of input scrap content. It includes data from 290 steel mills in Europe, America, China and India; the data, modelled by CRU[25], is from 2019.

The graph shows two clusters, one at either end of the recycled content axis. The cluster to the right is the near-100% scrap-based EAF production and the cluster to the left, BF-BOF production. Note that the limit of scrap input to the BOF process is around 30%, accounting for the spread of results in the zero to 30% recycled content range.

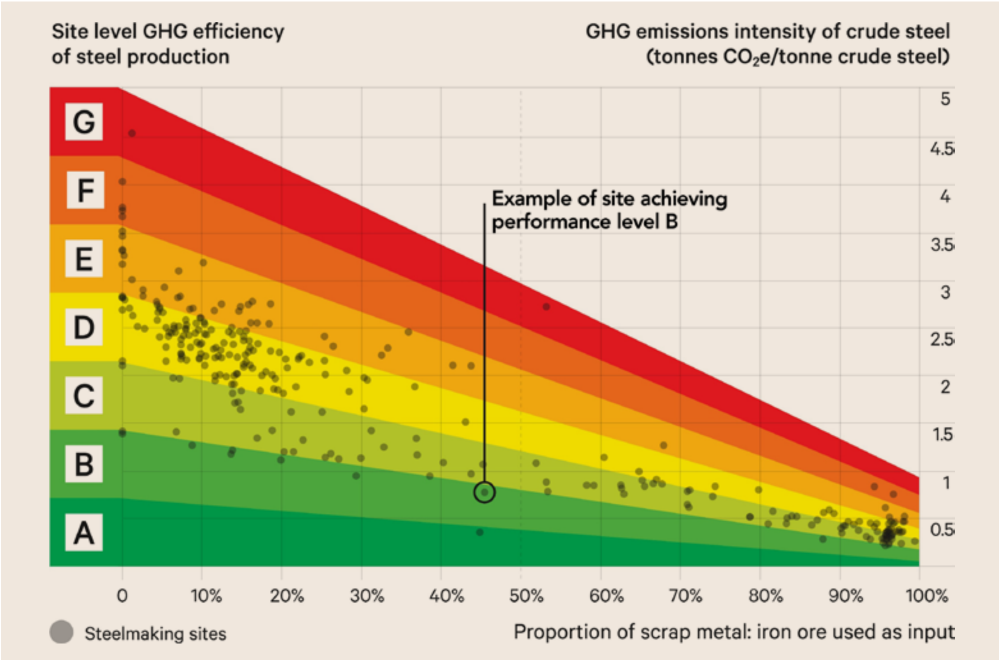


Figure 1.5 GHG emissions compared to input scrap content[26]

The efficiency banding in the graph is the approach advocated by ResponsibleSteel, SteelZero and others. Cognisant of the constrained global supply of scrap relative to the global demand for new steel, this approach incentivises all steelmakers to decarbonise. Rather than specifying an absolute embodied carbon target in terms of CO_2e per tonne of crude steel, it is recommended that if steel is used, it should be specified from, for example, a band C or B steel mill. Note that the banding or performance levels shown in Figure 1.5 are indicative only.

The ResponsibleSteel standard is aligned with the SteelZero initiative which has been developed for users of steel to encourage steel producers to decarbonise. Companies that sign up to SteelZero make a long-term commitment to procure/specify/stock 100% net zero steel by 2050 and make a minimum interim commitment to procure/specify/stock 50% of steel requirement by 2030, by meeting one or a combination of the following criteria:

1. Steel produced by a steelmaking site where the site's corporate owner has defined and made public both a long-term and a near-term emissions reduction target, validated by the Science-based Targets initiative (SBTi) or other quantitative, scientifically justified target of comparable ambition, quality and coverage.
2. Steel meeting the minimum threshold for 'Lower Emission Steel'*, or equivalent. This requirement can currently be met by procuring ResponsibleSteel certified steel meeting minimum Decarbonisation Progress Level 2 as shown in Figure 1.6.

“Lower emission steel” is described as the GHG emissions intensity threshold of <2000 to <350 $kgCO_2e$/tonne of crude steel dependent on 0 - 100% scrap share of metallic inputs; where the scope boundary covers from cradle to crude steel and the scrap boundary includes pre-consumer (home and manufacturing scrap) and post-consumer (end-of-life) scrap. This threshold aligns with ResponsibleSteel Decarbonisation Progress Level 2.

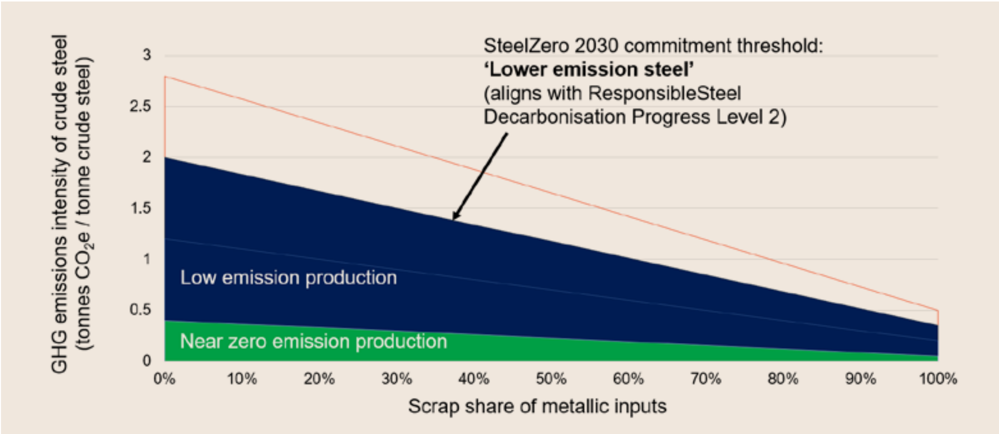


Figure 1.6 ResponsibleSteel decarbonisation progress targets

1.12 Early design stage embodied carbon assessment

The variation in the embodied carbon of structural steel, makes early design stage embodied carbon assessment and decision-making difficult since, at this stage of the project, the specific product or steel supplier (and production route) will generally not be known.

The approach adopted in carbon assessment guidance developed by RICS[12] and the IStructE[11], is to base the assessment on average values. For hot-rolled structural steel sections, both RICS and IStructE refer to the UK consumption average calculated by BCSA[20] which is currently 1.64 tCO_2e per tonne; this is the 4-year average for 2019-22.

Despite the limitations of setting project-level embodied carbon targets for steelwork in the context of global GHG emissions, it is recognised that some clients may continue to set such targets.

The variation in the embodied carbon of structural steel can result in a very efficient structural design using BF-BOF steel sections having a higher embodied carbon footprint than an inefficient design using EAF sections. This is not in alignment with the IStructE hierarchy (See Figure 1.1) adopted in this design guide and it is recommended that the first priority is to develop a structurally efficient design before calculating the embodied carbon impact of the structure. The efficiency of the structure may be measured in terms of steel weight per m^2 of floor area. An additional discipline is to review the average utilisation of the primary structure as a further metric of structural efficiency. This approach allows the assessor to uncouple (not ignore) the carbon impact of the steel from the efficiency of the structural design.

An example of this is shown in Figure 1.7 where the primary steel structure design is compared against carbon intensity ($kgCO_2e/m^2$) on the y-axis and against structural efficiency (kg steel per m^2 of floor area) on the x-axis. Ideally, the project should fall within the envelope of the dashed blue and orange lines. The carbon intensity for the project has been calculated using the UK consumption-based average value for steel sections. The error bar indicates the range of carbon intensity achievable using EAF and BF-BOF sections exclusively on the project. The ratio or split of BF-BOF and EAF sections to achieve the chosen embodied carbon target is also given; in this case, 67% BF-BOF and 33% EAF. This is the 'bounding approach' recommended by the IStructE.

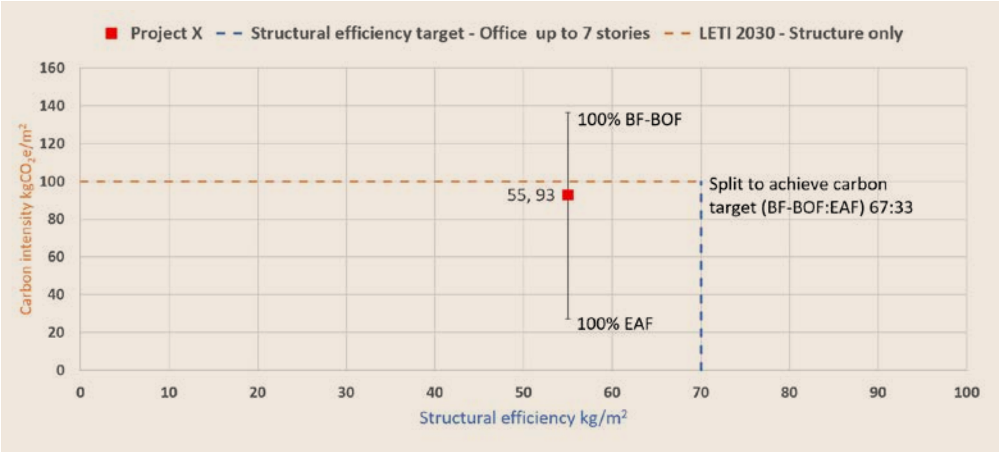


Figure 1.7 Uncoupling structural efficiency and embodied carbon intensity of a structural steel frame

The lack of granularity of available embodied carbon data makes it difficult to set targets for specific elements of a building, for example, just the primary structure. The LETI (A1-A5) embodied carbon targets, refer to the whole building. LETI has also provided rules of thumb for the breakdown of embodied carbon by building element and IStructE has used these breakdowns to derive structure-only targets (SCORS). However, IStructE's definition of superstructure includes upper floors, roof, stairs and load-bearing internal and external walls and partitions in addition to the structural frame.

By way of example, the LETI 2030 A1-A5 target for commercial office buildings is 350 $kgCO_2e/m^2$ of gross internal floor area. The proportion attributable to the superstructure of office buildings is 48%, i.e. 168 $kgCO_2e/m^2$. The proportion attributable to the structural frame only will depend on the flooring system, structural grids and the internal and external walls, etc.

In terms of structural efficiency, setting definitive targets is difficult because of the bespoke nature of buildings. The following indicative guidelines for multi-storey steel frames are provided but companies may wish to develop their own internal benchmarks and targets to help them design efficiency and to identify and understand why structural steel weights exceed targets on certain projects. This level of scrutiny of design efficiency can be overlooked if only embodied carbon metrics and targets are employed.

Typical weight ranges for multi-storey steel frames, and the supporting assumptions, are given in Table 1.4. The tabulated weights are for the steel members only.

Table 1.4 Typical steel weights in multi-storey buildings

Building	Columns	Floor beams	Bracing	Total
Low rise (2 to 6 storeys)	In S355: 3 – 7 kg/m^2 In S460: 3 – 6 kg/m^2	25 – 35 kg/m^2 Average 30 kg/m^2	3 - 5 kg/m^2	35 – 50 kg/m^2
Medium rise (7 to 12 storeys)	In S355: 8 - 14 kg/m^2 In S460: 6 - 10 kg/m^2		5 - 10 kg/m^2	40 – 60 kg/m^2

More detailed weights and embodied carbon values are given in Appendix B.

1.13 Minimising waste

All structural steel components are manufactured off-site in the controlled environment of a fabrication factory, where consistent structural elements with assured quality and full traceability can be created to meet the specific requirements of each project. In this environment, steel parts can be easily standardised, tested and certified. Any waste material produced during the fabrication phase, e.g. off-cuts, swarfe, etc. can be recycled and used again in the steelmaking process.

Off-site manufacture is more efficient, faster, leaner and safer than site construction. It also yields high quality products with fewer defects that require less ‘snagging’ on site, leading to savings in both time and money.

By-products from iron and steel making, including sludges, slags and dust, are beneficially used by the construction industry in a range of products including roadstone, lightweight aggregate and as a substitute for Portland cement.

Slag is the voluminous by-product of steelmaking accounting for approximately 90% by mass of steelmaking by-products. Global average recovery and recycling rates for blast furnace slag and steel-making (BOF) slag are 100% and 80% respectively, leading to an overall material efficiency rate of 97.6% worldwide.

Approximately 1.3 million tonnes of blast furnace slag is produced in the UK annually. Of this around 75% is quenched and this is then processed to produce ground granulated blast furnace slag (GGBS), which is used by the concrete industry as a cement replacement material. The remainder is air-cooled and is used as an aggregate.

During component manufacture, computer controlled, fully or semi-automated production lines ensure that wastage of steel is minimised. The typical wastage rate for fabricating structural steel products is just 4-5% and any off-cuts, trimmings, swarfe, etc. from the production process are 100% recycled into new steel.

Steel products are delivered to the construction site pre-engineered to the correct dimensions; consequently there is no site waste. Furthermore the quality of factory-produced steel construction products and the dimensional stability of the material itself, means that there are few defects and hence little site waste. Steel products are delivered to site with minimal packaging. Packaging comprises mainly timber pallets and bearers and plastic or metal strapping, which are generally reused by the haulage company making site deliveries.

When a building is deconstructed at its end-of-life, the ease with which steel construction products can be reclaimed, coupled with the economic value of scrap steel, means that virtually all steel is recovered and either reused or recycled. It is estimated that 99% of structural steelwork from deconstructed buildings in the UK is recovered and recycled or increasingly the case, reused.

DESIGN TO REDUCE EMBODIED CARBON

2.1 Hierarchy of design decisions

If construction is to proceed at all, options for the structural designer are indicated in Figure 2.1, taken from the Institution of Structural Engineers guidance[1]. The width of the slice, which diminishes, is indicative of the extent of the opportunity to reduce embodied carbon in construction.

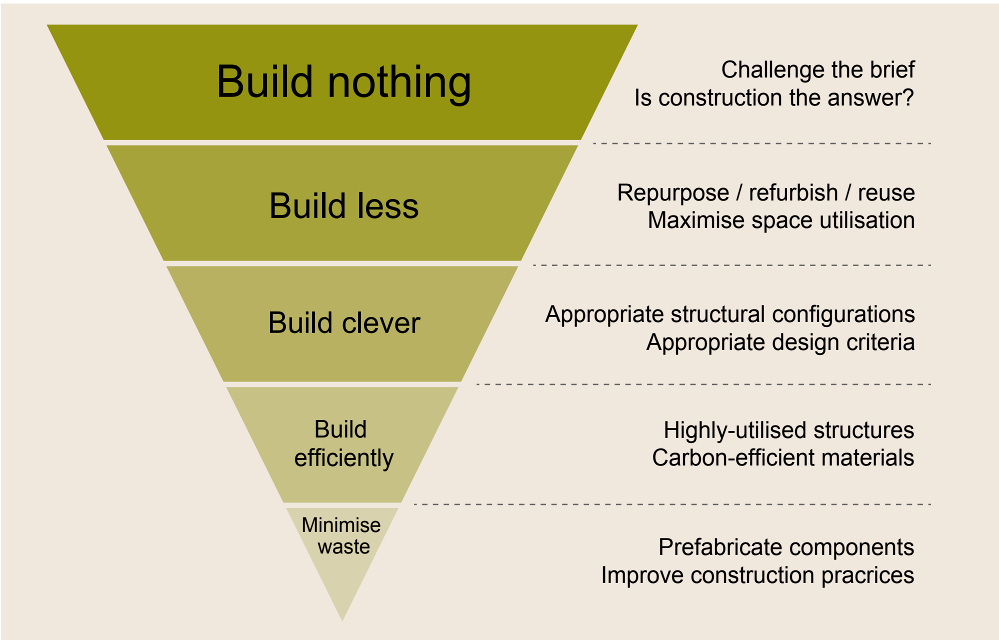


Figure 2.1 Hierarchy of design decisions

The following sections describe opportunities at each level in this hierarchy for structural engineers to reduce embodied carbon through more efficient design. Recognising that the structures constructed today will become the potential stock of building components to be reused in the future, Section 10 discusses measures to facilitate the future deconstruction and reuse.

BUILD LESS

In the hierarchy of Figure 2.1, "Build less" is to repurpose, refurbish and reuse existing building stock, rather than demolish and build new. Repurposing and refurbishment may use some new materials, but the intent would be to retain as much of the existing structure as possible. Reuse involves new structures that incorporate, where practical, reused materials recovered from redundant structures.

3.1 Utilising existing steelwork

Compared to some structural materials, steelwork is readily modified, strengthened and extended. New holes may be drilled and new fittings welded to accommodate all manner of revised configurations. Increased resistance can be provided by the addition of plates (Figure 3.1) or other members, by bolting or welding.

Additional members may readily be introduced into existing frames without damaging or reducing the resistance of the existing steelwork.

Figure 3.1 Steelwork used to adapt an existing structure

Adding further storeys to existing buildings, as illustrated in Figure 3.2, is a common way to provide additional space, enhancing the utilisation of existing structures. The additional storeys are generally constructed from lightweight steel, having assessed the existing structure and foundations for the relatively small additional load. In many cases, a realistic assessment of the variable actions (see section 4.3.1) demonstrates that the existing structure has sufficient reserves of resistance.

Figure 3.2 Light gauge modular roof top delevopment

In other cases, the tops of the existing steel columns can be exposed, surveyed and extended to provide additional enclosed space at the roof level.

Over-cladding and over-roofing are common techniques to insulate and /or prevent water ingress affecting older buildings perhaps constructed to less rigorous regulations, or buildings with faults, or buildings suffering general deterioration.

The common theme is that steelwork provides an opportunity to increase and extend a structure's functionality in contrast to a solution involving demolition and rebuilding.

3.2 Assessment of existing structures

To investigate the opportunity to repurpose, modify or extend the life of an existing structure, a thorough appraisal of the building will be required. Detailed guidance is available in the Institution of Structural Engineers guide on the appraisal of existing structures[27].

SCI publication P138[28] provides a history of the use of iron and steel, typical details and connections found in older buildings and recommended approaches to appraisal, testing, fire protection and strengthening.

The Historic Structural Steelwork Handbook[29] provides details of steel strengths and section properties for older steel sections.

3.3 Reuse of steelwork

If construction is to proceed, an ideal solution is to reuse steelwork recovered from other redundant structures, as illustrated in Figure 3.3. Reused steel has an upfront embodied carbon content of approximately 2% of new BF-BOF steelwork (see Table 1.3), representing a very valuable reduction.

Figure 3.3 Recovered structural steelwork

At the present time (2024), although interest in reused steel is increasing dramatically, only relatively small tonnages of recovered steel have been reused. This is expected to grow in the short term as clients, architects and designers appreciate the carbon savings achievable and the availability of suitable reclaimed sections increases.

No technical barriers exist to prevent the reuse of steelwork. Extensive design guidance is presented in SCI publications P427[30] and P440[31]. Re-fabricated reused steelwork will be CE/UKCA marked, offering the same reassurance with respect to steelwork execution as that of new steel.

Understandably, the key concern with reused steelwork relates to the mechanical properties of the recovered material since it is not coming directly from the manufacturer with accompanying test certificates and CE/UKCA marking. With reused steel, the stockist takes on the equivalent responsibilities of the manufacturer, firstly determining the relevant mechanical properties by non-destructive and destructive testing and then declaring these with the supplied members.

Reuse of existing steelwork is almost certain to involve refabrication, cutting to a new length and adding connections as a minimum. More extensive fabrication can have significant advantages – Figure 3.4 shows the re-fabrication of an 838 mm Universal Beam into a 1300 mm deep cellular beam.

Original 838 mm UB

Figure 3.4 Re-fabrication of a plain beam into a cellular beam

Refabricated 1300 mm deep cellular beam

3.3.1 Reuse efficiency

Reusing steelwork means less scrap is available to produce new steel via the electric arc furnace (EAF) route. As global demand for steel outstrips the availability of steel produced by the EAF route, the more carbon-intensive blast furnace route is still required to fulfil the global demand for new steel. There is therefore a balance between reusing steelwork inefficiently (essentially wasting resistance or stiffness) and scrapping the member to produce new steelwork.

Annex J of the National Structural Steel Specification[32] suggests “as a rule of thumb” that if a reused member is 20% heavier than an efficiently designed new member, recycling may be the more appropriate option in terms of carbon emissions. If reuse of recovered steel continues to grow, a larger volume and a wider range of sections sizes should become available. A wider range of available sections will enable the selection of members with just sufficient resistance, rather than an excess of resistance.

3.3.2 Reuse target

Many months may pass between the completion of a structural design and the start of fabrication. At the design stage, identifying specific members within stocks of reused steelwork is not appropriate as stock is likely to change in the intervening period.

Designers are encouraged to identify a target (for example a percentage of the overall tonnage) for reused steelwork for the steelwork contractor to attempt to source as the fabrication contract commences. To facilitate reuse, designers should:

- Develop a design which is as efficient as possible;
- Identify where reused steelwork may be used;
- Specify any limiting constraints, such as deflection limits or physical size;
- Provide envelopes of acceptable characteristics (such as minimum and maximum depth).

3.4 Design criteria checklist

1. Has the option of not building at all been properly considered?
2. Has the option of adapting, refurbishing or extending been considered?
3. Has the re-use of recovered steel members been specified, encouraged and facilitated?

BUILD CLEVER

For designers, “building clever” should cover a synthesis of structural arrangement and members, efficient use of materials and appropriate design criteria. “Building clever” also means facilitating sustainable behaviours in the future, discussed in section 10.

If “building nothing” is not an option, and “building less” has been properly recognised, the responsibility of a designer is to prepare a design which is safe, structurally efficient and with minimum embodied carbon, recognising the importance of overall whole life carbon as discussed in Section 1.2.1. As discussed in Section 1.12, the priority is firstly to develop a safe and efficient structure to minimise steel consumption, rather than to demonstrate low carbon by (for example) specifying only steel produced by the EAF route.

Overdesign is perceived as reducing risk and carries no significant penalty for the designer, but “doing good”[33] would not be so wasteful. Designers often cite the lack of time and fees required to prepare a design with finesse in the later stages of a contract, so clients may need to be prepared to spend more time on this critical stage or encourage their designers to develop the solution earlier. The IStructE Plan of Work[34] presents good practice when undertaking projects, identifying how far the design should be developed at each stage (adopting the same stages as RIBA[35]) and allowing a design contingency at each stage. Following this guidance should ensure that the design is properly developed as each stage progresses.

This Section and those following discuss the options for the designer in working towards a sustainable steel design.

4.1 Appropriate structural configurations

Structural solutions are a combination of the floor grid, the column spacing and the member types. Embodied carbon must be one of the considerations assessed by the designer when developing a solution to the client’s brief. Meeting the client’s functional requirements with a safe solution is paramount, which will involve balancing embodied carbon with requirements for useable space, servicing, heating, ventilation, lighting and access.

Design for Zero[33] calls for carbon-expensive architectural “statements” such as dramatic cantilevering structures, to be avoided. Whilst structurally and financially possible, the additional expense in embodied carbon is not justified.

The same principle should be applied to all structures: is the structural “feature” justified? Is there a solution which perhaps has less architectural drama, but meets the client’s needs and represents a more sustainable solution? Amongst other recommendations, Design for Zero challenges designers to:

- Take ownership for carbon, in the same way as designers are responsible for safety;
- Show clients the value of carbon calculations;
- Accelerate the shift to low-embodied carbon construction materials.

4.1.1 Grid dimensions

Although short spans may be perceived to offer the lowest upfront embodied carbon solution, short spans often do not meet the end client’s needs for column-free space, which facilitates adaptability. Selection of short spans could lead to premature redundancy of the working space – and a whole-life embodied carbon penalty.

When steelwork supports a concrete slab, much of the embodied carbon is associated with the slab and is therefore independent of the steel beam span. Beams with web openings offer a very lightweight, low carbon solution offering the advantages of long spans.

Figure 4.1 shows the A1-A3 embodied carbon (per unit floor area) of ribbon-cut, long-span cellular composite beams supporting metal decking and concrete. The graph has been generated using the average embodied scenario defined in Appendix D. The graph shows an embodied carbon increase of 27% by increasing the long span members from 9 m to 18 m. For any given secondary beam span, the selection of beams at 3.75 m centres leads to a more efficient design with less embodied carbon than specifying beams at 2.5m centres (see also Figure 4.4).

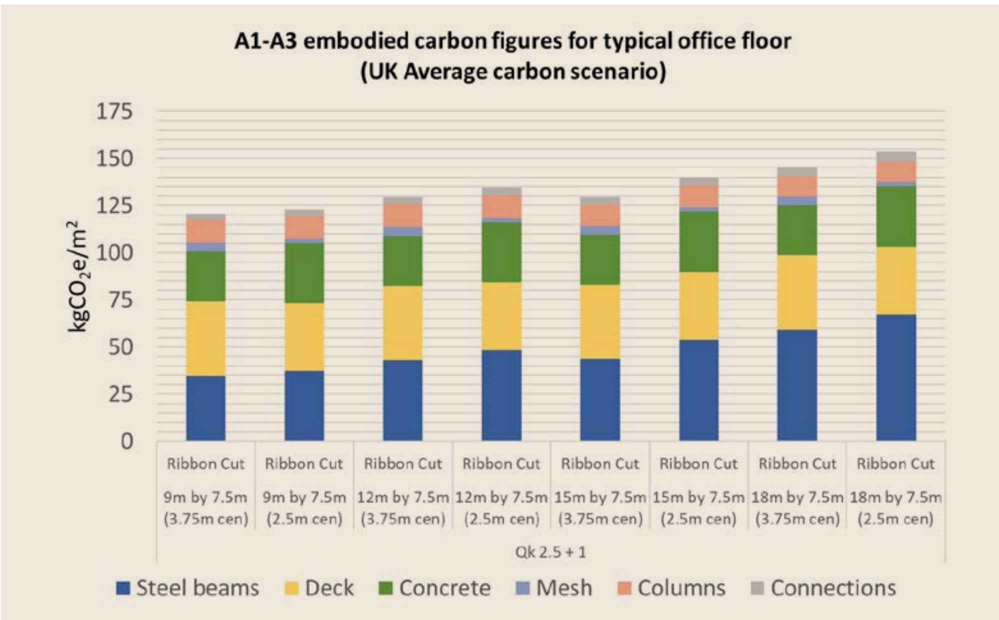


Figure 4.1 UK average embodied carbon values for typical office floor grids

Figure 4.2 shows the embodied carbon of the same composite beam-floor designs but generated using the low embodied carbon scenario defined in Appendix D. As shown, an embodied carbon intensity of 40-50 $kgCO_2e/m^2$ is achievable using this low-carbon scenario.

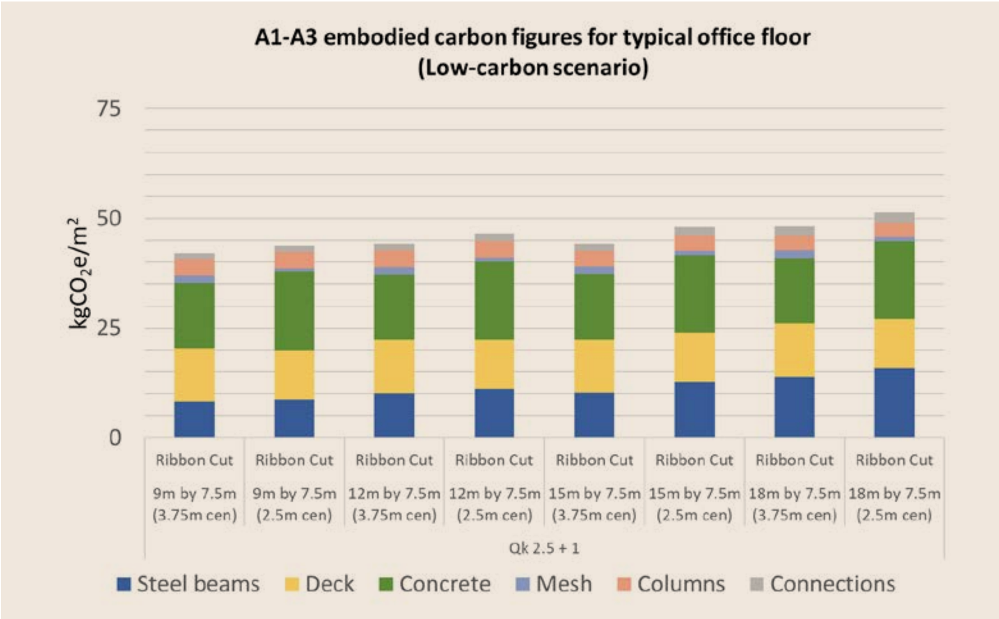


Figure 4.2
UK low embodied carbon values for typical office floor grids

Figure 4.1 and Figure 4.2 are both based on an imposed floor load of 2.5 kN/m^2 + 1 kN/m^2 allowance for moveable partitions, which are typical values used in London offices.

Since the calculated embodied carbon depends on both the design solution and the emission factors adopted for the steel, the profiled decking, reinforcement and carbon, designers are encouraged to discuss potential solutions with steelwork contractors.

Figure 4.3
Long span composite beams providing column-free space

Compared to other materials, steel solutions offer the opportunity of longer spans and flexible column-free space, as illustrated in Figure 4.3. The reduced overall weight of a steel solution – primarily because of the lighter floors - will often be reflected in smaller foundations and a reduced overall carbon burden.

Considering steel solutions only, longer steel spans will have an increase in embodied carbon for the superstructure than shorter spans. A typical relationship between secondary beam span and steelwork weight is shown in Figure 4.4, which has been prepared for a 150 mm composite slab, beams with web openings, 1.5 kN/m^2 allowed for services, finishes etc and a total variable action of 3.5 kN/m^2. Figure 4.4 is based on a primary beam span of 7.5 m – other arrangements show similar trends.

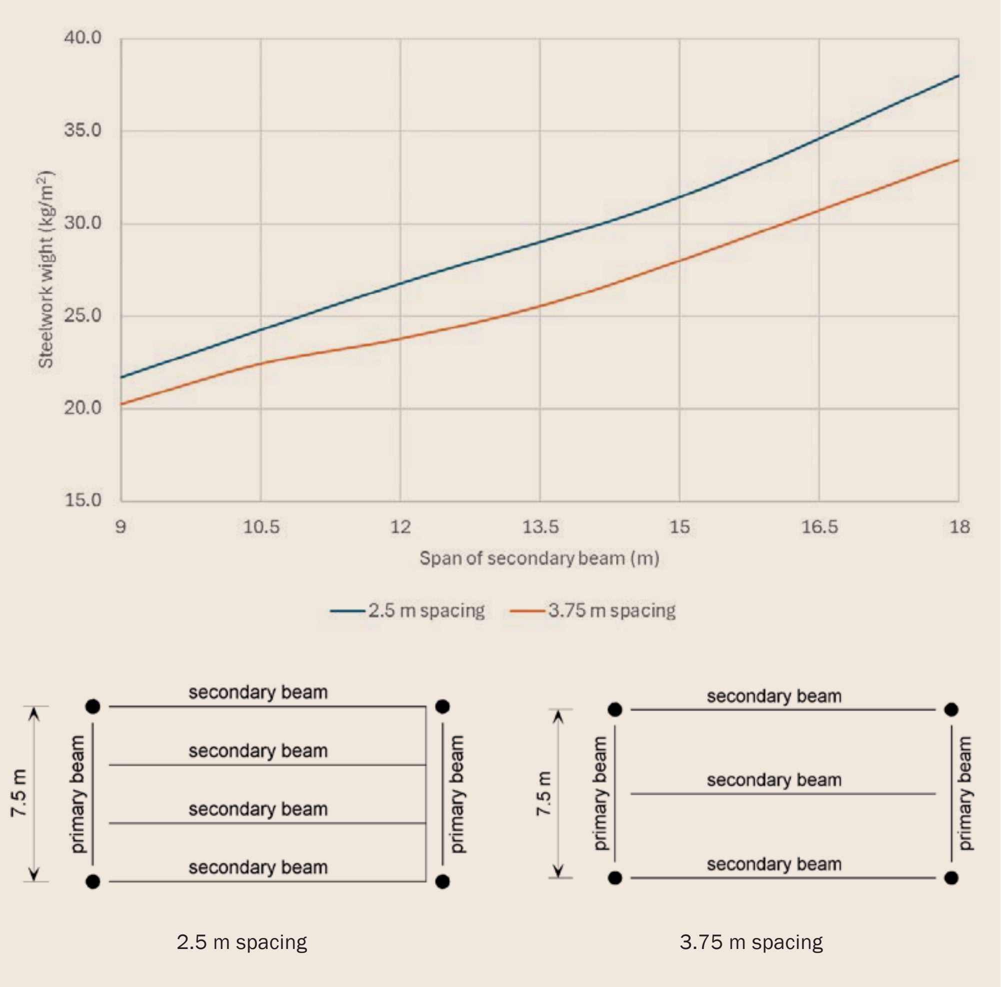


Figure 4.4
Steel weight (floor beams alone) for varying span secondary beam

4.1.2 Steel floor grids

Orthodox floors in steel framed structures comprise the floor slab and the supporting structure. Typical solutions include composite slabs, and more recently, the use of cross laminated timber (CLT) as a floor slab (Section 8). As might be expected, solutions involving CLT lead to a reduction in embodied carbon compared to a conventional composite slab. Embodied carbon values for the floor slab alone of around are given in Section 7.2 and for solutions using CLT in Section 8.7.

Given that a floor slab of some form is needed, once the choice of conventional composite slab or alternative has been made, the remaining contribution to the embodied carbon comes from the supporting steelwork, when the grid arrangement becomes important. In longer spanning configurations, the secondary beams span the longer distances, supported on shorter span primary beams, as shown in Figure 4.5. The alternative arrangement with multiple shorter secondary beams spanning onto longer primary beams is generally less efficient, increases the piece count and may be more sensitive to dynamic behaviour.

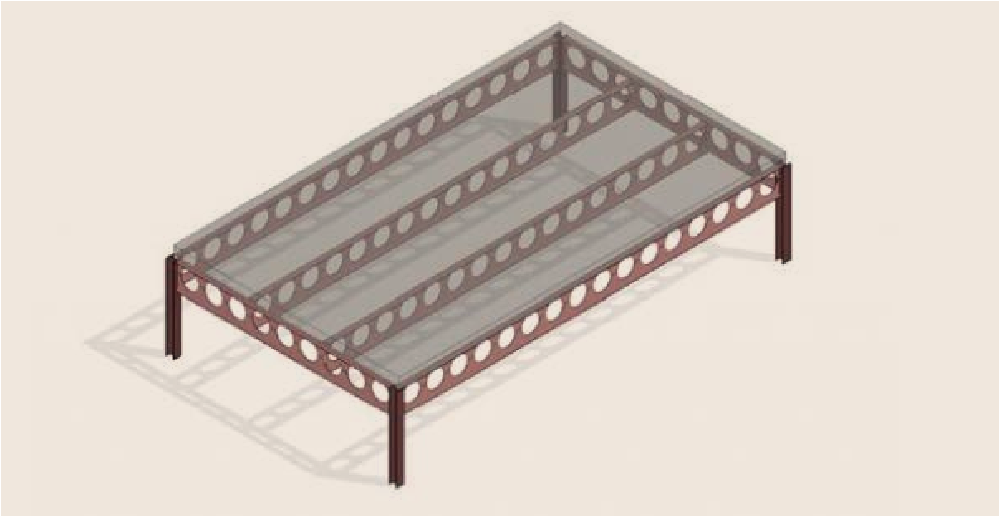


Figure 4.5 Typical long span arrangement

Two typical arrangements of a 15 m x 7.5 m grid are shown in Figure 4.6, with the associated steelwork weights for a single bay. The arrangement with shorter secondary beams is heavier, with more members to erect and more susceptible to dynamic behaviour. The steelwork weights relate only to the beams.

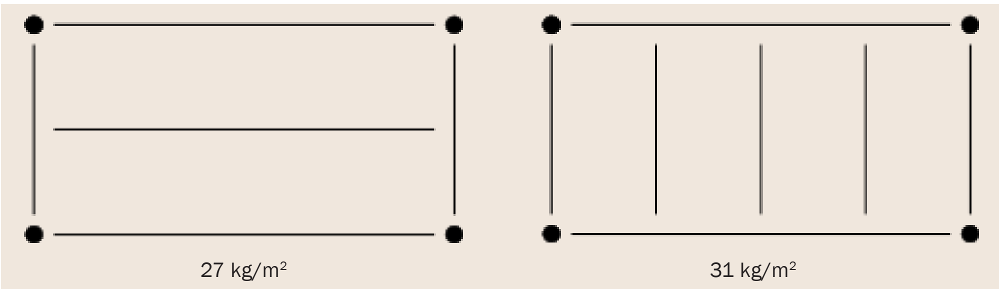


Figure 4.6 Alternative arrangements of secondary beams

For a given primary beam span, increasing the span of the secondary beam only carries a relatively small penalty in the embodied carbon of the overall superstructure. This is because in long span solutions, the beams are normally highly efficient fabricated beams with web openings and the contribution from the slab is unaltered.

4.1.3 Maximise space utilisation

In the context of "building clever", the objective of maximising space usage is to provide sufficient, efficiently used space to meet the brief. "Build clever" should not mean short spans which limit the flexibility of the structure. During the life of a building, long spans allow the space to be arranged to suit open plan offices, different layouts of cellular offices and variations of layout on different storeys.

4.1.4 Floor depth and services

Minimising the construction depth reduces the overall height of a structure (or might allow an additional storey when building height is restricted). A lower construction depth means the area of cladding is reduced around the entire façade, with associated cost and carbon benefits.

Shallow floor solutions can be heavier than floors with no restriction on overall depth. A holistic view of the overall embodied carbon in the entire structure is therefore required, considering any requirement to maximise the number of storeys, the resulting primary structural scheme and the impact on the façade.

Numerous shallow floor solutions are available where the steel floor beam is integrated within the floor construction, with services freely located below the soffit. Alternatively, services may be integrated within the steel beam depth utilising openings in the web. Extensive design guidance is available for both solutions[36],[37],[38],[39]. Shallow floor beams may be used with deep decking (similar overall depth as the steel beam) or with precast planks as shown in Figure 4.7.

Figure 4.7 Shallow floor beam and precast planks

4.2 Appropriate design criteria

The most significant influence on the design solution in any structural material, and thus the embodied carbon, is the choice of the value of the variable action adopted as a floor load. Climatic actions (wind, snow, temperature) only have a limited effect on most multi-storey building structures. Some designers consider the value of imposed floor loads specified in national standards to be excessive, believing the specified values represent loading never experienced in reality. Such an argument ignores the calibrated reliability implicit in codified requirements. The assumption in this present guide is that designers will not depart from the guidance in the standards.

The recommendation in this guide is that designers select the imposed floor load appropriate for the category of loaded area rather than being conservative by selecting a higher load than necessary. A client brief which specifies a load higher than required should be challenged and the implications in terms of cost and carbon explained and justified. The selection of the imposed floor load has a direct effect on the material required to support the design loads and thus on the embodied carbon.

4.3 Design for Ultimate Limit State

4.3.1 Imposed floor load

Variable actions are defined in the UK National Annex to BS EN 1991-1-1[40]. Distributed loads and concentrated loads are given for different categories of construction. The concentrated load is generally not relevant for the design of the structural frame, being used only for local verifications. Common floor categories and the associated distributed variable actions are given in Table 4.1

Category	Specific use	Example	q_k (kN/m²)
A1	Area for domestic and residential activities	All areas within self-contained single family dwellings or modular student accommodation Communal areas (including kitchens) in blocks of flats with limited use that are no more than 3 storeys, and only 4 dwellings per floor are accessible from a single staircase	1.5
A2		Bedrooms and dormitories except those in A1 and A3	1.5
A3		Bedrooms in hotels and motels; hospital wards; toilet areas	2.0
B1	Office areas	General office use other than in B2	2.5
B2		Office areas at or below ground floor level	3.0
C31	Areas where people may congregate	Corridors, hallways, aisles which are not subjected to crowds or wheeled vehicles and communal areas in blocks of flats not covered by A1	3.0
C51		Areas susceptible to large crowds	5.0
C52		Stages in public assembly areas	7.5
D	Shopping areas	Areas in general retail shops and department stores	4.0

Table 4.1 Minimum imposed floor loads (common categories)[40]

Designing office floors for 5 kN/m² is not uncommon, despite the guidance in Table 4.1 recommending 2.5 kN/m² for floors above ground level. Reference [52] records that less than 50% of designers would choose 2.5 kN/m² (or less) as the imposed floor load for an office – the remainder would use a value higher than that specified in the UK National Annex.

The imposed floor load is then further increased to allow for moveable partitions, as discussed in Section 4.3.2.

The British Council for Offices (BCO) Guide[41] also recommends 2.5 kN/m^2 for any floor other than the ground floor, endorsing the value specified in the UK National Annex.

Reasons for specifying larger values include possible future change of use. The IStructE guide Design for Zero[33] observes that the future cannot be predicted – and an upfront surplus of capacity should not be built-in if it required extra material. If increased resistance is required at some point in the future, the same guide recommends the adoption of appropriately designed strengthening works. Steelwork is ideally suited for this situation, being readily modified or reinforced.

4.3.2 Allowance for partitions

Historically, British Standards recommended an additional distributed load of 1.0 kN/m^2 to allow for moveable partitions. Under the Eurocode system, the allowance specified in clause 6.3.1.2(8) of BS EN 1991-1-1[42] depends on the weight of the moveable partition as shown in Table 4.2. The BCO Guide repeats these values and offers no more detailed recommendation.

Table 4.2 Allowance for moveable partitions (from BS EN 1991-1-1)

Partition self-weight (kN/m)	Additional uniformly distributed load (kN/m^2)
≤ 1.0	0.5
> 1.0 and ≤ 2.0	0.8
> 2.0 and ≤ 3.0	1.2

Typical self-weights of partitions, based on 2.4 m height, range from around 0.5 kN/m (light gauge steel and plasterboard) to around 0.7 kN/m (10 mm glass). Designers are encouraged to assess the weight of partitions and select the appropriate category. With proper consideration, the traditional allowance of 1.0 kN/m^2 could be reduced by as much as 50%.

4.3.3 Reduction factors applied to variable actions

Design standards reflect that a large floor area is unlikely to be fully loaded over the whole area at one time. A reduction in load is therefore allowed, depending on the floor area and load category type. The reduction factors can be applied to categories A, B, C and D as described in BS EN 1991-1-1, which covers most types of loading except storage loads.

In the UK, the reduction is given in clause NA.2.5 of the UK National Annex to BS EN 1991-1-1[40]. The reduction factor for individual beams is modest, as illustrated in Figure 4.8.

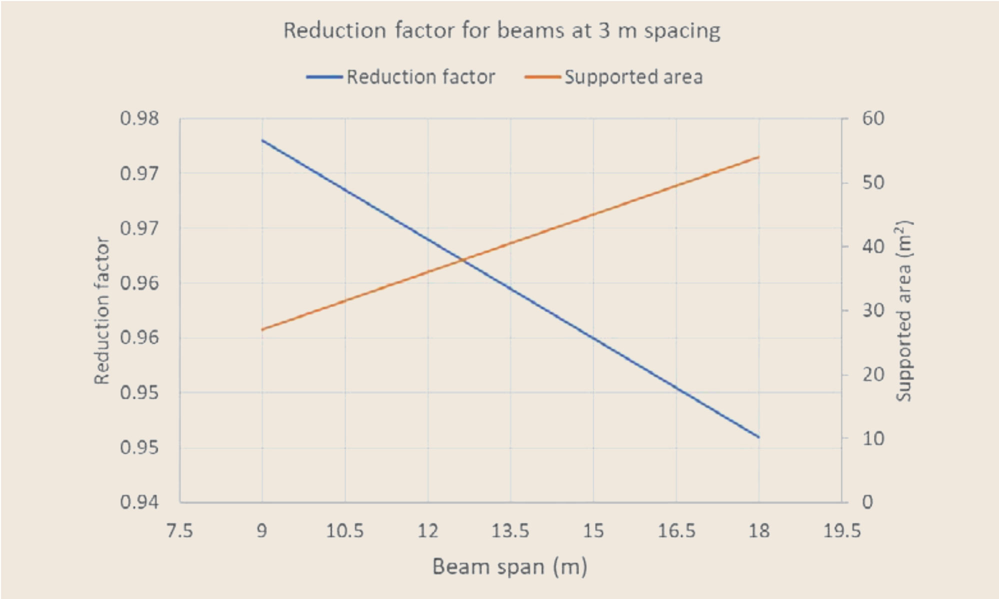


Figure 4.8
UK NA reduction factor α_A

Similarly, it is unlikely that all storeys of a building are fully loaded at the same time. A reduction in the contribution of the imposed load to the axial load in columns is given in clause NA.2.6 of the UK National Annex to BS EN 1991-1-1.

For columns lower down a multi-storey building the reduction can be considerable, reaching maximum of 50% if the column supports more than 10 storeys above. The reduction factor, α_n, is shown in Figure 4.9.

Reductions α_A and α_n cannot be used simultaneously.

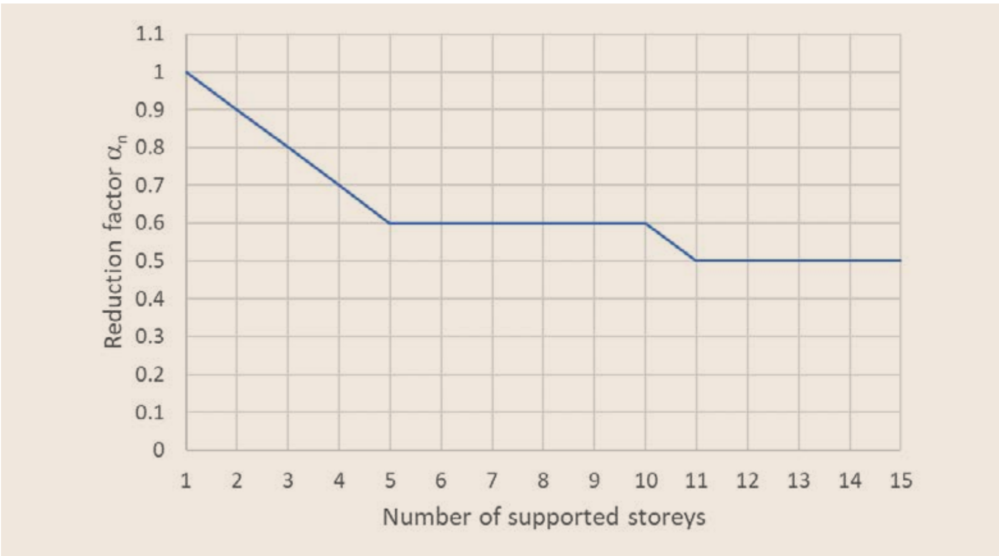


Figure 4.9
UK NA axial load reduction factor α_n
Imposed floor load from several supported storeys

4.3.4 Design combinations of actions

Combinations of actions are given in BS EN 1990[44], with partial and combination factors given in the UK National Annex[45]. BS EN 1990 covers both ultimate limit state (ULS) and serviceability limit state (SLS).

At ULS, the design value of the combination of actions may be calculated according to expression 6.10 of BS EN 1990, or by taking the more onerous result from both expression 6,10a and 6.10b. The option to use expressions 6.10a and 6.10b is a nationally determined parameter, which is permitted in the UK.

The optional use of expressions 6.10a and 6.10b is highly recommended, as the design value of the actions will be lower than that calculated according to expression 6.10.

With only one permanent action and one variable action (the typical case for the vast majority of members in a steel frame), Expression 6.10 of BS EN 1990 becomes:

$\gamma_G G_k + \gamma_Q Q_k$

where,

γ_G is the partial factor for permanent actions and has the value of 1.35 as given in the UK NA

γ_Q is the partial factor for permanent actions and has the value of 1.5 as given in the UK NA

G_k and Q_k are the characteristic value of the permanent and variable actions, respectively.

If there is more than one variable action, expression 6.10 becomes:

$\gamma_G G_k + \gamma_Q Q_k + \gamma_Q \psi_{0,i} Q_{k,i}$

The ψ_0 factors are specific to the type of action. Thus, wind actions have a specific value of ψ_0 and imposed floor loads another and are found in the UK National Annex. The subscript is an important identifier – in expression 6.10, ψ_0 values should be used, not ψ_1 or ψ_2.

If there is more than one variable action, each must be identified in turn as the 'leading' or 'main' variable action, with the other variable actions being combined, but each with its specific ψ_0 value.

With only one variable action, expression 6.10a becomes:

$\gamma_G G_k + \gamma_Q \psi_0 Q_k$

Since $\psi < 1.0$, the result from expression 6.10a is lower than that from expression 6.10.

With only one variable action, expression 6.10b becomes:

$\xi \gamma_G G_k + \gamma_Q Q_k$

ξ is given in the UK National Annex as 0.925, so the result from expression 6.10b is lower than that from expression 6.10.

Table 4.3 illustrates the comparison between expressions for a typical set of actions on a composite floor.

The characteristic actions in the example are:

Permanent actions:	Slab:	3.0 kN/m^2
	Ceiling + services + finishes:	0.85 kN/m^2
Variable actions:	Imposed floor load:	2.5 kN/m^2
	Allowance for movable partitions:	0.8 kN/m^2

(It should be noted that BS EN 1991-1-1 does not specify 1.0 kN/m^2 as an allowance for movable partitions, but specifies options of 0.5, 0.8 and 1.2 kN/m^2).

Table 4.3 Design value of actions

Expression	Calculation	Design value (kN/m^2)
6.10	1.35 x (3.0 + 0.85) + 1.5 x (2.5 + 0.8)	10.15
6.10a	1.35 x (3.0 + 0.85) + 1.5 x 0.7 x (2.5 + 0.8)	8.66
6.10b	0.925 x 1.35 x (3.0 + 0.85) + 1.5 x (2.5 + 0.8)	9.76

In this particular example, the design value of actions has reduced by 4% by the use of expression 6.10a and 6.10b. The reduction increases as the permanent action increases. In typical office buildings, expression 6.10b will generally be found to give the value to be used in design, being the more onerous of the two expressions.

4.4 Design for Serviceability Limit State

Verification at SLS includes the assessment of deflections and the dynamic response of floors.

4.4.1 Deflections

In the UK, the National Annex to BS EN 1993-1-1 states that the characteristic combination defined in clause 6.5.3 of BS EN 1990 should be used and that permanent actions should not be included in that combination. The result is that deflections should be verified under unfactored variable actions, which has been practice in the UK for many years.

If total deflections are to be considered, members can be precambered to offset the deflections due to the permanent actions. Precambering of composite beams may be used to counteract the deflection of composite beams at the construction (wet concrete) stage. Recommended practice is that precambering should only be specified for beams over 8 m in length and a minimum precamber of 25 mm, although smaller cambers can be provided. As a rule of thumb, the specified precamber is often calculated as $\frac{2}{3}$ to $\frac{3}{4}$ of the deflection due to the permanent actions. This recognises that even nominally pinned connections have some stiffness and the beam deflections are likely to be less than those calculated for a perfectly pin-ended member.

Within the National Annex, Table NA.2 gives limits for vertical deflections, but these are carefully described as *suggested* limits. Thus, there is no obligation to blindly adhere to the tabulated values – there may be situations when different values are appropriate.

Similarly, Table NA.3 gives *suggested* limits for horizontal deflections.

If deflection limits other than those tabulated in the UK National Annex are to be used, these should be discussed and agreed before the design is undertaken. The impact of potentially increased deflections on other components (for example glazing) should be carefully considered.

4.4.2 Verification of beam deflections

The deflection of a beam is generally calculated assuming a perfectly pinned end connection, even if the beam is composite and clearly has significant continuity. Even nominally pinned joints have some stiffness, which will reduce the deflection. Previous work on semi-continuous joints[46] has shown that relatively low joint stiffness will reduce the midspan deflection considerably.

Figure 4.10 shows the coefficient β used in deflection calculations for a simply supported beam. For a perfectly pin ended beam the value of β is 5, leading to the standard expression. The “support” stiffness is a combination of the stiffness of the joint and the stiffness of the supporting members – typically a column. The figure is included here simply to demonstrate that modest stiffness leads to a considerable reduction in midspan deflection.

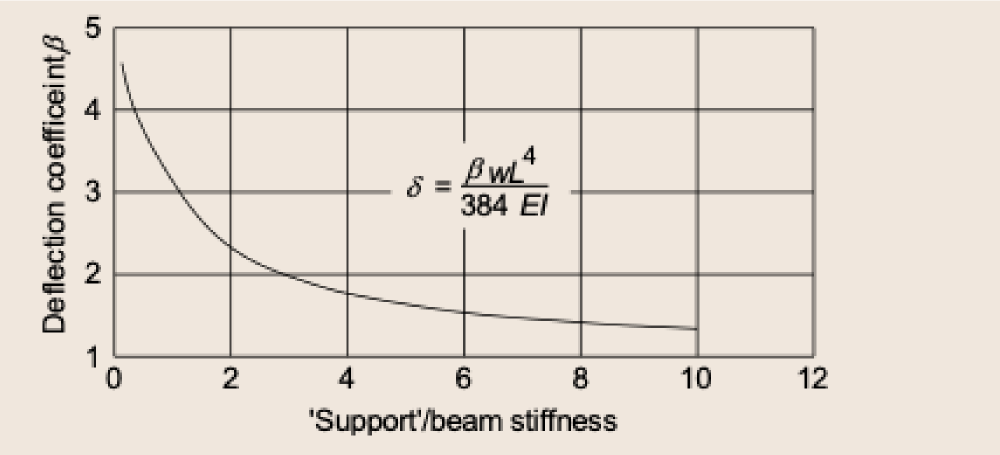


Figure 4.10 Deflection coefficent β as a function of “support” stiffness

Although practice is to verify beam deflections based on the unfactored variable actions, this does represent an extreme case. A more realistic assessment could be to reflect the “frequent” combination of actions, given by expression 6.15b in BS EN 1990. Within this expression, the variable actions are reduced by the application of a ψ_1 factor. The ψ_1 factor for office areas, as given by Table NA.A1.1 in the UK National Annex to BS EN 1991-1-1 is 0.5 (0.7 for congregation and shopping areas). Adopting this approach would lead to a significant reduction in the calculated deflection – but would conflict with the requirements of the current National Annex.

The vertical deflection limit widely used in practice is span/360. This suggested limit in Table NA.2 of the UK National Annex to BS EN 1993-1-1 is for "Beams carrying plaster or other brittle finish". Best practice is for the project team to carefully assess the loading conditions and deflection limits and the impact of deflection on features such as partitions. An absolute deflection limit of 25 mm is often specified, being the typical movement which can be accommodated by the details at the heads of internal partitions. The cumulative deflection of secondary beams supported by primary beams may need to be considered, especially if features such as partitions are to be located diagonally with respect to an orthogonal beam grid.

Although not codified in UK standards, some designers also verify a deflection limit of span/200 or span/250 under *total* loads.

For beams also supporting facades, a more onerous deflection limit is often applied, typically span/1000 and an absolute limit of 10 mm. Early discussion with the façade designers to agree the necessary limits is recommended. More detailed advice can be found in Reference [47].

4.4.3 Dynamic response

In the UK the traditional approach to verify conventional floors for dynamic response has been to ensure the natural frequency of each beam exceeds 4Hz. The check is undertaken by calculating the deflection δ due to self-weight, services, ceiling and 10% of the imposed load, assuming the beam to be simply supported.

The natural frequency f_1 is then given by:

$$f_1 = 18/\sqrt{\delta}$$

The proposed limit minimises the likelihood of resonant excitation occurring when the first harmonic component of the activity coincides with the fundamental frequency of the floor system. However, this limit does not give any indication of the level of floor response in service, resulting in some designs not meeting the acceptability service criteria. Conversely, some designs could be over-conservative. To ensure that vibration serviceability criteria are met, the designer should make realistic predictions of the floor response that will be encountered in service. The floor response may be compared to acceptance criteria which vary depending on the use of the floor. Detailed advice is contained in SCI P354[48]. The methods described in P354 can be conservative in some situations, so a finite element (FE) analysis is recommended. A floor response calculator (based on the results of many FE analyses) is available on www.steelconstruction.info.

Figure 4.11 shows the results of a dynamic analysis of a floor plate constructed from 15 m span beams with web openings. The image shows the response factor in different areas of the model.

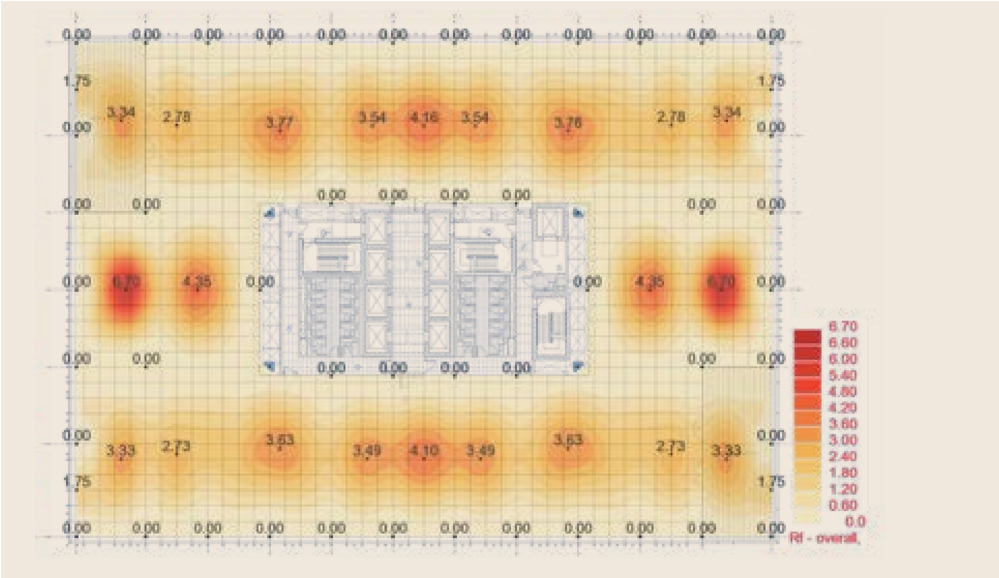


Figure 4.11 Graphical results of FE analysis of dynamic response

The approach to footfall vibration analysis utilising response factors based on finite element analysis is rigorous but also conservative, since is based on response factor limits for continuous vibration. In the vast majority of cases, vibration resulting from footfall will in be intermittent in nature. The vibration dose value (VDV) is an alternative approach which can be used to assess the dynamic response of a floor.

A VDV analysis effectively permits the vibration response of a floor to be greater than the prescribed continuous vibration limits, but only for a limited period of time. There is flexibility in the approach, with a VDV being calculated for a particular scenario and compared against published limits corresponding to a 'low probability of adverse comment'. Alternatively, an acceptable limit may be agreed and a limiting number of instances of vibration within the specified exposure period may be back-calculated. For example, if walking in a corridor produces a high response factor elsewhere, the VDV approach leads to the maximum number of times the corridor can be used in a single day before adverse comments are received.

The VDV approach can be a more tangible measure of vibration performance than a response factor, since the approach addresses expected activities on the floor.

The VDV approach is described in the SCI P354 and includes limiting VDV values from BS 6472-1:2008[49].

If the dynamic response of certain areas of the floor is initially unacceptable, measures which may be taken to resolve the problem include:

- Ensuring that all continuity between spans has been included in the model;
- Allowing for damping by non-structural components;
- Identifying the areas of the floor with the unacceptable response. It may be that in consultation with the client, small areas of unacceptable response may be ignored. If necessary, the local area can be managed, preferably by adding secondary beams or by adding more mass (typically a thicker slab);

- Increasing stiffness without increasing steel weight - for example using beams with web openings;
- Considering vibration dose values.

It should be noted that contrary to expectations, long spans generally demonstrate satisfactory dynamic behaviour, due to the significant mass of floor slab which is mobilised.

For similar reasons, a slimfloor system where the beams typically support a large floor area generally demonstrate better dynamic behaviour than arrangements with short, closely spaced lightweight members, when the VDV approach may be helpful.

4.5 Design criteria checklist

1. Have clear spans been designed, facilitating flexible use and future adaptability?
2. Have alternative floor solutions been considered in conjunction with the number of storeys and embodied carbon content of the entire structure, including the façade and foundations?
3. Has the available space been maximised, and thus the building carbon footprint minimised?
4. Has the imposed floor load been selected to suit the current requirements, rather than a possible future change of use?
5. Has an appropriate allowance been made for partitions?
6. Have the pair of expressions 6.10a and 6.10b been used to determine the combined design values of actions?
7. Has full advantage been taken of the reductions factors which may be applied to most variable actions, particularly when calculating the design loads in the lower columns?
8. Has advantage been taken of the reduced buckling length of columns with end fixity?
9. Have the deflections been calculated under an appropriate level of load?
10. Do the members have a brittle finish, or is a less onerous deflection limit appropriate?
11. Rather than the simplified approach to floor dynamics, has the floor been modelled and a response factor determined?
12. Is the response factor appropriate for the use of the structure?
13. Have VDV been considered?

BUILD EFFICIENTLY

Building efficiently could be considered as a combination of a thoughtful structural arrangement and ensuring the individual components do not have significant spare resistance i.e. utilisation factors close to 1.0. This does not mean the required reliability is compromised – partial factors have already been applied to the loads to determine design combinations of actions (ultimate loads). Real loading should never exceed the characteristic actions, and in many cases – such as floor loading – will never reach even the characteristic values assumed.

Studies have shown that spare resistance often remains in designs for non-structural reasons, generally related to design time and fees. If programmes and fees were sufficient to accommodate the inevitable changes during design development and were sufficient to cover finesse in the final design, significant spare resistance and accompanying carbon could be eliminated.

Although the design of every member is important, grid arrangements and member selections which are repeated throughout a storey and possibly on multiple storeys should be prioritised as having the greatest impact.

5.1 Early engagement

Early engagement with steelwork and other contractors to develop a low weight and low carbon solution is strongly recommended. In a very competitive environment, one aspect of a steelwork contractor's competence is the ability and experience to develop the most economical solution, which is generally also the lowest carbon solution. Steelwork contractors can advise on all aspects of steel construction.

5.2 Highly utilised structures

Designers will be immediately familiar with the concept of utilisation ratios, particularly in the design of steel structures. In the following discussion, utilisation refers equally to both resistance and serviceability verifications.

A utilisation ratio is of the form required resistance / actual resistance provided, although generally expressed as design actions / design resistance. There will be a utilisation ratio for the resistance of each member, and a different value for the serviceability assessment (usually deflection) of the form Actual deflection / Deflection limit. A utilisation ratio of 0.99 indicates that the member is highly utilised – a lower value indicates that there is some degree of spare resistance.

The utilisation ratio is not necessarily proportional to the value of the design actions – for example a utilisation ratio of 0.8 does not necessarily mean that the actions can be increased by 25% or the resistance reduced by 20%, but a value lower than 1.0 means that there is some spare resistance. It may be possible to reduce the material content of the structure, thus reducing the embodied carbon content.

5.2.1 Utilisation ratios in practice

In 2014 Moynihan and Allwood concluded that in the steel framed structures they studied (which were multi-storey structures, not single storey buildings), the average utilisation was less than 50% of capacity[50]. A further conclusion from that research was to note the potential to save 36% of the steel weight – which would immediately exceed the industry's target reduction in embodied carbon of 17.5%[3] by design efficiency by a considerable margin. Figure 5.1 shows a typical floor plot from Reference [50] – only the red lines indicate members with a utilisation of at least 0.75.

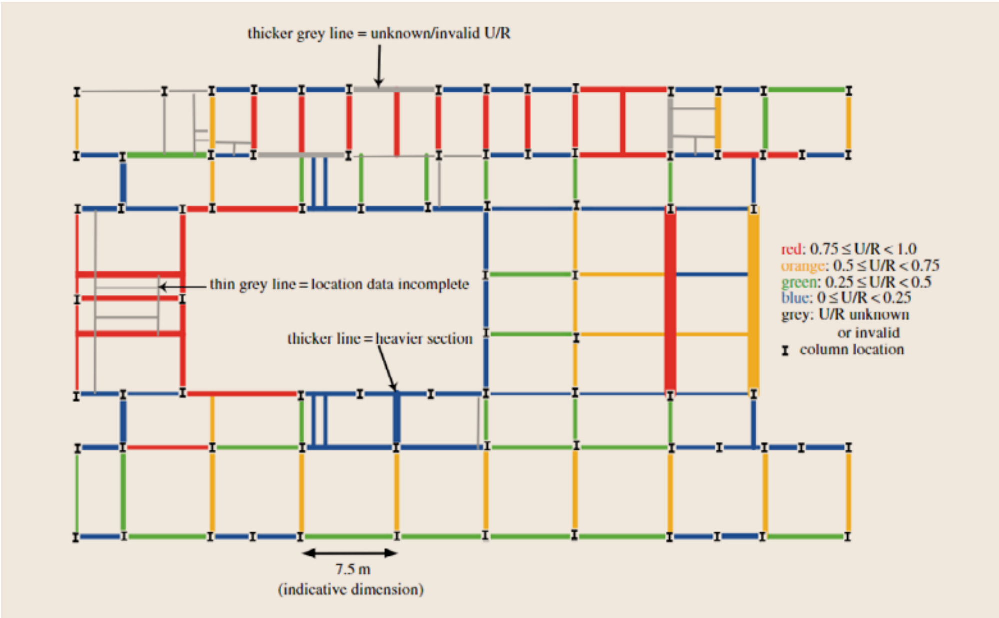


Figure 5.1 Typical floor beam utilisations (from Reference [51])

Some reasons for the apparent inefficient design were noted in the 2014 study, including:

- Rationalisation – using the same steel section when a lighter section would suffice. Rationalisation can be applicable both to the design process and the fabrication process,
- Insufficient programme time to redesign the structural frame more efficiently when the contract was agreed, at a point after the initial design,
- A view that changes were inevitable, so some allowance (spare resistance) was always necessary,
- Poor grid layout.

A subsequent study from 2018[51] noted that utilisations were often deliberately limited to 0.8, as an allowance for changes during the design project. This second study noted that lighter steel sections could not necessarily be selected for certain members, such as those required for stability or where member selection was governed by size constraints. the study still concluded that limiting the utilisation ratio to 0.8 meant at least 20% of the mass of the steel was not necessary for safety or service. As observed previously, the utilisation ratio is not always directly proportional to the design actions, but the general point that under-utilisation means that excess material is used was well-made.

The first Meicon report[52] identified a number of other issues which militated against material efficiency in design, including:

- Ease of construction is more highly valued than material efficiency
- Material efficiency was not requested by clients (this may be changing in 2024, when clients increasingly ask for a low embodied carbon solution)
- There are no penalties borne by the designer for a conservative design.

5.2.2 Target utilisation ratios

There is no structural reason why the utilisation ratio should not be as high as 1.0. In most cases, the discreet sizes of steel members will mean that some modest over-provision is generated.

The primary reasons for under-utilisation were found to be an allowance for changes and insufficient time for finesse in the design once the construction contract was agreed. A client wishing to demonstrate best practice with minimum embodied carbon in the final solution should recognise that additional design time might be necessary[52]. As suggested in Section 4, development of the design throughout the stages of work is recommended.

Anecdotal evidence suggests that steel weight – and therefore embodied carbon - can often be reduced when designs are developed in collaboration with steelwork contractors. The main principles are to:

- Consider alternative grid arrangements;
- Minimise the number of members – for example by designing longer composite slab spans to remove secondary members;
- Focus on efficient solutions for members which repeat throughout the project;
- Design non-repeating features – such as wider bays in an otherwise regular grid – as unique solutions, rather than the selected sizes being adopted for the entire floor grid.
- Consider alternatives to solutions involving significant fabrication effort;
- Select members which are highly utilised.

If all steel members in a project were designed to the highest possible utilisation ratio, the result *could* be a collection of unique member sizes and unique connections, reducing the efficiency of fabrication and increasing the cost of the structure. This will not happen in practice, as buildings have repeating identical arrangements of members. The repeating nature of most buildings make it even more important to design the typical arrangements to the highest possible utilisation.

5.3 Rationalisation

Rationalisation is the selection of members, and construction details such as connections to reduce variation. Reasons for rationalisation include:

- reduced member design effort;
- reduced joint design effort;
- economies of scale, particularly when buying the steel members – small tonnages cost a premium;
- availability of less common steelwork sizes;
- aesthetic appearance if exposed;
- ease of routing services;
- efficiency in fabrication.

The inevitable outcome of rationalisation is that some members and joints may be under-utilised, increasing weight and embodied carbon.

Most reasonably sized steelwork contractors operate numerically controlled equipment which can readily manage diverse elements, meaning rationalisation is not an important priority. Small tonnages of certain sections (those which are infrequently rolled) can be difficult and more expensive to obtain. Advice on section procurement can be obtained from Steelwork Contractors. The Blue Book[53] indicates section sizes which are readily available.

A less rationalised design will inevitably demand more design input, but Poole[54] asserts that designers have the tools available, that changes can be quickly re-analysed without manual intervention. Poole suggests that the benefits of rationalisation to the designer are minimal, provided there is reasonable allowance of time in the programme. Furthermore, the designer should have the health and safety of the global population in mind – that wasting any material should be avoided.

5.4 Repetitive members

Members which repeat – for example floor beams used in a regular grid and on multiple floors - should be optimised. Increased design effort to reduce beam weight and cost (for example utilising the benefits of connection stiffness, considering asymmetric sections, assessing multiple options) can have considerable benefit for members which repeat multiple times. A typical floor solution is a mix of grid arrangement, slab, decking, shear studs and steel beam – which may be rolled, fabricated and include web openings, - which should all be considered in order to develop an ideal solution. A small reduction in individual component weight can result in a considerable saving when accumulated over multiple floors.

5.5 Minimum weight

Selecting the lowest weight solution which meets the design requirements is traditionally an imperative (as lowest weight is thought to mean lowest cost) and could be considered to be even more important if the measure is reduced embodied carbon. Lowest member weight is often an option within structural design software, so can be an easy choice.

Lowest member weight is not always lowest cost if the joints require excessive reinforcement. Joints where (for example) the plastic resistance of the full cross section is utilised yet must be drilled and material removed for a bolted joint, are clearly inappropriate. Selecting the lowest member weight will result in the lowest embodied carbon, unless additional reinforcement adds more weight than simply selecting a heavier member. Steelwork contractors are able to advise on joint design, cost and embodied carbon.

5.6 Carbon-efficient steel frame solutions

UK practice is characterised by simply supported beams with nominally pinned joints to columns. Even in composite construction, where the floor slab provides significant continuity across supports, verification generally assumes simple supports.

Resistance to lateral forces is generally provided by discreet vertical bracing systems for low rise structures, and for high rise structures a concrete core. These arrangements facilitate the assumption that the beams are simply supported and means most columns are primarily designed for axial load, simplifying design.

For carbon efficiency, it should be noted that there is typically three times as much steel weight in the floor systems than in the vertical columns. A carbon-efficient solution is characterised by highly utilised floor steelwork, using a variety of member sizes to suit the spans rather than a repeating design based only on the most onerous design situation.

In general terms, a more carbon-efficient solution will be achieved by:

- ensuring the steelwork is highly utilised, designed for the span;
- using members without redundant capacity;
- using higher strength materials where appropriate, such as columns and transfer structures;
- paying special attention to the floor construction, which contains the largest proportion of embodied carbon in the superstructure;

- replacing concrete with mass timber;
- utilising some degree of continuity at the joints.

5.7 Continuous beams

The benefits of continuity are:

- smaller, lighter beams with reduced embodied carbon, compared to a simply supported non-composite member
- reduced deflections (compared to simply supported beams)

The joints must be moment-resisting, so are more expensive, and require increased design effort. Internal bending moments in columns are larger than those which are assumed from nominally pinned joints, meaning the column section size may increase.

Due to the necessary moment-resisting joints, continuous beams are best arranged on one or other axes of the supporting columns, but not in both directions. Moment connections may be aligned with the major axis of the column, as shown in Figure 5.2. Joints of this form may be designed in accordance with BS EN 1993-1-8[55], which includes extensive guidance, re-presented in SCI publication P398[56].

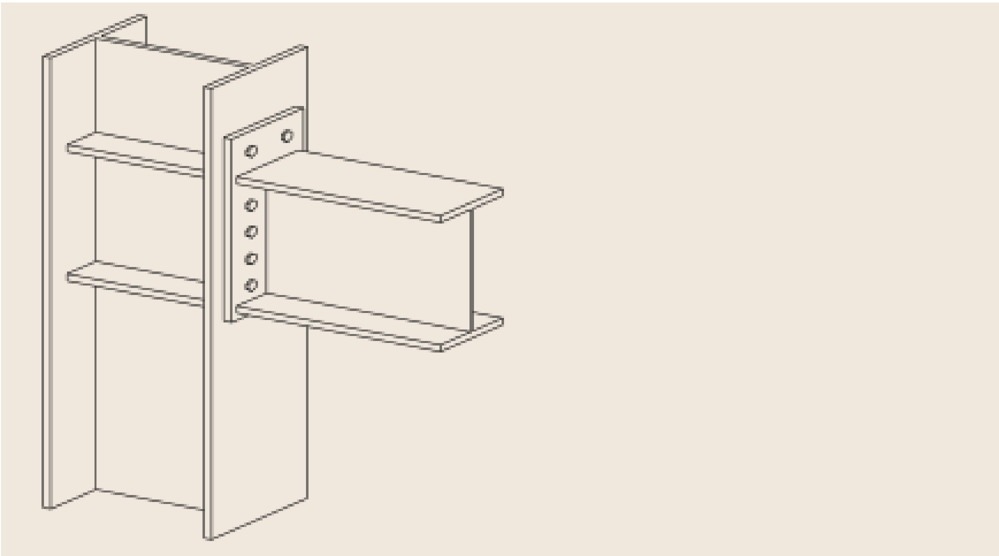


Figure 5.2 Typical major axis moment resisting joint

It should be noted that the column is likely to need at least some reinforcement, which may make connections to the minor axis more involved. If a detail such as Figure 5.2 is adopted, out of balance moments (which must be considered) will be transferred to the column. To avoid significant column moments, nominally pinned connections should be considered where beams are connected to one side only of façade columns. Pattern loading as shown in Figure 5.3, which is not considered for braced frames, must be considered for continuous construction.

If the continuity is also used to provide frame stability and to resist lateral loads, the number of load combinations will increase significantly.

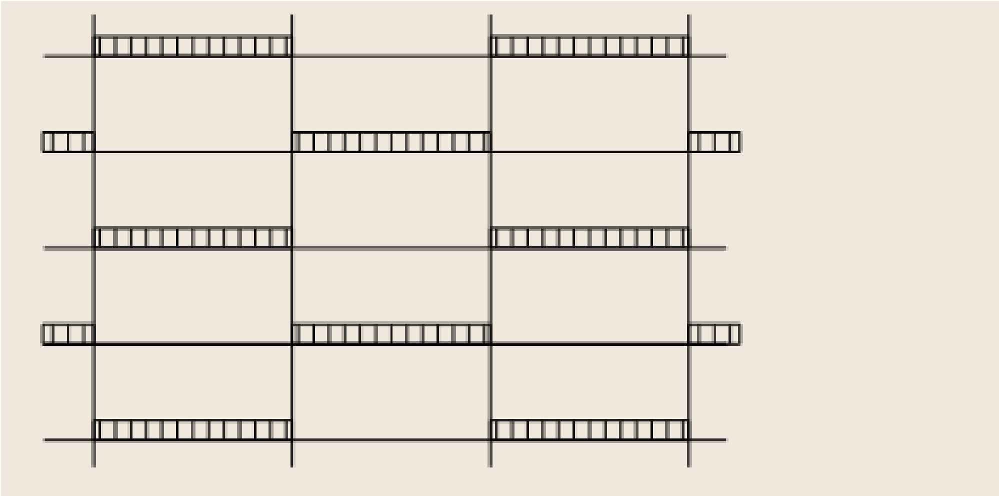


Figure 5.3
Pattern loading

In simple construction with nominally pinned joints, it is usual to conclude that the top flange in compression is continuously laterally restrained by the floor construction. In continuous construction, hogging moments at the supports are expected, which needs an additional design check. In these zones the bottom flange is in compression and is unrestrained, as shown in Figure 5.4. If the beam is composite with profiled steel decking, advice is given BS EN 1994-1-1[57]. In other circumstances, the complete beam may be modelled in software such as *LTBeam*[58] to determine the value of M_{cr}.

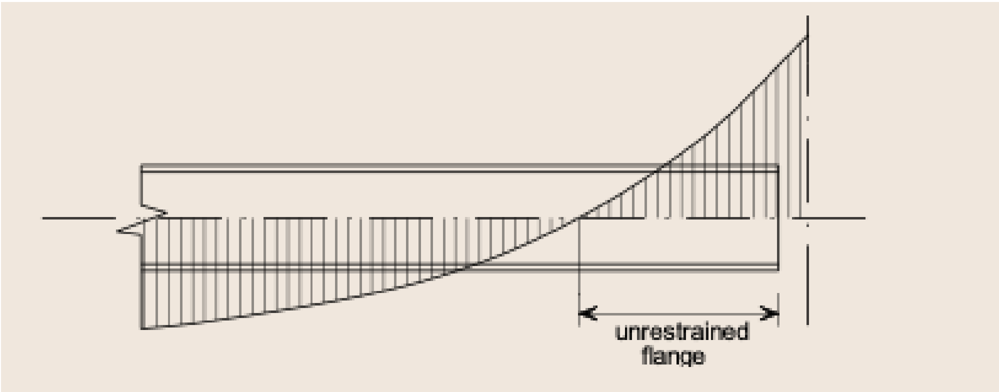


Figure 5.4
Unrestrained bottom flange in continuous construction

Deflections cannot readily be calculated manually but will be obtained from the frame analysis.

The design process will involve some iteration, as distribution of forces and moments and the deflections depend on the stiffness of the selected sections, which may change when the member resistances are verified for the applied forces and moments.

Within continuous structures, column design is considerably more involved than the simple rules for columns in braced frames[59]. Column verification will involve the use of expressions 6.61 and 6.62 in BS EN 1993-1-1, so the use of design software is recommended.

5.7.1 Parallel beam approach

A structural arrangement which utilises the benefits of continuity is known as the parallel beam approach. In this arrangement, a continuous beam is connected on each side of the supporting column to form a pair of beams. A second pair of beams is located above the first pair in the orthogonal direction, again passing each side of the supporting column, forming a grillage. The general arrangement is shown in Figure 5.5.

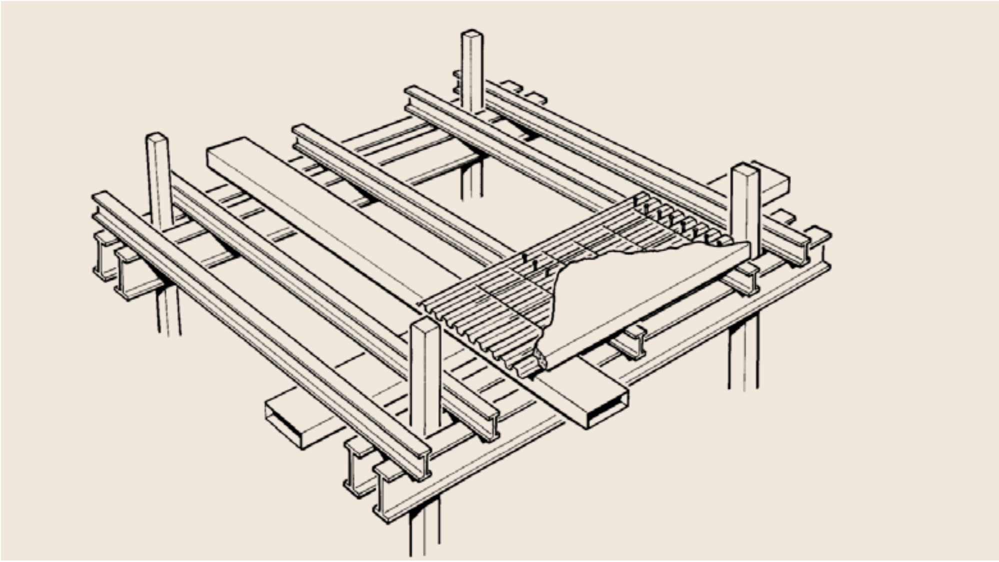


Figure 5.5 Parralel beam approach

The separated layers of steelwork in the parallel beam approach facilitate services running in either direction, whilst the dual beam arrangement results in shallow members. The principles of the design method are comprehensively described in Reference [60].

5.8 Semi-continuous braced frames with partial strength connections

The use of semi-continuous connections in braced frames facilitates many of the advantages of continuous design, without the penalties of increased column sizes. The approach is directly applicable to non-composite construction, including precast concrete planks and CLT floor slabs. The principles may be extended for use with frames including composite beams.

The design of semi-continuous braced frames requires only modest additional effort compared to assuming the beams are simply supported, particularly when standard connections are used. If the beams are non-composite and the frame is braced, designers are encouraged to adopt semi-continuous design. Detailed design guidance is presented in SCI Publication P183[61] showing savings in beam weight of between 10 and 30%. Although P183 was written for design to BS 5950[62], the principles remain appropriate for design in accordance with the Eurocodes[63].

Semi-continuous connections may be used to facilitate a significant reduction in beam weight compared to a simply supported member. Under gravity loading, the hogging moment at one or both supports reduces the required moment resistance of the beam. The additional deflection expected when a smaller beam is selected is offset by the continuity at the support.

The design approaches described in this section are only appropriate for frames where lateral stability is provided by some other means – typically braced bays or a core. Use of semi-continuous connections in unbraced frames is outside this current guidance and not recommended due to the design complexity when connections load and unload.

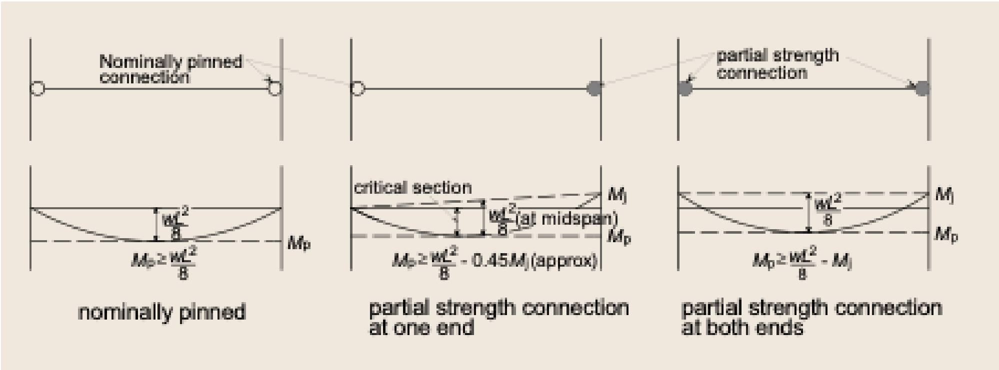
Since the selection of the beam is inextricably linked to the specified connection, the frame designer must be responsible for both aspects of design.

The semi-continuous design method presented in P183 was developed for simple frame layouts where the beams are on an orthogonal grid.

5.8.1 Overview of the method

Semi-continuous design in braced frames is an elegantly simple approach. Instead of designing a beam for the free bending moment which assumes pinned ends, the moment resistance of the end connections is used to reduce the maximum sagging moment, as illustrated in Figure 5.6.

Figure 5.6 Design of beams in semi-continuous braced frames



Since the maximum sagging moment is reduced, a smaller or lighter (or both) beam may be specified. The larger deflections resulting from the selection of a smaller member are offset by the stiffness of the connections.

Column design is straightforward. In contrast to “simple construction”, involving nominal moments arising from assumed eccentricity at connections, columns are verified for the known moments applied by the beams. The moments applied by the beams are equal to the moment resistances of the connections. Since the connections only have modest resistance, the column is generally no larger than the equivalent in “simple construction” and generally requires no additional fabrication effort associated with local strengthening.

Connections in semi-continuous braced frames are typically of the form shown in Figure 5.7 and may have an extended end plate or a flush end plate (as drawn).

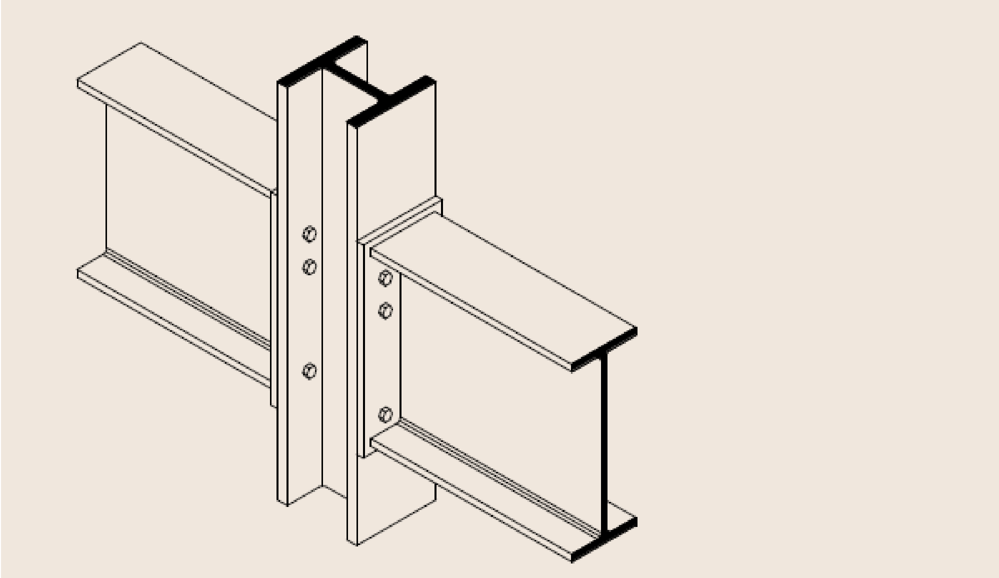


Figure 5.7 Semi-continuous connection – flush end plate

The connections are particularly important and integral to the design approach for the frame. The standard connections have critically important characteristics which have been verified by physical testing[64]. The details in P183 prescribe end plates in S275 steel, which is increasingly difficult to source. Section 5.8.3 recommends how the detailing of these standard connections should be amended to reflect the use of higher strength plates and columns.

Idealised essential joint characteristics are illustrated in Figure 5.8. The joints have:

- A known stiffness, which is utilised when determining deflections. Using Eurocode terminology the joints are classified as semi-rigid.
- A known limited resistance, so that moments transferred to the column are controlled. Using Eurocode terminology, the joints are classified as partial-strength.
- A guaranteed rotation capacity before failure, so that after reaching the joint resistance, the joint will rotate as a plastic hinge. The joints are ductile.

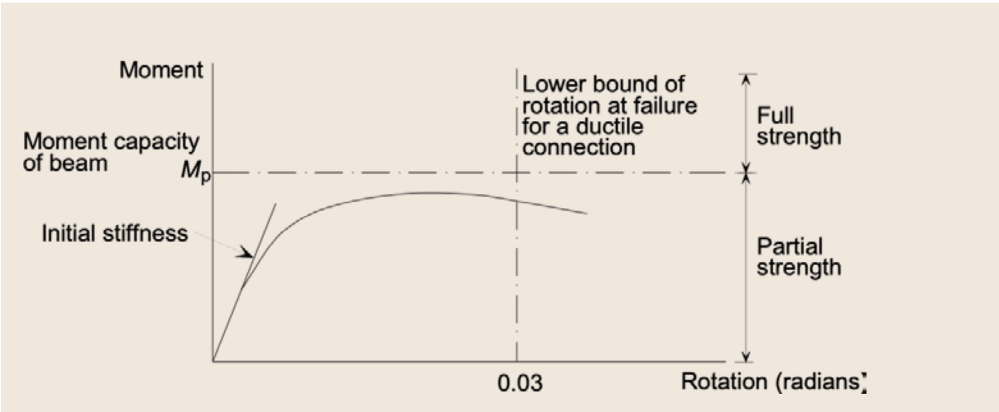


Figure 5.8 Joint characteristics in semi-continuous braced frames

5.8.2 Eurocode links

The design approach described in Section 5.8.1 is accommodated by the Eurocodes. Table 5.1 of BS EN 1993-1-8[55] presents a matrix of joint models, methods of global analysis and joint classification. As described in Section 5.8.1, the joints are semi-rigid and partial-strength. According to Table 5.1, the joint model is semi-continuous, and the method of global analysis is "Elastic-plastic".

The analysis assumes elastic behaviour, with plastic hinges at the joints, appropriately described as "Elastic-plastic". The benefit of semi-continuous design are utilised in reducing beam size and the beam deflections. Coefficients to calculate beam deflections are given in P183. Typical end plate connections on beams subject to a UDL reduce the deflection by approximately 45% compared to a perfectly pinned connection.

5.8.3 Joint resistance

P183 contains tables of "beam side" and "column side" resistances, which are combined to determine the resistance of the joint. The tables were based on the Eurocode design model for bolted moment-resisting joints but used so-called component resistances determined in accordance with BS 5950. As component resistances are hardly different between BS 5950 and BS EN 1993-1-8, the joint resistance will be almost identical, meaning the tabulated values may still be used for designs according to BS EN 1993 if the steel grades remain as prescribed in the standard details.

The details of the standard joints ensure that ductile behaviour is facilitated by the deformation of the relatively thin end plate. Neither the bolts nor welds are critical components, as they could fail in a non-ductile manner. In almost all cases, the resistance of the joint is limited by the "beam side" of the joint.

If column sections are changed to S460, as recommended in other sections of this publication, the joint resistance is generally not affected, provided the "beam side" is the weak link.

If S355 end plates are substituted for the S275 recommended in P183, (which is simply a reflection of the age of the document) the failure mode *could* change and become the bolts. To avoid this, it is recommended that if S355 end plates are used:

- 12 mm S275 end plates should be replaced by 10 mm S355;
- 15 mm S275 end plates should be replaced by 12 mm S355.

The substitution of thinner plates should ensure that the end plate remains the "weak link" in the joint, rather than the bolts.

Substituting thinner plates in S355 rather than the standard S275 will modify the joint resistance, so the resistance should be verified by calculation rather than using the tabulated values in P183. The stiffness of the joint may also change, so the reduced coefficients should be recalculated in accordance with Annex B.2 of P183.

For convenience, it is recommended that properly verified software is used to determine the joint resistance. The geometry of the standardised details should be followed. The design output should be investigated to ensure that the yielding of the end plate is the critical component check, so that ductile behaviour is ensured.

If double sided joints are modelled as separate single sided joints, in some cases the joint resistance may be limited by the resistance of the web panel in shear. If the joint is in reality double sided, the column web will only be subject to the net shear, or zero net shear if the joints are balanced.

5.9 Semi-continuous joints

An alternative to using the semi-continuous partial strength joints described in Section 5.8 is to use semi-continuous joints without the partial strength characteristic. In this case, the selection of beams, columns and joints is all part of the same process, so some iteration will be required to arrive at a solution. The frame design and joint design should be completed concurrently. Using software which integrates frame design and joint design will facilitate an efficient design process.

5.9.1 Overview of the method

In the following summary design steps, it is assumed that the frame is stabilised by bracing, or a core, or a dedicated continuous frame (i.e. the beams, columns and joints do not contribute to the stability of the frame).

5.9.1.1 Initial analysis

An elastic analysis of the frame should be completed with estimated sizes of beams and columns. The joints between beams and columns should be modelled with an appropriate spring stiffness. It may be convenient to examine the stiffness of typical beam to column joints, so that a reasonable initial estimate of the joint stiffness can be included in the frame analysis.

5.9.1.2 Verification of joint resistance

Having obtained the forces and bending moments at each joint, the joints must be configured and the resistances established. This calculation may be completed by hand, but software is recommended.

5.9.1.3 Verification of joint stiffness

The joint stiffness must be determined. A method is provided in BS EN 1993-1-8. The method is laborious, so the use of properly verified software is recommended. Not all joint design software has the facility to calculate joint stiffness.

It should be noted that the calculated initial stiffness $S_{j,ini}$ is only appropriate whilst the applied moment does not exceed 2/3 of the moment resistance of the joint $M_{j,Rd}$, as shown in Figure 5.9. If the applied moment is close to the moment resistance, the stiffness is reduced considerably.

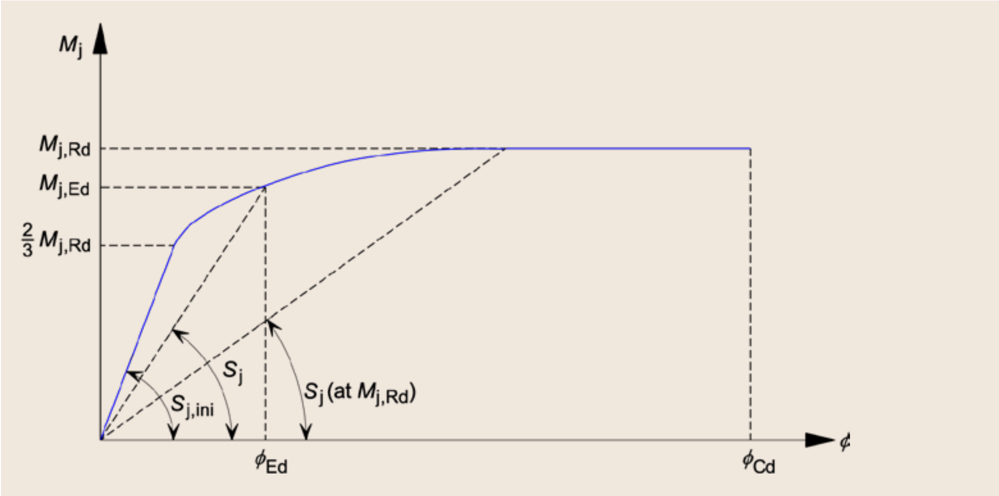


Figure 5.9 Variation of joint stiffness

5.9.1.4 Beam verification

The beam should be verified in the normal way, at ULS and SLS. The effect of combined shear and moment should be considered at the ends of the beams, where both the shear force and the bending moment may be at their maximum values.

5.9.1.5 Column verification

The column should be verified in accordance with expressions 6.61 and 6.62 of BS EN 1993-1-1. A design tool[65] is available on www.steelconstruction.info which is appropriate for members in S355 steel. The design moments in the columns are not limited by partial strength joints, so may be larger than when using partial strength joints.

5.9.1.6 Revise the arrangement and repeat the process

It is unlikely that the initial selection of beams, columns and joints will be ideal. Modifications to the joints will change their stiffness, which together with modified beams and columns demand a new analysis and a repeat of the entire process. Increasing the joint resistance (perhaps to ensure the applied moment is no more than 2/3 of the resistance) will invariably increase the stiffness and therefore modify the frame analysis.

5.10 Carbon-efficient member form

The design bending moment in simply supported beams reduces towards the supports, whilst the design shear load increases. In terms of resistance, regular profiles are inefficient, providing excess bending resistance for much of the span. Shear resistance is generally not a critical verification unless concentrated loads are applied close to the supports. A number of solutions are available which use material efficiently by varying the member properties along the member length, though it should be recognised that fabrication effort is increased.

5.10.1 Welded asymmetric members

In the UK, asymmetric members are typically fabricated either from plate, or from rolled sections which are cut and re-welded to form an asymmetric section, as shown in Figure 5.10. Asymmetry can be particularly beneficial in composite construction where the top flange of the steel member makes only a modest contribution to the resistance once the concrete has cured.

Fabricated from plate

Fabricated from rolled sections

Figure 5.10 Asymmentric members

With both forms of fabricated member, services are often passed through openings in the web. In members fabricated from plate, openings may be cut in a variety of shapes, as required. In members fabricated from sections, openings are usually regular and circular, but may be elongated to form larger openings for services.

Typically, the lightweight solutions possible with fabricated members means that they are used in association with long spans, providing flexible column-free space. Beams with large web openings formed from cut and rewelded rolled sections or fabricated from plate are generally the lightest weight, lowest embodied carbon steel solution.

Consistent comparisons between standard rolled sections and fabricated sections are difficult, since rolled sections are available only in standard sections – much more flexibility in design is possible using plate, or by using parts of different rolled sections rewelded into an asymmetric member. Compared to standard rolled sections, fabricated members with large web openings demonstrate a reduction in weight generally in excess of 25% and in some cases as much as 45%.

Both types of fabricated member with large web openings are designed using bespoke software[66],[67]. Design guidance is provided in SCI publication P355[37].

Beams with large web openings allow the passage of services within the depth of the beam, leading to a reduction in construction depth compared to standard rolled sections.

5.10.2 Hybrid fabricated girders

Hybrid fabricated girders are those where the flanges are of higher strength than the web, recognising the greater demands for flange strength (to resist bending) than the web, which primarily resists shear. Higher strength material for the flanges reduces the overall weight (and embodied carbon) of the member.

The design of a hybrid member is hardly more involved than that for a member of the same steel grade throughout. As plate girders tend to be deep to improve their bending resistance, the members often have a Class 4 cross section and the design is completed in accordance with BS EN 1993-1-5[68]. Guidance on the design of hybrid girders has been presented in *New Steel Construction*[69].

5.10.3 Web-tapered members

Web tapered members as shown in Figure 5.11 are fabricated from plate, with the depth of the member broadly following the form of the bending moment diagram. The straight taper is generally preferred for ease of fabrication and the opportunity to nest the web plates. Two halves of the web may be butt welded at midspan.

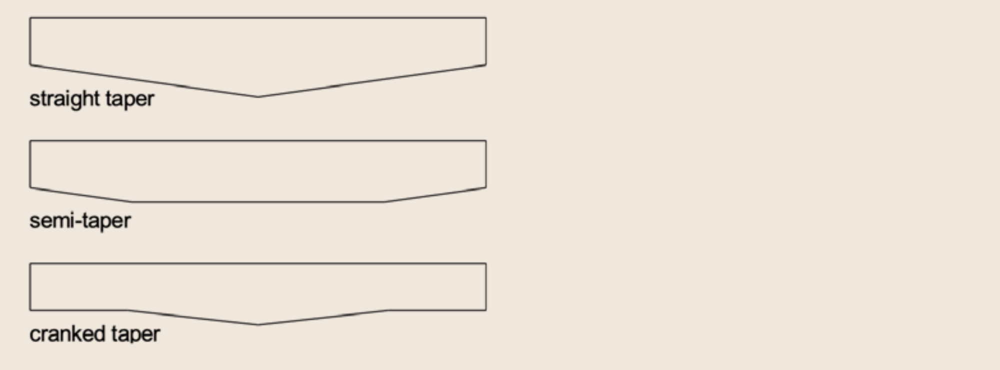


Figure 5.11 Web-tapered beams

Design principles for tapered composite beams are given in SCI publication P059[70]. Restrained members must be verified at different cross sections along the member. The temporary (unrestrained) situation must be verified – software such as *LTBeamN*[58] may be used to determine M_{cr} for a doubly tapered member as shown in Figure 5.11. Deflection may be assessed by modelling the beam as a number of intermediate members with differing section properties. At locations where the flanges change direction, the web must be verified for local crushing – P059 and Reference [71] offers advice.

Webs should generally be a minimum of 8 mm and the h_w / t_w ratio no more than approximately 70. Although more slender webs may be designed, the member becomes difficult to manipulate during fabrication.

Flanges should be a minimum of 8 mm thick if shear studs are to be through deck welded on site. The maximum width to thickness ratio c/t should be limited to 14ε (see Table 5.2 of BS EN 1993-1-1 for classification limits and definition of ε).

5.10.4 Stepped members

An alternative to the tapered and cranked members described in Section 5.10.3 is to form a stepped member from rolled sections (Figure 5.12), reducing the section to reflect reduced design effects.

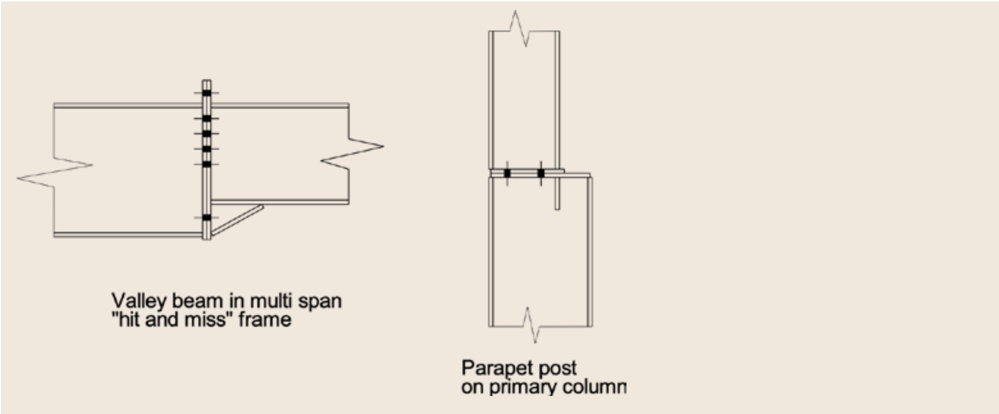


Figure 5.12 Examples of stepped member

5.11 High strength steel

In the following sections, "high strength" steel refers to S460. Higher strength grades are available, but not in common use in construction.

5.11.1 High strength steel columns

The use of S460 steel for columns in multi storey buildings offers an immediate advantage economically, structurally and in terms of reduced carbon, since the embodied carbon is the same as lower strength grades.

S460 is becoming more readily available in the UK (2024). S460 typically costs around 10% more than S355 but delivers increased resistance up to a maximum of around 40%, meaning smaller, lighter sections may be specified saving cost and carbon. Smaller section sizes and weights may not be readily available, so Steelwork Contractors should be consulted for advice.

Appendix C.2 presents column serial sizes chosen for a typical 10 storey building. Assuming the selected serial sizes are readily available, the specification of S460 reduces the total weight of each column running full height of the structure by 25%. In a typical multi-storey building, columns account for around 10-20% of the total steel tonnage.

The design advantage for S460 comes from both the increased strength and the selection of buckling curves in BS EN 1993-1-1. When designing with S460 steel, a more advantageous buckling curve is prescribed by Table 5.2 of the standard. The minor axis buckling curve for UC sections, which is usually critical in design, improves from curve "c" for S355 to curve "a" for S460. In BS EN 1993-1-1:2022, which is due to come into effect in 2028, the relevant minor axis buckling curve is "b" for S460, which reduces the design advantage.

The ratio between the flexural buckling resistance of a UC section in S460 compared to S355 is shown in Figure 5.13 for typical column lengths used in multi-storey buildings. The figure illustrates the advantage of using a higher buckling curve for higher strength sections – the increased resistance is greater than the simple 30% effect of increased strength. Figure 5.13 also shows the calculated column resistance when the 2022 version of BS EN 1993-1-1 is implemented in 2028.

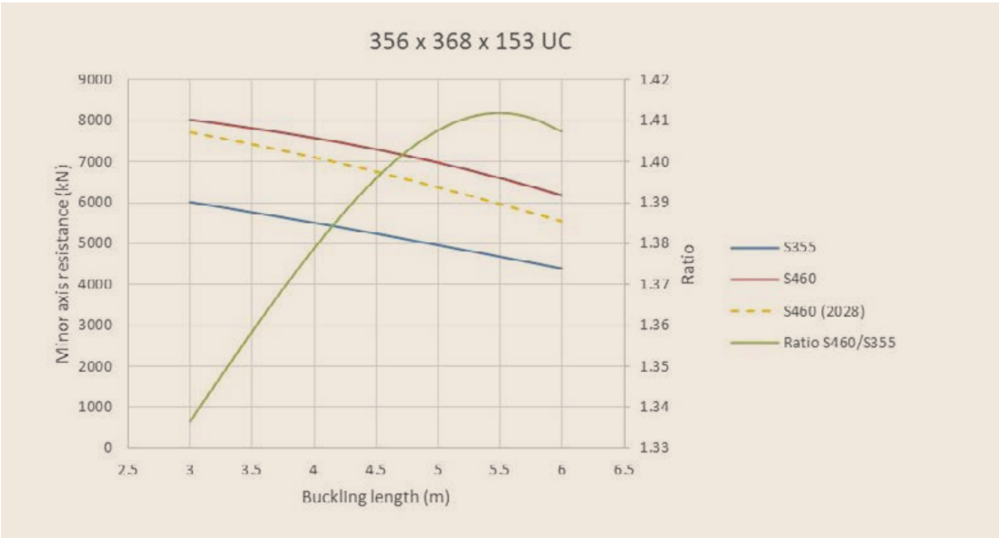


Figure 5.13 Comparative flexural resistance – 356 x 368 x 153 UC (minor axis)

When designing columns, the use of buckling lengths that reflect the restraint conditions at each end of the member is recommended. Although BS EN 1993-1-1 does not give advice, effective length factors are presented in Table 22 of BS 5950 which are appropriate for columns in multi-storey braced frames.

5.11.2 High strength steel beams

For fully restrained beams, any advantage of using S460 steel is simply due to the 30% increase in strength – a smaller beam with less embodied carbon may be selected, provided deflection or the dynamic response does not then become critical.

If a smaller section is selected, the deflection may become critical.

If beams are unrestrained, the advantage of using S460 is not so pronounced and diminishes as the unrestrained length increases – elastic critical buckling, which is independent of strength, becomes dominant.

5.12 Design criteria checklist

1. Has the target utilisation for members been set at 1.0? If not, why not?
2. Have the utilisation ratios for all members in the structure been reviewed?

3. Has the member with the most onerous design condition been designed and repeated throughout? Or has design effort been expended designing members for their individual circumstances?
4. Have repeating members been optimised?
5. Have the likely joint details been considered, and a preliminary design undertaken to establish if a reasonable joint is feasible?
6. Have the advantages of column free space been recognised?
7. If the frame is braced, has benefit been taken of semi-continuous, partial strength joints? If not, why not?
8. Has the use of asymmetric or tapering fabricated members been considered?
9. Have column buckling lengths which reflect the end conditions been used?
10. Has S460 been considered for the columns? If not, why not?


HARE

MINIMISE WASTE

This primary design responsibility is to avoid overdesign, which wastes material. Designers also have the opportunity to reduce future waste, by facilitating future reuse, discussed in Section 10.

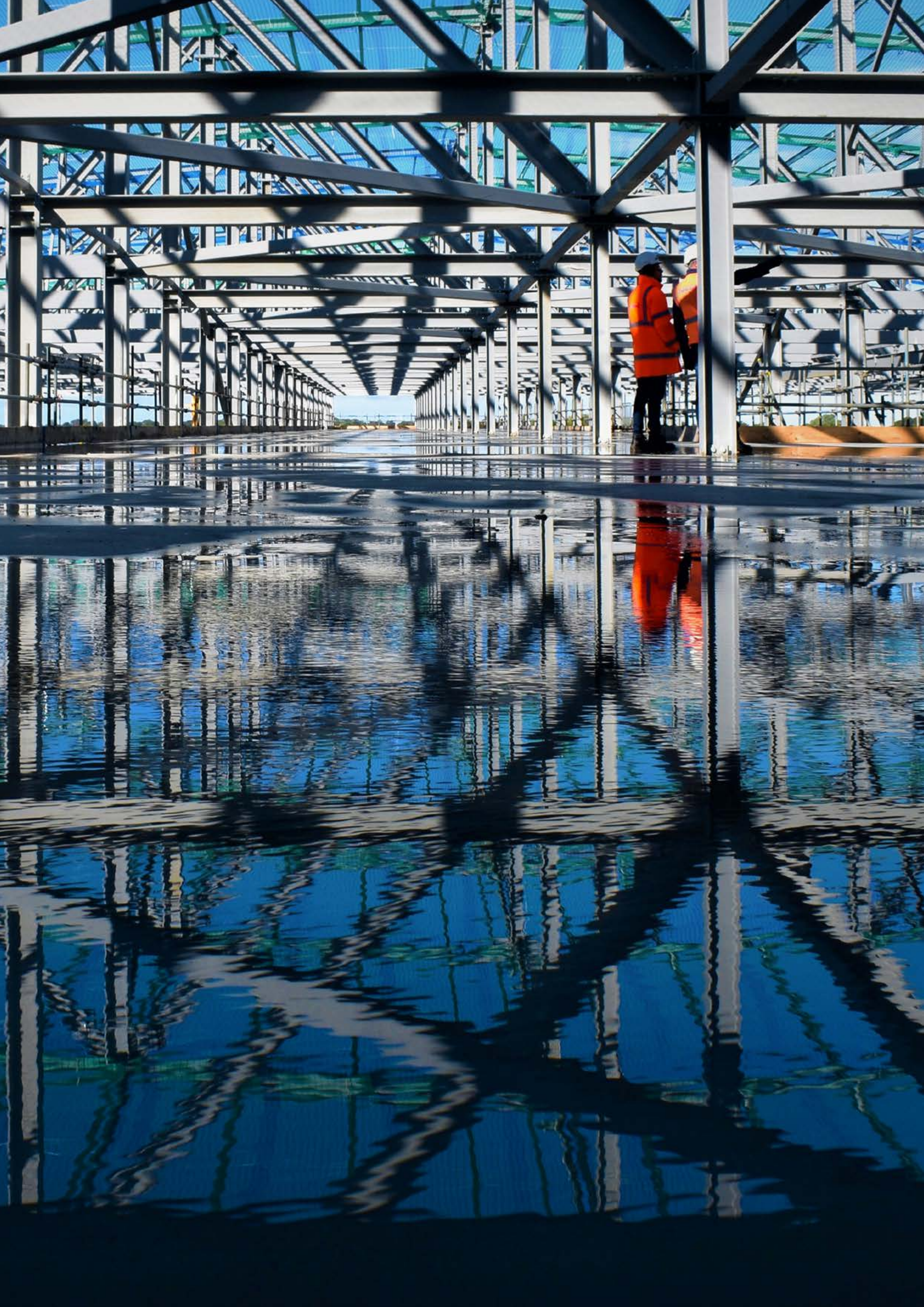
Since steel is valuable, waste associated with fabricating steel sections is negligible, typically 4-5% by weight. As an off-site production process, steel fabrication is undertaken in a controlled environment that ensures quality and minimises waste compared to site-based construction. Off-cuts from sections, web openings cut from members, swarf from drilled holes and any unused material from fabrication (Figure 6.1) is all recycled.

Engagement with steelwork contractors about project programme and lead periods can enable alignment of procurement with mill rolling dates. In this way, project-specific section lengths can be procured rather than stock lengths which generate more off-cuts.

Figure 6.1 Nested components

It is estimated that 10% of all waste arises from demolition activities[34]. Steel framed structures are more readily adapted or extended to accommodate a change of use, extending the useable life of the existing construction. On demolition, all steel sections are valuable, either as sections to be reused or as scrap to be remelted into new steel products.

A more general discussion on designing out waste is presented in Reference [72] .



STEEL-CONCRETE COMPOSITE CONSTRUCTION

Composite construction is the dominant form of construction for the multi-storey building sector in the UK. Its popularity is due to the strength and stiffness that can be achieved with minimum use of materials, utilising the compressive strength of concrete and the tensile strength of steel. By joining the two materials together structurally their strengths can be exploited resulting in a highly efficient and lightweight design.

Some designers simply specify typical composite solutions (for example, a 150 mm slab depth), known from previous designs to have adequate resistance. In many cases, the standardised solution of decking gauge and profile, slab depth and reinforcing mesh has unnecessary resistance which could be avoided with a solution developed for the specific design criteria.

There are various ways to minimise the embodied carbon of composite construction:

- Maximise the deck span to reduce secondary steel;
- Arrange profiled sheets to be double or even triple spanning;
- Consider trapezoidal decking profiles in preference to re-entrant;
- Utilise propping of the slab during the construction stage;
- Adopt the minimum slab thickness needed to ensure appropriate insulation in the fire condition;
- Use “low carbon” concrete;
- Use asymmetric steel sections with web openings.

Concrete with a high proportion of cement replacement (“low carbon” concrete) has a slower strength development compared with concretes made with conventional cements. The slower strength gain has no effect on composite slabs with profiled steel sheet, making this form of construction a suitable candidate for cement replacement concrete.

7.1 Multi-span profiled decking

The advantage of multi-span profiled decking is that the continuity over internal supports significantly reduces both the deflections and the sagging moment that the deck experiences. In turn, this means that in some cases, thinner, lighter decking may be utilised compared to a single span solution.

The benefits due to the reduction in sagging moments for double spanning decking are not as straightforward as the change in an elastic bending moment diagram might suggest. Failure may be governed by combined effects at the intermediate support.

Using a double span reduces deflections at the construction stage, meaning less additional concrete if the slab is poured to a level, or reduced risk of ponding (sometimes a concern in exposed structures like car parks) if poured to a thickness.

7.2 Decking profile

For the same overall slab thickness, trapezoidal decking profiles use less concrete than re-entrant profiles, so are preferred in a minimum carbon solution. Decking types are shown in Figure 7.1.

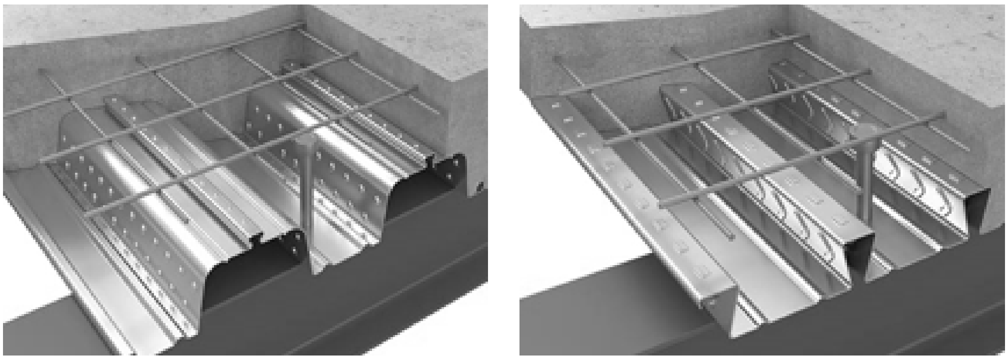


Figure 7.1 Decking profiles[73]

Trapezoidal deck

Re-entrant deck

For an identical structural design condition, a trapezoidal profile is generally lighter than a re-entrant profile and involves less concrete. If there are no other constraints, a trapezoidal profile may be preferred. A trapezoidal deck has a larger second moment of area compared to an equivalent depth re-entrant profile meaning the gauge can be reduced on a like-for-like basis resulting in a carbon saving.

Typical weights and embodied carbon values are given in Table 7.1 for an unpropped 150 mm deep slab and 1 mm thick steel profile.

Table 7.1 Embodied carbon (A1-A3) floor comparison for typical decking profiles

Decking profile	Steel weight (kg/m^2)	Reinforcement (kg/m^2)	Concrete weight (kg/m^2)	Embodied carbon A1 – A3 kgCO_2e/m^2	
				Low carbon	Average
Re-entrant (55 mm)	15.3	3.02	338	33	79
Trapezoidal (60 mm)	11.6	3.02	284	27	64

The embodied carbon values in Table 7.1 have been determined using the factors tabulated in Appendix D.

It should be recognised that although trapezoidal profiles have a higher structural resistance and stiffness, re-entrant decks may be preferred as the increased mass of concrete can offer improved acoustic and dynamic performance.

The spanning capabilities of trapezoidal profiles generally exceed those of equivalent re-entrant profiles, as seen in Figure 7.2.

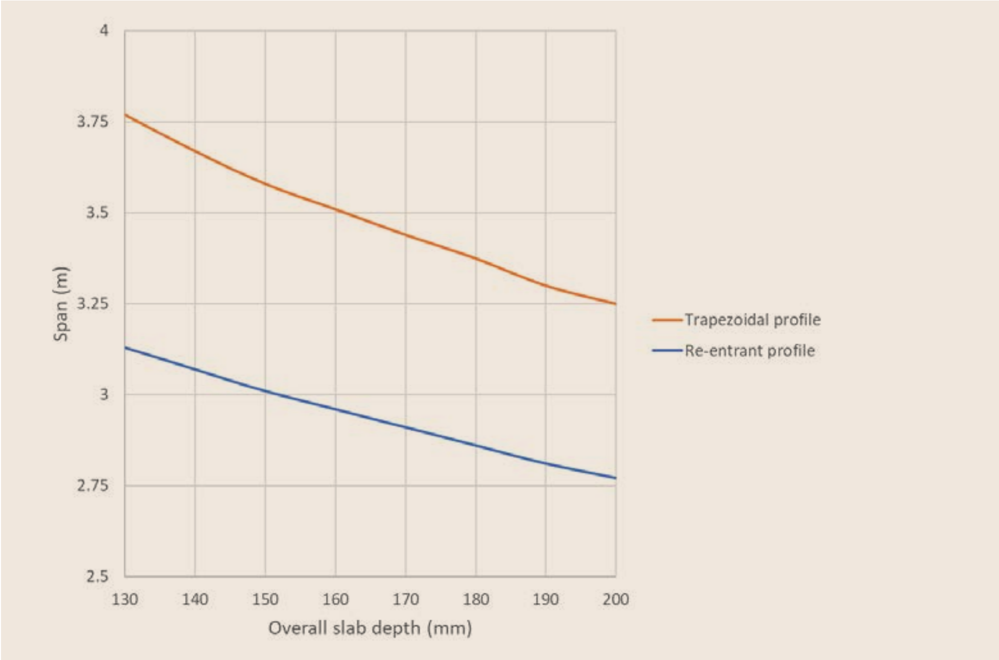


Figure 7.2 Comparitive structural resistance of re-entrant and trapezoidal decking profiles (typical profiles in 1.2 mm material, single span)

7.3 Propping during the construction stage

Propping of composite slabs as illustrated in Figure 7.3 is normally avoided as it is likely to compromise the construction programme, being required to remain in place until the concrete slab has gained sufficient strength. Propping may interfere with the early access to the floors below for following trades. The forces from the props obviously bear on lower storeys, which themselves must have sufficient strength, or the propping must be carried through several storeys.

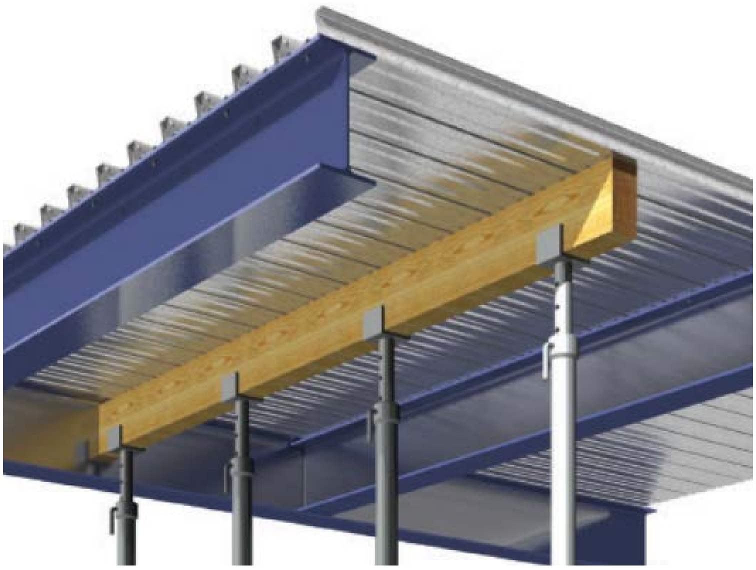


Figure 7.3 Propping of metal deck

Despite the potential impact on the construction programme, propping of the composite slab, whether at midspan or at third points, may enable material to be saved when (as is normally the case) the construction stage governs. Propping of the steel beams may also be beneficial. Extensive propping may impact the construction programme, so it should be limited in scope.

In unpropped construction, the deflection check at the construction stage is often the limiting factor in design – meaning deeper stiffer beams must be provided and the spans of the floor slab reduced. Propping can be used to ensure the construction stage does not govern the design.

Reduced deflection at the construction stage also means reduced ponding of the wet concrete, so the design load carried by the beams and slab reduces at both the construction stage and the normal stage.

Propping does have structural implications – the design codes increase the minimum percentage of reinforcement over the supports from 0.2% to 0.4% of the concrete cross section. This is to control the increased tendency to crack if props have been used at the construction stage.

Depending on the decking and slab depth, the effect of propping is to increase the spanning capacity by around 40%. Conversely, propping may mean that a reduced thickness of decking – with lower embodied carbon - may be utilised. Typical weights and embodied carbon values for various decking profiles are given in Table 7.2.

Table 7.2 Comparison of embodied carbon for varying gauge of decking profiles

Decking profile	Thickness (mm)	Steel weight (kg/m^2)	Embodied carbon A1 – A3 kgCO_2e/m^2		Saving
			Low carbon	Average	
Re-entrant (55 mm)	0.9	13.26	11.6	37.5	28%
	1.0	15.30	13.4	43.3	17%
	1.2	18.35	16.1	51.9	Baseline
Trapezoidal (60 mm)	0.9	10.50	9.2	29.7	25%
	1.0	11.62	10.2	32.9	17%
	1.2	13.97	12.2	39.5	Baseline

The embodied carbon values in Table 7.2 have been determined using the factors tabulated in Appendix D.

A comparison of maximum spans for single spans, double spans and propped spans is shown in Figure 7.4. The comparison is for a 60 mm trapezoidal profile and 130 mm slab with a total characteristic variable action of 3.5 kN/m^2. The figure shows the significant advantages of double spans and propping compared to an unpropped single span, realised as:

- Thinner gauge profile for the same span, or
- Longer spans and reduced secondary beams.

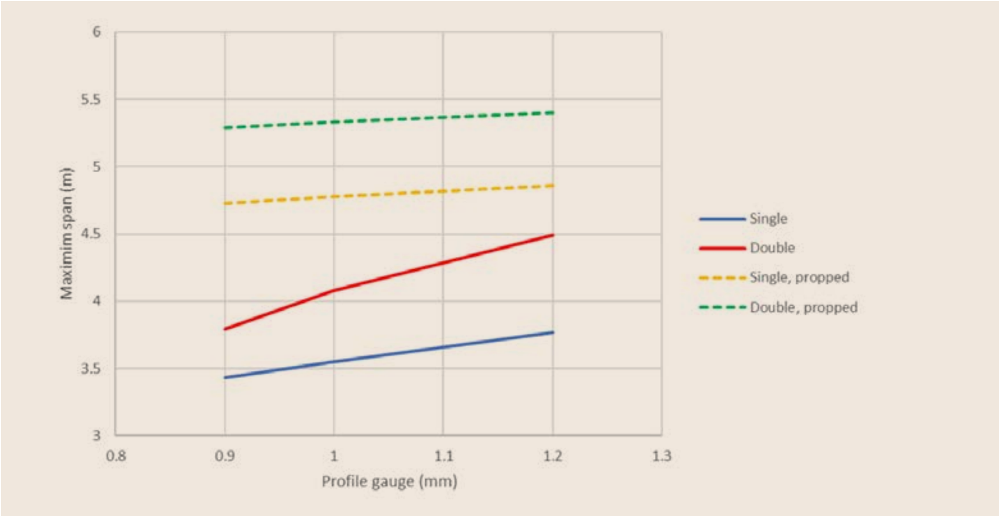


Figure 7.4 maximum spans for different support arrangements and profile thickness

7.4 "Low carbon" concrete

Significant reductions in embodied carbon can be achieved by replacing some of the cement with Supplementary Cementitous Materials (SCMs), including:

- Ground granulated blast furnace slag (GGBS)
- Fly ash
- Silica fume
- Limestone powder
- Calcinated clays
- Pozzalana

Cements and their broad designations (a formal term) are identified in BS 8500-24[74], with compositions given in BS EN 197-1[75].

The embodied carbon content of concrete with a range of cement types, described by their broad designation, is given in Table 7.3[76].

Table 7.3 Embodied carbon in various concrete mixes

Cement type	Percentage of addition	$kgCO_2e/m^3$	$kgCO_2e/kg$
CEM1	0	283	0.116
IIA	6 – 20	228-277	0.093 - 0.113
IIB	21 – 35	186-236	0.076 - 0.096
IIIA	36 – 65 GGBS	120-198	0.049 - 0.081
IIIB	66 – 80 GGBS	82-123	0.034 - 0.050
IVB	36 – 65 fly ash or pozzolana	130-188	0.053 - 0.077

(based on a cement content of 320kg/m^3 of concrete)

In addition to the "standardised" cements in Table 7.3, many of the larger concrete suppliers have their own proprietary low carbon products, which may be used when agreed with the specifier.

Reference [76] points out that composite performance depends on certain material properties in addition to the compressive strength. All relevant properties should be considered when specifying (or accepting) low carbon concrete for use in composite construction.

As previously noted, the longer striking times associated with “low carbon” concrete is not relevant for composite slabs with profiled steel sheet and has no impact on the construction programme.

It is important to note that the supply of some SCMs, e.g. GGBS, is limited and constrained and therefore although specifying high CEM I replacement rates may help to achieve project-level embodied carbon targets, it is unlikely to lead to global GHG emission reductions[24].

7.5 Fibre-reinforced concrete

Some manufacturers of profiled steel decking have completed testing to demonstrate the resistance of their decking used in conjunction with a fibre-reinforced concrete. More recently, small scale tests on fibre reinforced concrete (without decking) are used to determine spanning capabilities for different dosages of fibres. The fibre reinforcement may be steel or synthetic fibres.

A typical dose of steel fibres is 25 kg/m^3, meaning that in a typical 140 mm deep slab with trapezoidal decking the steel fibres weigh approximately 2.3 kg/m^2. A typical mesh (A193) weighs 3.03 kg/m^2, demonstrating a reduced weight of steel, in addition to the other benefits arising from the use of fibres. Reference [77] discusses the use of fibres in composite construction.

7.6 Beams with web openings

Beams with web openings are a common section choice for composite beams, offering long spans, light weight and the opportunity to readily integrate services within the structural zone.

Circular or rectangular openings may be cut into the webs of rolled I sections or H sections or formed in members fabricated from plate or rolled sections. Openings may be isolated or based on a regular pattern. Members formed with regular circular openings are commonly known as cellular beams. Some circular openings may be partly or fully filled with plate to provide locally increased resistance, or larger openings formed by connecting adjacent cells. Beams with web openings need more intumescent coating than beams with solid webs.

In composite construction, compared to standard rolled sections, fabricated members with large web openings demonstrate a reduction in weight generally in excess of 25% and in some cases over 40%.

Extensive design guidance is available in SCI publication P355[37].

Beams with web openings require specific calculations to determine intumescent coating thicknesses. Coating manufacturers have developed software to calculate this, but require accurate details of hole sizes and locations.

7.7 Asymmetric steel sections

In the final condition, the top flange of the steel section in a composite beam contributes little to the resistance, so a fabricated asymmetric section will involve less steel weight and less embodied carbon.

BS EN 1994-1-1[57] and guidance in P355[37] covers asymmetric steel sections where the area of the bottom flange is no more than three times the area of the top flange. Top flanges should be at least 120 mm wide to accommodate joints in the decking, and at least 8 mm thick to allow shear studs to be through-deck welded without damage.

Smaller top flanges reduce the lateral torsional buckling resistance, which may become critical during lifting, or at the construction stage before restraint is provided by profiled decking. A reduced steel section will increase the deflection, which may be important at the construction stage. Temporary propping (Section 7.3) may be used to address both effects.

Asymmetric sections are usually fabricated from pate (Figure 7.5) or cellular beams fabricated from different rolled sections (Figure 7.6).

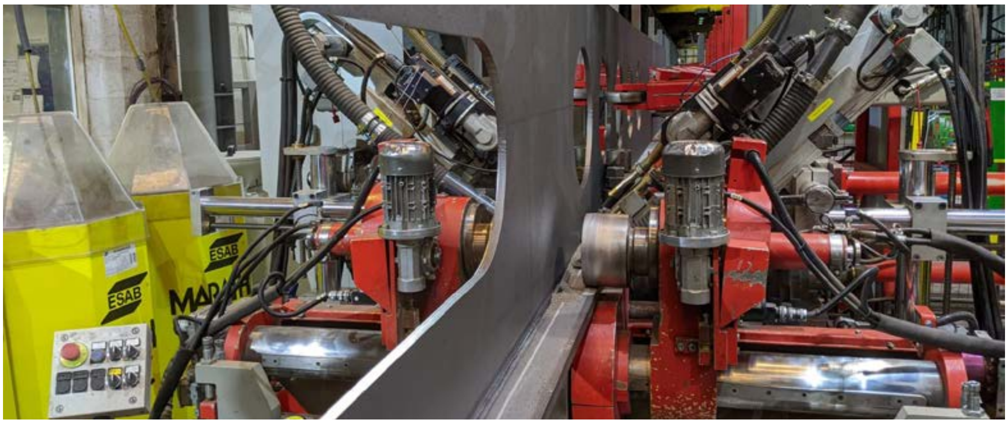


Figure 7.5 Asymmetric section fabricated from plate

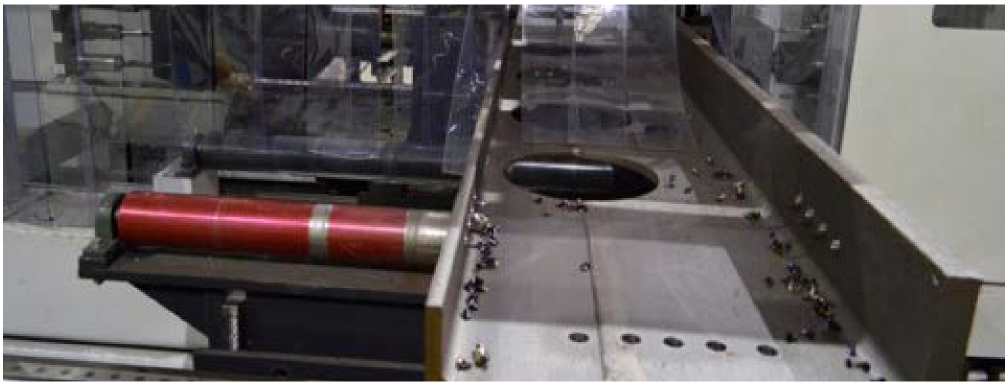


Figure 7.6 Asymmetric cellular beam fabricated from rolled sections

Fabricated members used as floor beams in composite construction generally have web openings to facilitate the integration of services. Design guidance is available[37] for both types of member, including software.

7.8 Design criteria checklist

1. Has the use of a mass timber slab been considered as an alternative to concrete (see Section 8)?
2. If concrete has been selected, has trapezoidal profiled decking been considered?
3. Has the use of “low carbon” concrete been considered?
4. Has the use of double span decking been maximised?
5. Has propping been considered and discussed with the client and main contractor?
6. Have different beam types been considered for the structural grid, including beams with web openings?
7. Have asymmetric steel sections been considered?
8. Has the floor slab been designed to be demountable in the future? (see Section 10).


W21

STEEL-TIMBER HYBRID CONSTRUCTION

8.1 Introduction

Combining mass timber with structural steel creates a competent, highly engineered solution with significantly lower upfront embodied carbon than composite designs using concrete-based slabs. Because the floors typically account for more than half of the embodied carbon of the superstructure, incorporating timber in the slabs is an efficient and rational use of timber to deliver low-carbon solutions. New aesthetic opportunities are also enabled, as illustrated in Figure 8.1.

Figure 8.1
Steel and timber solution

The environmental benefits are considerable, although the construction cost is currently (2024) higher than common solutions. However, once reduced programme times (up to 20% quicker) and reduced waste are accounted for, the cost of using mass timber can be comparable to traditional construction[78].

The fire performance of timber is often perceived to be a disadvantage, although this should not prevent its use, as explained in Section 8.4. Water ingress is an important concern and should be addressed carefully during transport, construction and building use. Early engagement and specialist advice from the supply chain are key to delivering resilient steel-timber hybrid construction. Attention to the fire performance and durability of timber construction are essential to ensure that the building can be insured and covered by a warranty. The first floor and roof may be constructed in traditional steel-concrete construction to enhance the resilience and robustness of the structure.

Although the steel beams and mass timber panels are positively connected (typically using screws), the composite action between both materials is generally disregarded in design at ULS due to the lack of non-contradictory design guidance. However, the benefit of composite action may be included when assessing the dynamic response of a floor.

The remainder of this section serves as an introduction to the use of mass timber to replace the usual concrete-based slabs. Several research initiatives are currently ongoing (2024) to develop guidance on steel-timber hybrid construction, potentially unlocking increased structural efficiency and resilience.

8.2 Mass timber floor solutions

Several engineered timber products are suitable for use as floors, including:

- Cross Laminated Timber (CLT)
- Laminated Veneer Lumber (LVL)
- Glued-laminated timber (glulam/unilam)
- Ribbed panels
- Timber cassette panels.

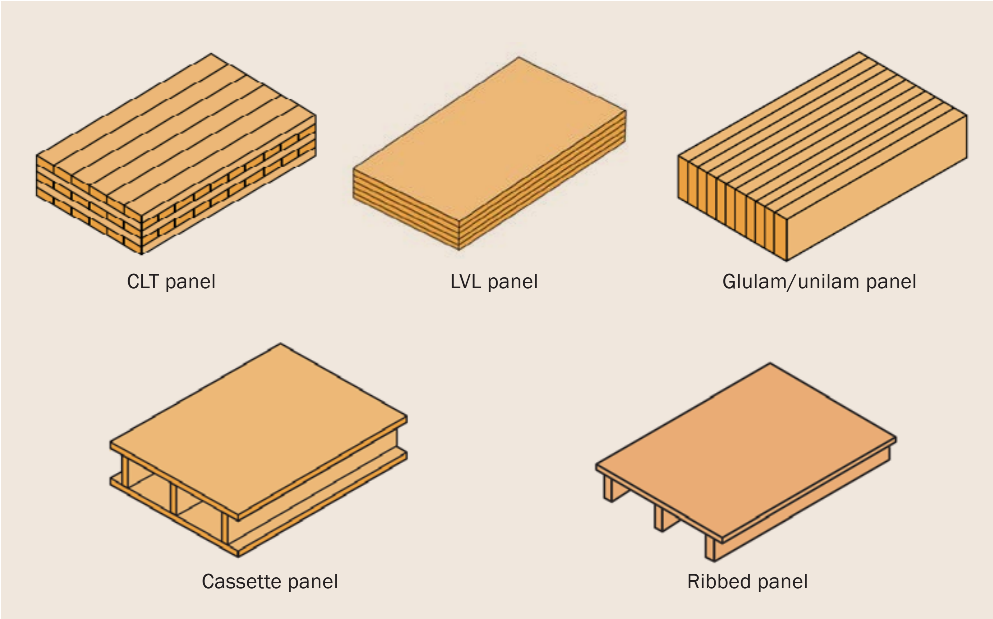


Figure 8.2 Examples of engineered timber floor panels

Of the five alternatives illustrated in Figure 8.2, CLT is currently (2024) of the most interest in the UK. CLT panels consist of several layers of softwood timber boards stacked and glued together, with the grain of each layer alternating in adjacent layers. Having layers with the timber grain parallel to both orthogonal directions, the panels can efficiently act as a floor diaphragm. Although the panels are reasonably stiff, the connection between panels must be considered. To improve performance, the panels may have a staggered layout.

There are no standard CLT panel configurations. Typically, each layer is between 20 mm and 40 mm thick (although layers of 80 mm are also available, commonly made of two 40 mm layers). The panels have an odd number of layers (typically 5 or 7 layers in flooring applications). Transportation constraints limit the sizes of the panels. The panels generally have a width of up to 2.95 m and a length of up to 16.5 m. CLT is manufactured and shaped using numerically controlled machinery, resulting in fabrication tolerances similar to those of structural steelwork. Special attention should be given to co-ordinating the steel-to-steel connections with the mass timber panels to avoid clashes.

8.3 Design of CLT slabs

Thicknesses up to 320 mm can be used for flooring applications. A span-to-depth ratio of span/25 is reasonable for a preliminary structural design, but the fire and dynamic performance of the floor (due to the relatively low floor mass) may dictate thicker panels. Advice from manufacturers should be obtained while designing the CLT panels.

The floor build-up needs to produce a satisfactory acoustic performance. Adding mass responsibly to the floor build-up improves the acoustic and dynamic response. Dry build-ups are preferable, to facilitate future recovery and reuse of the floor components (dry screed boards or gravel can be used). A wet concrete topping can also be considered, which facilitates other benefits (see Section 8.6).

8.4 Fire safety

One of the major disadvantages of steel frames incorporating exposed mass timber panels is that the structure adds itself to the fire load. The impact of mass timber panels must be recognised in the assessment of the performance of the building. After timber ignites, a layer of char is formed, which slows further burning to a predictable speed. This allows suitable-sized CLT panels to retain appropriate capacity at the required period of fire resistance. Unprotected CLT panels can provide an adequate load-bearing capacity at fire resistance periods over two hours. All details must be carefully assessed to ensure not only load-bearing capacity but also integrity and insulation requirements.

A performance-based fire assessment (rather than simple calculation models) should be undertaken by a fire engineer to demonstrate the self-extinction of multistorey structures with exposed mass timber components. The fire design of CLT panels needs to account for the type of adhesive used to glue the panel layers. Some adhesives show delamination of charring layers, which affects the performance-based assessment and increases the risk of flashovers. Heat-resistant adhesives are available and may be specified to improve performance. Ensuring that the CLT specification is consistent with the fire design assumptions is essential.

As previously noted, allowing for composite behaviour between steel and timber at ambient temperature is not yet common. Assuming composite behaviour at elevated temperature has not yet (2024) been sufficiently studied to be an option. As the connectors are directly exposed to heat, they lose shear resistance and stiffness, which may not be sufficient to even laterally restrain the steel beam. Unless proven otherwise, the connectors should not be assumed to provide effective restraint in fire conditions.

8.5 Sustainability benefits and circularity

Since trees absorb carbon dioxide as they grow, timber makes a positive contribution to lowering greenhouse gasses. The use of timber harvested from sustainably managed forests encourages new growth and more carbon to be absorbed. Note however that, as with ferrous scrap and GGBS, responsibly sourced timber is a limited resource and takes time to grow. In addition, the land available to grow responsibly sourced timber is limited.

Compared to in-situ concrete alternatives, mass timber panels can be more easily recovered to be reused or adapted to suit new applications. Steel-timber is more readily dismantled than steel-concrete systems, resulting in less damage to the steel members. At the end of life, if the timber panels cannot be reclaimed and reused, they can be used as biomass fuel, releasing the stored carbon but generating energy.

CLT panels are lighter weight than traditional composite slabs, reducing the demand on the supporting steelwork and leading to lower foundation loads (see Section 8.7) – a further cost and carbon benefit. However, because composite action between the steel beams and timber is not generally allowed for in steel-timber design, and due to the dynamic response typical of a lightweight floor, using mass timber alone (without a concrete topping) typically leads to heavier steel beams.

8.6 Steel-timber-concrete hybrid floors

Even though a concrete topping hinders deconstruction and adds embodied carbon to the slab, the in-situ concrete may be used to generate steel-concrete composite action between the topping and the supporting steel beams, for which adequate transverse reinforcement detailing is needed. Possible composite solutions are shown in Figure 8.3. A concrete topping delivers a solution similar to a traditional composite floor whilst benefiting from the use of mass timber, which can reduce the weight and upfront embodied carbon of the floor beams (see Section 8.7).

The concrete topping will act as a floor diaphragm, facilitating opportunities to use other forms of timber elements in the slab (Figure 8.2) since unidirectional panel behaviour is acceptable. When a topping is used, the performance in fire and robustness is improved and resilience to water ingress increased. The dynamic and acoustic performance are also enhanced with a concrete topping. The timber-concrete composite behaviour of the slab can also be considered, which improves structural efficiency and further reduces the upfront embodied carbon of the floor (CLT manufacturers can typically advise on the design and detailing of solutions).

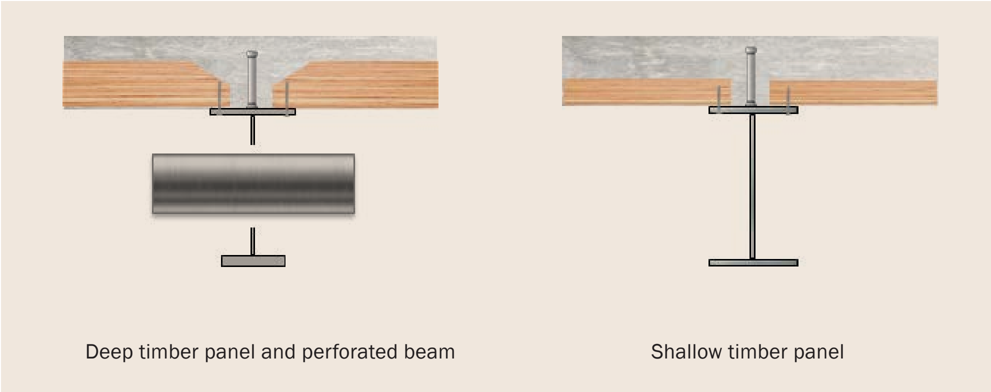


Figure 8.3 Composite details with CLT

8.7 Embodied carbon of steel-timber hybrid frames

The embodied carbon and total mass for CLT floor slab solutions are presented in Table 8.1 compared with a typical 150 mm composite floor with a 60 mm trapezoidal deck (see Table 7.1). The values are for the floor slab alone.

Table 8.1 Embodied carbon and total mass in floor slab solutions

Floor slab	$kgCO_2e/m^2$ Low-Carbon	%	$kgCO_2e/m^2$ Average[2]	%	Total Mass (kg/m^2)	%
150 mm composite floor 60 mm trapezoidal deck	27	100	64	100	299	100
150 mm CLT + 70 mm concrete topping	18	67	37	58	245	82
220 mm CLT	10	37	24	38	94	31

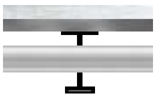
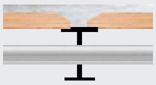
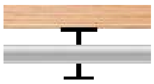
The embodied carbon values in Table 8.1 have been determined using the factors tabulated in Appendix D.

The impact on the embodied carbon of the whole structure is less pronounced than considering the floors alone, but still demonstrates a valuable reduction in embodied carbon. Table 8.2 illustrates a typical multistorey office steel-concrete framed building compared with floors using mass timber panels (slabs according to Table 8.1).
A 7.5 x 15 m column grid was considered, for which 34 kg/m^2 of steelwork (columns and beams) were estimated for the steel-concrete and steel-timber-concrete hybrid floors. For the (non-composite) steel-timber solution, 46 kg/m^2 of steelwork (columns and beams) was estimated, reflecting the loss of steel-concrete composite action.

Table 8.2 shows that an embodied carbon saving in the floors and columns of at least 20% is realised for the hybrid systems using mass timber panels. The lower superstructure self-weight will also reduce the demand for the substructure, enabling further cost and carbon benefits. The comparison also suggests that steel-timber-concrete floors can compete with the steel-timber alternative in terms of embodied carbon due to the structural efficiency realised by the steel-concrete composite action.

Considering that the superstructure typically accounts for 48% of the embodied carbon of the whole structure, the reduction potential for the entire structure would be about 10%. A further reduction is expected due to the lower superstructure self-weight, which reduces the demand on the substructure.

Table 8.2 Embodied carbon and total mass in superstructure solutions

Floor slab		$kgCO_2e/m^2$ Low-Carbon	%	$kgCO_2e/m^2$ Average[2]	%	Total Mass (kg/m^2)	%
150 mm composite floor 60 mm trapezoidal deck		38	100	119	100	333	100
150 mm CLT + 70 mm concrete topping		29	76	92	77	278	83
220 mm CLT		26	68	99	83	140	42

The embodied carbon values in Table 9.2 have been determined using the factors tabulated in Appendix D.

8.8 Design criteria checklist

1. Have steel-timber and steel-timber-concrete solutions been considered as alternatives to traditional concrete-based floors?
2. Has the project team acknowledged the holistic benefits of using mass timber – the environmental, aesthetic and biophilic benefits, the speed of construction, a lightweight solution with reduced demand on the substructure?
3. Have early engagement and specialist advice from the supply chain been sought to achieve an efficient and resilient solution?
4. Was a project water management plan developed to monitor and manage the moisture content of the timber panels?
5. Has the fire design demonstrated the self-extinction of the fire, and have appropriate design strategies and details been implemented to satisfy load bearing, integrity and insulation requirements?
6. Is the mass timber specification consistent with the project requirements, including the type of adhesive?

FIRE PROTECTION

Since the strength of steel reduces as the temperature increases, steel members are commonly protected, typically by intumescent coatings or by boards. In many cases, the specification of the protection is the responsibility of the coating or board manufacturer, perhaps as a sub-contractor to the steelwork contractor, or via a separate contract.

The theme of this section is that proper assessment of the necessary fire protection demands information relating to the design at ambient temperatures – experience is that such information, whilst readily available, is hardly ever provided. This means that the design of the protection is often based on conservative assumptions, wasting materials, time and carbon.

In some cases, steel members need no fire protection, or may need no additional fire protection if they are partially encased. SCI publication P186[79] offers guidance.

Of particular importance due to the high numbers of floor beams in a structure is that in typical floor slabs, many composite secondary steel beams may be left unprotected. Guidance is given in BRE Digest 462[80] and SCI Publication P288[81].

9.1 Design methods

The analysis to demonstrate that secondary composite beams may remain unprotected in accordance with SCI P288 is the responsibility of the structural engineer responsible for the overall design. In other cases, it is common in the UK for a third party to specify the necessary fire protection.

For fabricated plate sections and beams with web openings, manufacturers of the fabricated member have software which considers the ambient design and fire design concurrently. Users of such software packages are encouraged to detail the fire design data on their drawings to ensure the member-specific performance data is used to determine the necessary thickness of intumescent coating. If this data is not provided, a conservative assumption is that the member was fully utilised at ambient temperature, which results in a carbon and cost penalty.

In the UK, two approaches to fire design are commonly used, both described as “simple” calculation methods. The objective of both methods is to specify the necessary fire protection to ensure that the structure does not fail before the resistance period specified in the Building Regulations Approved Document (or equivalent documents in other parts of the UK).

To be undertaken satisfactorily, both methods require important information regarding the ambient temperature design to be communicated.

Numerical worked examples are presented in SCI publication P403[82]

9.1.1 Critical temperature method

The critical temperature method described in BS EN 1993-1-2[83] determines the temperature which must not be exceeded – fire protection is required to stop the critical temperature being exceeded.

The critical temperature method described in clause 4.2.4 of BS EN 1993-1-2 is only applicable to tension members and fully restrained beams. The scope of the method does not include composite beams, although many designers use the method, assuming it to be conservative. BS EN 1994-1-2[84] presents a simple critical temperature method for certain composite beams in clause 4.3.4.2.3.

The critical temperature method acknowledges that in the fire condition, the applied actions will be lower than in the ambient design condition, secondly that the fire condition is an accidental situation and finally that the member may not be fully utilised.

A simple approach to fire design is often adopted, calculating the reduced steel strength required to carry the actions in the fire condition. The critical temperature is then back-calculated from Table 3.1 of BS EN 1992-1-2 to be the temperature which corresponds to the reduced yield strength.

In the absence of appropriate design information about the ambient temperature design, the conservative assumption must be to assume the member was 100% utilised in the normal condition – an unlikely situation – which will lead to unnecessary protection material.

Conservative limiting temperatures for columns and beams are given in Table NA.1 of the UK National Annex to BS EN 1993-1-2[85], but these also depend on the degree of utilisation. Table NA.1 does not cover composite beams but describes steel beams where the top flange is protected by concrete. Conservative critical temperatures are also given in the “Yellow Book”[86].

Rather than adopting the conservative values from the UK NA or the “Yellow Book”, calculating the critical temperatures for the actual design situation will reduce the cost and the embodied carbon of the protection system.

9.1.1.1 Example - comparison of critical temperatures

In Table 16 of the ASFP “Yellow Book”, the default critical temperature for a non-composite, protected beam, supporting a slab, in a shopping area, is stated as 583 °C.

This temperature is based on the assumptions that the ratio of permanent actions to variable actions is 1:1 and the member is fully utilised at ambient temperatures. In this situation, the reduction factor η_{fi} has the value of 0.62. The reduction factor η_{fi} is the ratio of the design loading in the fire condition to that at ULS, as specified in BS EN 1993-1 2. In the fire condition, the design loading is unfactored. In addition, the variable action is reduced, reflecting the reduced probability that all the characteristic variable action will be present in the fire condition.

If the utilisation at ambient temperature was in fact only 85%, at elevated temperatures the degree of utilisation becomes 0.53. Based on equation 4.22 from BS EN 1993-1-2, the critical temperature increases to 608°C by simple interpolation of Table 3.1 of BS EN 1993-1-2.

If the ratio of permanent actions to variable actions becomes 1:1.5 *and* the member is only 85% utilised at ambient temperature, the critical temperature increases to 620°C by simple interpolation of Table 3.1 of BS EN 1993-1-2.

The UK National Annex to BS EN 1993-1-2 requires the design to calculate the utilisation factor μ_0, which is the product of the reduction factor η_{fi} and the utilisation at ambient temperatures (for example, $\mu_0 = 0.62 \times 0.85 = 0.53$ in the previous example). Table NA.1 from the UK NA yields 608°C as above, for a "Protected beam supporting concrete slabs or composite slabs".

The reduction in coating thickness depends on the product and the *A/V* ratio of the member to be protected. Coating thicknesses available in the public domain are generally linked to set temperatures – such as 620°C and 550°C as mentioned in the ASFP "Yellow Book". This limited availability of data means that manufacturers must be consulted to determine the reduction in coating thickness that results from an accurate calculation rather than accepting conservative inputs.

Typical coating thicknesses for the preceding three examples, based on a fire resistance period of 60 minutes, are shown in Table 9.1 illustrating that a properly engineered solution demands less coating material. In some instances, the reduction in thickness can also lead to a reduction in the number of coats, saving both material and time.

Table 9.1 Varying intumescent coating thickness with critical temperature

Critical temperature (°C)	coating thickness (μm)	Difference
583 (based on conservative assumptions in the ASFP guide)	640	baseline
601	570	89%
620	500	78%

Based on data from real projects, the approximate reductions in coating thickness possible when the critical temperature is properly determined in contrast to conservative assumptions are shown in Table 9.2.

Fire resistance period (minutes)	Saving if coating thickness properly calculated
60	10%
90	20%
120	35%

Table 9.2 Approximate reduction in intumescent coating thickness

9.1.1.2 *Intumescent coating specification*

Once the critical temperature has been determined, the required thickness of intumescent coating depends on data provided by the coating manufacturer and on the *A*/*V* ratio of the selected section (commonly known as H_p/A), which is given in section property tables. As the *A*/*V* ratio increases, the required thickness increases, to a point where it is simply inappropriate to select the member.

The 'Yellow Book' Volume 2: Part 4[87] tabulates required coating thicknesses for different protection periods, for ranges of *A*/*V*, for different products.

Hollow sections have a high *A*/*V* ratio. Depending on the product and the period of fire resistance required, the maximum *A*/*V* ratio for which protection thicknesses are listed may lie between 100 and 200 m^{-1}. Many hollow sections cannot therefore be protected, especially at the longer periods of fire resistance – a different member should be considered. If the section lies towards the maximum possible *A*/*V* ratio, the necessary thickness is likely to demand multiple coats which will be costly and time consuming whilst successive coats cure. The steelwork contractor will be able to suggest alternative solutions.

9.1.2 Steel temperature development method

The temperature development method of clause 4.2.5 of BS EN 1993-1-2 is used for unrestrained beams and columns. The method involves calculating the increase in temperature of a protected or unprotected member, following the standard fire curve. In small time intervals, the member temperature (which generally lags behind the fire curve) is calculated and the reduced member resistance is calculated. The reduced resistance is compared with the reduced design loads in the fire design condition and the process repeated until the design resistance falls below the design loads. This process establishes the time to failure, which is compared to the required period of fire resistance.

If the load in fire condition is not known, either conservative assumptions must be made, or the method not used.

9.2 Embodied carbon of intumescent coatings

EPD are available for certain intumescent coatings. The functional unit used is $kgCO_2e/m^2$, which makes comparisons difficult, since the coating thickness varies depending on the A/V ratio of the member and the period of fire resistance.

For typical plain steel floor beams, the total embodied carbon in the coating is approximately 10% of the total embodied carbon of the steel (based on an Module A1 value of 1.64 tCO_2e per tonne of steel section from Table 1.3). If the steel is specified from EAF production, the relative contribution increases. For beams with web openings, the relative contribution from the intumescent coating may increase for the reasons discussed in Section 9.2.1, but because the steel itself is lighter, the total contribution may still be smaller than that from protected plain beams.

As discussed in Section 9.1.1.2, the required coating thickness for member with a high *A/V* ratio will require thick coatings particularly for longer periods of fire protection. A lower total embodied carbon value may be possible by increasing the weight of the member and reducing the demand for thick protection.

9.2.1 Beams with web openings

Calculation of the fire protection of beams with web openings is more involved than when plain members are used. Tests have shown that:

- The steel between the openings (the “web posts”) heats up faster than the rest of the cross section;
- The level of protection provided by the intumescent coating is affected by the geometry of the openings;
- The web posts are often the critical element when determining the structural resistance of the member, particularly at elevated temperatures.

These effects are addressed by a “web post factor” which is product specific and related to the geometry of the section. The web will require an increased thickness of protection compared to a standard plain beam. Specific calculations are required to determine the required thickness, which are undertaken by the coating manufacturer, who must therefore have full details of the member and web openings.

9.3 Design criteria checklist

1. Has an exercise been undertaken to verify which secondary beams may remain unprotected?
2. Has sufficient information been provided on member utilisation to enable the fire protection to be specified without undue conservatism?
3. Have the characteristic values of the permanent and variable actions been provided, so that the correct reduction factor in the fire limit state can be calculated?
4. Have members with a very high *A/V* been selected - which cannot realistically be protected?
5. Have members been selected to give the lowest combined embodied carbon value including intumescent coating?


Tyvek

DESIGN FOR THE FUTURE

An effective way to the reduce embodied carbon of a new building is to reuse steel members recovered from redundant buildings (Section 3.3). Reuse is likely to become more important and more mainstream in the future, which will be facilitated by the steps described in this section.

10.1 Comprehensive information to facilitate future reuse

The first step in the process of reuse is to know what material is being recovered from redundant structures. As-built drawings are useful, but a comprehensive data model or material passport for the structure would be better. Associating data such as steel grade and sub-grade with individual members is entirely possible and should be a necessary part of the as-built building information model.

10.2 Joints

Almost all building steelwork is bolted together on site – there is little site welding in the UK which would otherwise make recovery of steel members more difficult.

Current recovery practice (2024) usually involves cutting or shearing an individual member near the connection rather than attempting to unbolt the structure. Recovered members are then refabricated with new joints to suit the new design. Offcuts from site and during refabrication are recycled as scrap steel.

In the short term, it is unlikely that recovery of complete members with their connections will result in new structures using the original members without refabrication. Firstly, the value of recovered steelwork would need to reflect the additional effort of dismantling. Secondly, it is unlikely that steel members would immediately suit the layout of a new building.

In the longer term, moral, legal and commercial drivers may increase the possibility of complete members being reused in their original form. For that reason, standardised joints following the “Green Books”[88],[89] should be adopted wherever possible.

It is noted that there is a small market for and examples of the reuse of whole buildings, for example, agricultural buildings. For these relatively simple, low-rise buildings, the structure may be dismantled to facilitate the reuse of the whole structure at a new location.

10.3 Demountable composite construction

Composite slabs and composite beams are the most common form of floor construction for multi-storey buildings in the UK, and present obvious challenges if the components are to be deconstructed and reused in the future.

SCI publication P428[90] provides comprehensive guidance on alternative forms of shear connector, which can be used so that slabs can be readily separated from beams in the future. Allowing for future deconstruction and reuse may have an initial cost and embodied carbon penalty, so it is important to consider the overall whole life carbon benefits.

For demountable construction, the standard through-deck welded shear studs are replaced with fixings which can be disconnected below the beam flange. The shear connector typically either protrudes through the beam and may be unbolted, as shown in Figure 10.1, or connected via a coupler, as shown in Figure 10.2.

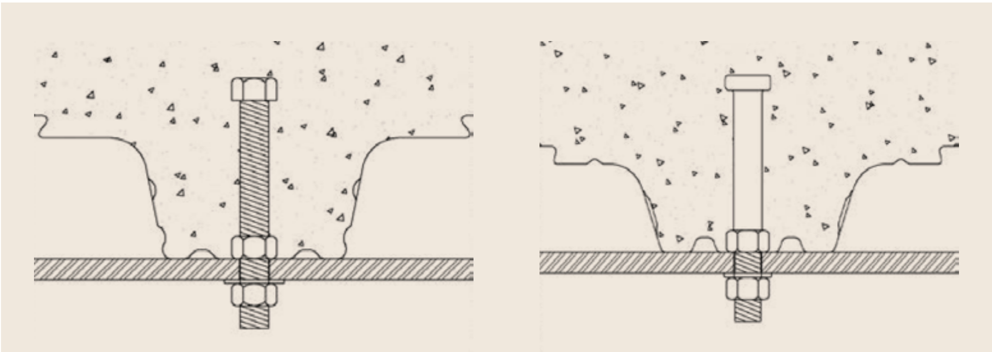


Figure 10.1 Bolted arrangements of shear connectors

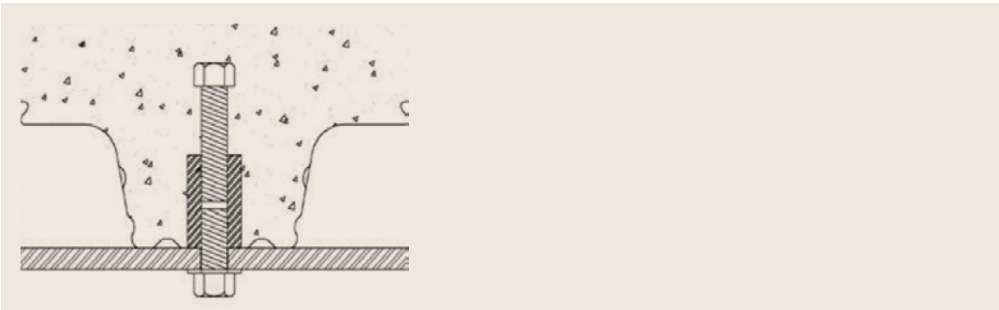


Figure 10.2 Shear connector with coupler

SCI P428 also makes practical proposals about beam spans and column layouts, proposing secondary spans of 12, 15 and 18 m. To allow for deconstruction and reuse, the primary beams, spanning 6, 7.5 or 9 m are designed as non-composite.

Tests have shown that demountable shear connectors are likely to be more flexible than orthodox welded studs – this increased flexibility is accounted for in the design guidance presented in P428.

SCI P428 proposes that the imposed floor load be taken as 5 kN/m^2 to allow for future reuse. This present guide recommends that the selected imposed floor load should consider only the current usage, recognising that the future cannot be predicted with confidence. If building information is appropriately recorded (Section 10.1), a future scenario could be envisaged where a "material bank" of building components with known properties is available[33], from which designers can select components for reuse as is done with new components.

10.4 Other floor solutions

The use of mass timber as a floor slab is introduced in Section 8. Timber floor slabs have the advantage of replacing the carbon-intensive concrete floor slab with a sustainable product and are eminently demountable. Timber slabs may be reused in their original form or modified to suit a new application.

Re-usable and readily demountable floor cassettes, as shown in Figure 10.3 are an appropriate floor solution for medium rise residential buildings. The technology is not new – such buildings are often constructed from modules with either a composite floor slab or timber on light steel joists.

Figure 10.3 Light gauge steel floor cassette

Composite floor cassettes of the form shown in Figure 10.4 offer the advantages of offsite construction, fast erection of a completed floor and the opportunity to readily demount the panel for reuse elsewhere.

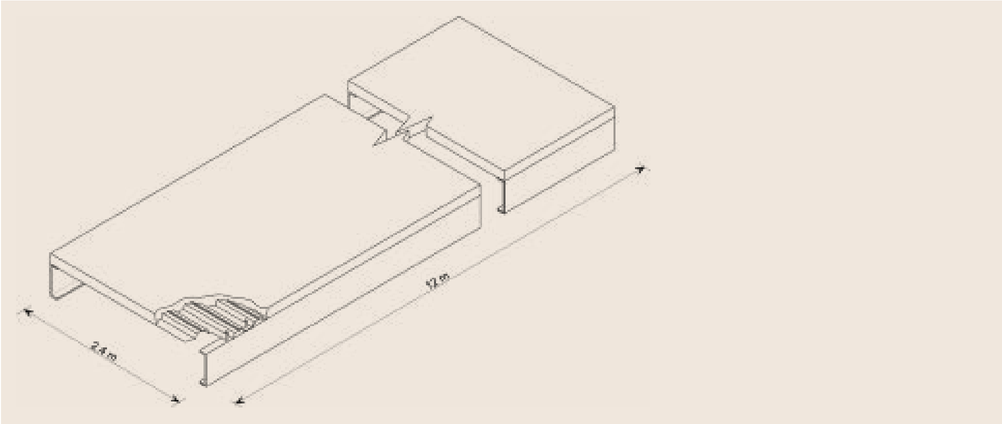


Figure 10.4 Composite floor cassette

10.5 Design criteria checklist

1. Is there commitment within the project team to provide a comprehensive model or material passport for the completed structure, with steel grade and sub grade for each member?
2. Are there any connections or other details which will mean that disassembly in the future is more difficult?
3. Can the floor slabs be designed to be demountable?

BUILD CLEVER – BUT NOT *TOO* CLEVER!

The forgoing sections have reminded designers of the many ways in which embodied carbon can be reduced through good design. Some of the recommendations are simple best practice, whilst others involve more complexity in design, or construction practice, or both. This section serves as a warning to ensure that structural reliability is maintained, and to ensure that theoretical analyses do not obscure the reality of three-dimensional solid objects, with associated tolerances and imperfections.

11.1 Joint stiffness

Although BS EN 1993-1-8 has a method to calculate stiffness, it is laborious by hand and limited in scope. Software is widely available which will calculate a joint stiffness even for quite unorthodox joints, meaning that joint stiffnesses may be readily incorporated into a frame analysis.

Although software may report a joint stiffness, the result should be critically assessed. Any calibration is likely to be against very limited test results, so the results for complex connections rely entirely on the processes implemented within the software.

The steel material is almost certain to be stronger that the nominal strength incorporated within software. Stronger material may change the behaviour of a joint, potentially exposing more brittle components to higher loading than assumed. Joints which are stronger than assumed will pass more moment to a supporting column.

Detailing a joint to achieve a certain stiffness about one axis will inevitably affect the stiffness in the other axes. Although frame analysis software will permit different stiffnesses in different axes, the real stiffness in each axis will depend on the details of the joint – it may not be possible to detail a real joint to meet assumed stiffness characteristics. A better approach is to assess the stiffness of the detail and use the resulting values in the frame analysis. If this approach is adopted, the joint design and frame design should proceed concurrently and be undertaken by the same designer.

An assessment of likely joint stiffness during the frame analysis stage is particularly important if the members are large, such as plate girders, or subject to large loads, including tie forces. In both cases the connection detailing may preclude the assumption that the joint can be classed as nominally pinned, potentially transferring more moment into the supporting structure.

If three-dimensional analyses are completed, designers are encouraged to make appropriate releases in the analysis model. If this is not done, joints can be subject to bending about both axes, axial forces and torsion. The combination of effects can mean that an otherwise simple joint becomes complicated and expensive. The assumption that joints are *nominally* pinned or *nominally* rigid has proved to be appropriate in decades of practice and is recognised by design standards.

11.2 Temporary works

Extensive temporary works have an associated cost and carbon penalty. In some situations, a better solution overall is to reduce the requirement for temporary works, even if the cost and carbon content of the primary structure is increased. For these and other reasons, projects that involve extensive temporary works should be discussed with the Steelwork Contractor. Safety is paramount, so designs which introduce (or leave) additional risks to be managed on site are generally not good designs.

11.3 Load paths to ground

Experienced steelwork designers always aim to provide short, direct paths to transfer applied loads to foundations. Structures designed in this way are normally simpler, more robust, easier to erect and demand less temporary works.

11.4 Joint details

Most fabrication is associated with adding joints to otherwise plain members. Minimising weight should not be so extreme that members need extensive reinforcement at joints. Highly utilised members within trusses are a typical example where the members meeting at a joint can lead to stresses above yield, notably in web panels subject to longitudinal stresses in combination with shear.

Prefabricated truss panels will generally be bolted on site – the member should be selected so that the necessary bolt holes at the splices do not reduce the member resistance below the design forces.

The physical size of a joint should be recognised at the frame analysis stage. In contrast to an analysis model where all members may meet at the same node, the real members may mean the joint is quite different to the analysis model. This is particularly the case if members intersect at relatively shallow angles, when the members may be connected over a considerable length, as shown in Figure 11.1. If the diagonal member is a UC section, with approximately 40% of the axial force in each flange, the intersection of one heavily loaded flange is a considerable distance from the model node. In some situations, it is better to move the intersection point to give a more compact joint, accommodating the resulting effects of eccentricity within the design model.

Much more information on the communication of connection design intent and design forces is given in Reference [91]. Joint design in the UK is usually the responsibility of the Steelwork Contractor, based on information supplied by the designer of the structure. Best practice for the designer of the structure is:

- To consider the physical reality of the likely joint – it may not be physically possible to connect multiple members at the same location;
- To anticipate the joint detail and therefore its classification as nominally pinned, or nominally rigid or semi-continuous – for example it may not be possible to detail a nominally pinned joint if the members are large or carry large design forces;
- To arrange primary and secondary beams on an orthogonal grid, so that connections to the columns are on their major and minor axes;
- To provide design accurate forces in equilibrium at complex intersections;
- To make appropriate releases in the analysis model, so that joints are not unnecessarily subject to torques or minor axis moments.

Figure 11.1 Shallow member intersection

11.5 Minimum element dimensions

In some types of construction, minimum dimensions must be observed. The more common requirements are:

- In composite construction, a minimum flange thickness of 0.4 times the shear stud diameter (7.6 mm for a 19 mm stud);
- In composite construction with decking, a minimum flange width of 120 mm (75 mm if the decking is continuous over the support);
- In composite construction with precast concrete slabs, a minimum flange width of 235 mm with site welded shear connectors or 220 mm with shop welded shear connectors[38]. With special control of tolerances, the minimum width may be reduced to 170 mm for shop welded shear connectors;
- For fabricated girders, webs should generally be a minimum of 8 mm and a h_w / t_w ratio less than 70. Although more slender webs may be designed, the member becomes difficult to manipulate during fabrication. For flanges, c/t should be limited to 14ε.

11.6 Propping

Propping during the construction phase can facilitate the use of lighter sections and longer spans but may have programme implications. Extensive propping should only be considered with the agreement of the main contractor and client as props can prevent access for following trades.

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APPENDIX A MODULAR APPROACH TO ASSESSING THE SUSTAINABILITY OF CONSTRUCTION

Within the EN suite of standards governing the sustainability assessment of construction works, life cycle assessment information is broken down into Modules (A, B, C and D) and sub-modules (A0, A1, A2, A3, etc), this so-called modularity approach is intended to promote transparency of reporting of impacts across the whole building life cycle.

Modules A to C fall within the building system boundary. Module D, which is outside the system boundary, is a measure of potential environmental loads and benefits, in particular relating to recovery, reuse and recycling of products and materials.

It is important to note that Module D values are ‘potentials’ that are based on things/scenarios that might happen at some point in the future, not actual impacts as is the case for Module A.

Module D is split into sub-modules; D1 (relating to reuse and recycling potentials of construction products and materials) and D2 (relating to exported energy). Module D1 is an important metric for structural steelwork which are inherently reusable and recyclable products.

Module D1 is intended to be a measure of the benefits of reusable and recyclable products/materials and to encourage measures to deliver a future circular economy, for example through design for deconstruction and reuse.

It should be noted that BS EN 15804:2019 mandates the reporting of Modules A1-A3, C1-C4 and D for almost all construction products. Despite this, Module D is generally not considered in whole life embodied carbon assessments in the UK and Module D1 targets are not adopted on projects.

Modules A and D reflect impacts and emissions at different points in time; generally many years apart in the case of long-lived products like buildings. The time-dependency of GHG emissions is something that is being studied but no definitive guidance has yet (2024) been developed.

The urgency to address climate change means that in general, greater importance is placed (rightly or wrongly) on reducing emissions today rather than reducing them in the future, i.e. reducing Module A impacts rather than increasing Module D benefits. In practice, both today's and future emissions need to be simultaneously tackled.

A further point is that Module D1 is generally quantified, under BS EN 15804, based on the substitution (or saving) of primary steel making, i.e. around 2.5 tCO_2e per tonne today. But, in say 50 years, primary steelmaking should be substantially decarbonised and therefore Module D1 calculated today will have significantly over-estimated the future saving.

Some EU member states have developed national rules, assessment methods and databases of environmental impact data in which a proportion of the Module D1 saving can be taking into account today, i.e. aggregating a proportion (typically 30-50%) of the potential Module D1 benefit with today's Module A impact.

A.1 Calculation of Module D

Calculation of Module D1 is complex and is related to some fundamental concepts relating to life cycle assessment (LCA), goal and scope of assessment, etc. It is not intended to cover this complexity here.

In the context of steel however, the Module D1 calculation approach can be explained relatively simply, using the following equation:

$$\text{Module D1} = \sum(\overbrace{RR_i}^{\text{Output flows}} - \overbrace{S_i}^{\text{Input flows}})(\underbrace{X_{re,i}}_{\text{LCI for secondary production or recovery}} - \underbrace{X_{pr,i}}_{\text{LCI for primary production}})\overbrace{Y_i}^{\text{Yield factors}}$$

The first bracket is a measure of 'net flow' through the system. For example, for BF-BOF production with 15% scrap input (S) and an end-of-life recycling rate (*RR*) of 90%, the net flow over the product life cycle is 90% - 15% = 75%.

The second bracket reflects the 'potential saving' achieved through future recycling (or reuse X_{re}) relative to primary (BF-BOF) production (X_{pr}). For example, if a steel section is recycled, 0.5 tCO_2e is emitted per tonne (via recycling in an EAF) but 2.5 tCO_2e is "saved" per tonne, i.e. avoiding primary (BF-BOF) production.

For this scenario therefore Module D1 = (0.9 - 0.15) x (0.5 – 2.5) = -1.5 tCO_2e per tonne.

In the above, a negative value indicates a 'saving' whereas a positive value indicates a 'burden'.

For the case of steel sections today, the Module D1 value for BF-BOF production represents a large 'potential saving' typically -1.6 tCO_2e per tonne whereas for 100%

scrap-based EAF production Module D1 is generally a small ‘potential burden’, i.e. a positive value.

Annex D of BS EN 15804:2019 provides end-of-life formulae for the assessment of the environmental impact of different information modules of construction products including Module D1.

A.2 How to use Module D1

BS EN 15804 and BS EN 15978 both mandate reporting Module D1 but state that it should not be aggregated with other modules and should be reported and communicated separately.

Despite this clear recommendation, there is no definitive guidance in standards about what can and cannot be done in terms using Modules A and D to make construction decisions. While BS EN 15804 and BS EN 15978 make clear recommendations about how the environmental impact data should be reported, how they are used to make decisions is outside the scope of these standards.

Although Module A embodied carbon targets are widely used in the UK, there are currently (2024) no Module D1 benchmarks or targets.

Without considering Module D1, a product that can be reused or a material that can be closed-loop[1] recycled, is treated in exactly the same way (in terms of its environmental impact) as one that is, for example, landfilled or incinerated. Module D1 targets for buildings (or alternative circular economy targets) are therefore needed to incentivise longer-term low-carbon and circular economy decision-making.

[1] Closed-loop recycling refers to the recycling of a product into a new product without degradation of properties, e.g. recycling scrap steel into new steel.


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APPENDIX B TYPICAL STEELWORK WEIGHTS

B.1 Published values of steelwork weights

Many sources quote weights of steelwork and of embodied carbon for various types of structure and arrangements.

Unfortunately, there is generally no common basis on which reliable comparisons can be made between quoted values. Some sources are for the steelwork in floor grids alone, neglecting the columns. Other sources include the steel beams, the profiled steel decking and the reinforcement in the concrete. Others include the concrete and make allowance for the foundations. In making any comparison, the product scope included in the quoted value should be critically appraised.

Steel weights and associated embodied carbon obviously vary considerably depending on the value of the variable action assumed. Identifying the structure as an "office" does not clarify the value of the variable actions, which might vary between 2.5 kN/m^2 and 5 kN/m^2, plus an allowance for moveable partitions.

The clear span also influences the steel weight, as does the limitation on floor construction depth – if any.

The detailed background to all quoted values should be investigated before conclusions are reached.

Sections B.2 and B.3 provides some indicative values, with limited background. The recommended rule of thumb is that any steel weight above 50kg/m^2 for steelwork alone in an "office" structure should be challenged – there must be reasons why this typical value is exceeded.

B.2 Typical steel weights for medium rise multi storey buildings

Typical composite slab: Floors alone, including steel beams, decking and reinforcement: 40 kg/m^2. This value is based on 15 kg/m^2 for decking and reinforcement (see Table 7.1 and a value of 25 kg/m^2 as a typical weight of steel beams taken from Figure C.3).

Typical composite slab: Entire steel structure, including floors on 7.5 x 12 m grid, decking and reinforcement, Q_k = 2.5 kN/m^2: 45 kg/m^2.

Typical composite slab: Entire steel structure, including floors on 7.5 x 18 m grid, decking and reinforcement, Q_k = 2.5 kN/m^2: 55 kg/m^2.

Typical composite slab: Entire steel structure, including floors on 9 x 15 m grid, decking and reinforcement, Q_k = 2.5 kN/m^2: 65 kg/m^2.

Cellular beam floor steelwork alone (no columns, no decking, no reinforcement), 7.5 x 12 m grid, Q_k = 2.5 + 1 kN/m^2: 25 - 28 kg/m^2.

Cellular beam floor steelwork alone (no columns, no decking, no reinforcement), 7.5 x 12 m grid, Q_k = 4 + 1 kN/m^2: 28 – 32 kg/m^2.

B.3 Other forms of construction

Typical steelwork weights for other forms of construction may be found in Reference [92], reproduced below:

Single bay buildings with roof trusses	26 - 38 kg/m^2
Single bay portal frames*	
without overhead cranes	31 – 47 kg/m^2
with overhead cranes	60 – 100 kg/m^2
Multi span portal frames*	
without overhead cranes	28 – 44 kg/m^2
with overhead cranes	55 – 100 kg/m^2
Aircraft hangers	45 – 85 kg/m^2
Grandstands	51 – 105 kg/m^2
Car parks	31 – 57 kg/m^2

* The quoted weights are typical for frame geometry of the 1990s and may not be appropriate for the much taller buildings currently (2024) constructed.


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APPENDIX C
INDICATIVE SAVINGS

This appendix presents typical weight (and associated carbon) savings for the recommendations presented in this guide. In many cases, the maximum possible saving may not be realised, primarily because steel members are rolled in discrete sizes.

Where relevant, an explanation of the saving is given.

C.1 Design values of actions (4.3.1 and 4.3.2)

Variable action (section 4.3.1) Use of 2.5 kN/m^2 instead of 5 kN/m^2

Partitions (section 4.3.2) Use of 0.5 kN/m^2 instead of 1.0 kN/m^2

Using the example in section 5.3.4, assuming the same slab for both options, design values of actions are:

With 5 + 1: 14.2 kN/m^2, using expression 6.10 from EN 1990

With 2.5 + 0.5: 9.3 kN/m^2, based on the more onerous of expression 6.10a and 6.10b from EN 1990

This represents a 34% saving in design load.

C.2 Use of reduction factors for variable actions (5.3.3) and high strength steel (6.11.1)

Figure C.1 shows the column selected weights for a ten storey building with varying imposed load, steel grades and use of the reduction factors for variable actions. At each floor the area supported by one column was taken as 108 m^2. The permanent action was taken to be 3.5 kN/m^2 with an additional permanent load of 0.85 kN/m^2. The buckling length was taken to be 3.5 m and each column size was used for two storeys. The design axial load was calculated using expression 6.10 of EN 1990.

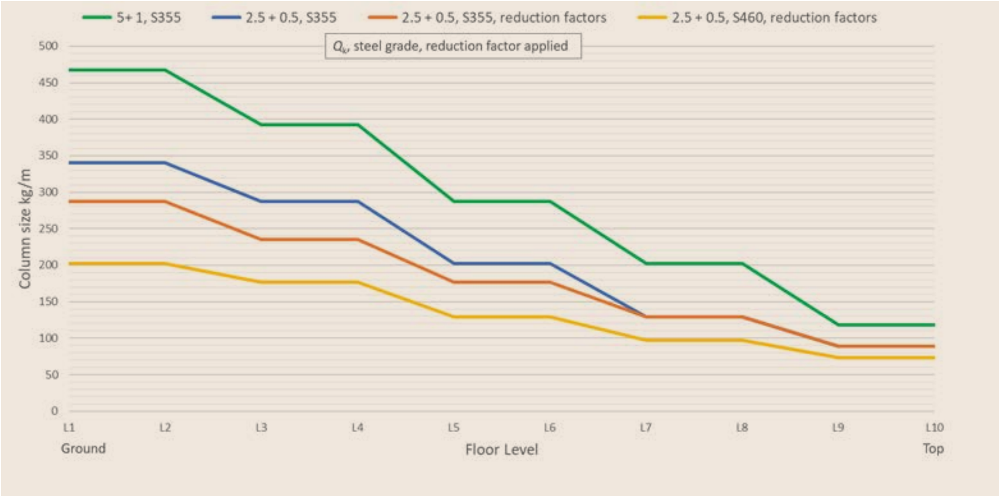


Figure C.1 Indicative column weights

As previously noted in Section 5.11.1, the use of S460 shows a reduction in total weight of each column of 25%, compared with S355 (applying the reduction factors in each case) assuming every section is readily available.

Since rolled sections are only available in discreet sizes, the utilisation in the lower portion of each column length varies between 83% and 97%. The utilisation in the upper portion of each column length is lower, since the axial load is lower.

C.3 Use of expressions 6.10a and 6.10b (Section 4.3.4)

Using the example in Section 4.3.4, the reduction in the design value of actions is 4%

C.4 Fully utilise members (5.2)

Although Reference [51] identified the potential to save 36% of the steel weight in a building, this value assumes every steel member could be 100% utilised. Since some members are selected based on serviceability requirements, some selected to meet minimum sizes and recognising that steel members are rolled in discreet sizes, a more realistic potential saving is estimated below.

10% saving in beams, comprising 60% of the total superstructure = 6%

10% saving in columns, comprising 15% of the total superstructure = 1.5%

Total potential saving on the superstructure = 7.5%

Comments

Although a reduction in steel weight will reduce the foundation loads, the effect is small, noting that the weight of the floor slabs is unchanged.

C.5 Use of semi-continuity (5.8)

Examples presented in Reference [61] show weight savings of between 10 and 30% compared to non-composite steel beams.

Assuming a typical saving of 15%, and that beams comprise 70% of the total superstructure, the potential weight saving = 0.15 x 70 = 10.5%

Comments

It should be noted that the comparison presented above relates to non-composite plain steel beams, whereas the current (2024) common solution is composite beams with web openings. It is assumed that the columns do not increase in weight.

C.6 Decking profiles, slab depth, reinforcement (7)

Figure C.2 shows a comparison embodied carbon (A1-A3) for different slab types. This graph has been generated using the average embodied carbon factors defined in Appendix D. Substituting fibre for reinforcing mesh reduces the overall embodied carbon content by between 1 and 2%. The data is based on a 9 x 12 m grid and a 90 minute period of fire resistance. The composite slab was designed to span 3 m, and the CLT span designed to span 4.5 m.

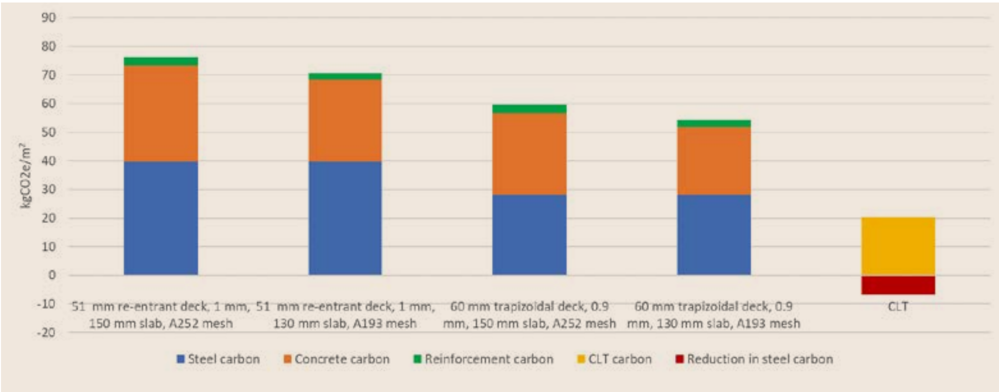


Figure C.2 Embodied carbon content – floor slab

The CLT solution shows a reduction in embodied carbon for the steel, as fewer secondary steel members are required.

C.7 Members with web openings (7.6)

Steelwork intensity for various grid arrangements of beams with web openings are shown in Figure C.3. The “secondary” beam is always the longer span member. The results in Figure C.3 are based on a 150 mm deep slab, a superimposed permanent action of 1.5 kN/m^2 and a total variable action of 3.5 kN/m^2. Figure C.3 indicates an increase in weight as the clear span increases, which must be assessed against the overall whole life benefits of readily adaptable clear spans.

Increasing the clear span from 9 m to 13.5 m increases the weight of the floor steelwork by 25%, but this is a small proportion (in the order of 7%) of the embodied carbon of a building superstructure.

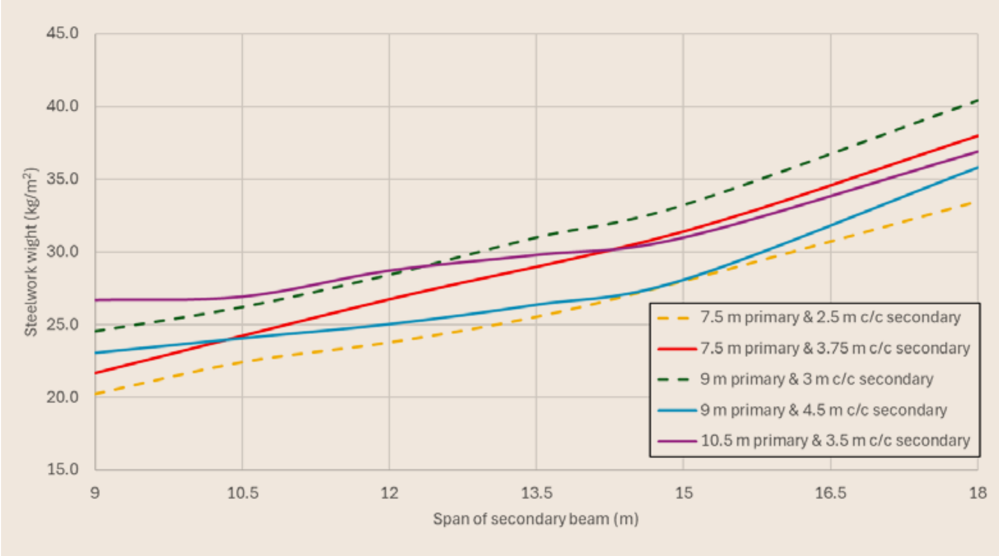


Figure C.3 Steel intensity – beams with web openings

Weight savings vary but typically range between 20 and 45% when beams with web openings are compared to standard rolled sections for longer span beams.

Assuming a typical saving of 25%, and that beams comprise 70% of the total superstructure, the potential weight saving = 0.25 x 70 = 17.5%.

The increase in fire protection coating necessary for beams with web openings must be considered.

C.8 Use of hybrid floors with mass timber panels (8)

Replacing a conventional composite slab with CLT or CLT with concrete topping facilitates an embodied carbon reduction of approximately 20% embodied carbon for the floors and columns in a typical multi-storey building (Table 8.2).

APPENDIX D DERIVATION OF “LOW” AND “AVERAGE” CARBON EMISSION FACTORS

The embodied carbon values presented in Table 7.1 and Table 7.2 have been derived using the Module A1-A3 carbon emission factors given in Table D.1.

‘Average’ embodied carbon factors are based on the recommended default values provided in the IStructE guide on calculating embodied carbon[11] unless otherwise indicated.

‘Low’ embodied carbon factors are based on the ‘typical lower bound’ provided in the IStructE guide on calculating embodied carbon[11] unless otherwise indicated.

Table D.1 Embodied carbon emission factors

Product	Embodied carbon emission factors (Module A1-A3) ($kgCO_2e/kg$)	
	“low” embodied carbon scenario	“average” embodied carbon scenario
In-situ concrete C25/30	0.056 (70% GGBS)	0.10 (25% GGBS)
Steel rebar and mesh (UK CARES sector average EAF)	0.3[1]	0.76
Galvanised profiled steel decking	0.876[2]	2.83
UK and Europe CLT	0.11	0.25
Structural steel sections	0.33[3]	1.64[4]
Structural steel plate	0.913[5]	2.45

Notes

1. XCarb® Recycled and renewably produced Reinforcing steel in bars and coils EPD-ARC-20210245-CBA2-EN
2. XCarb® recycled and renewably produced Hot Dip Galvanised steel coils with zinc coating EPD S-P-11500
3. XCarb™ Recycled and renewably produced Structural steel sections and merchant bars EPD-ARC-20210132-CBB1-EN
4. UK consumption based average for hot-rolled sections (2019-22 average). BCSA and ISSB.
5. XCarb® recycled and renewably produced steel Heavy Plates S-P-10991

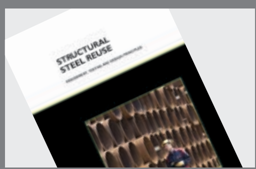
BEST PRACTICE FOR DESIGNING LOW EMBODIED CARBON STEEL BUILDINGS

Minimising carbon emissions is a priority for all parties engaged in construction, with ambitious targets to decarbonise within the next 25 years. In the structural steelwork industry, in addition to manufacturers changing the steel production process, designers have a critical responsibility to develop structurally efficient, highly utilised structural solutions.

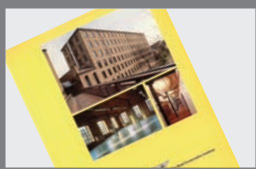
This guide sets out the opportunities to develop designs with low embodied carbon, by building less, building clever, building efficiently and minimising waste. The steps to a low carbon design are not innovative or difficult – they are the options to be adopted unless there are very good reasons to do otherwise.

Complementary titles

P445 | Steelwork joint design – process and information



P427 | Structural steel reuse



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