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RECOMMENDATIONS FOR REUSE OF STEEL PRODUCTS

— Volume 2: Building design recommendations to facilitate future deconstruction and reuse —

Technical Committee 14
Sustainability& Eco-Efficiency of Steel Construction,
in the frame of of European RFCS
ADVANCE Project

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ECCS TC14

Sustainability & Eco-Efficiency of Steel Construction

Recommendations for Reuse of Steel Products

Volume 2: Building design recommendations to facilitate future deconstruction and reuse

2nd Edition, 2025

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FOREWORD

Using reclaimed structural steel members on a project is an effective strategy to reduce the environmental impact of a building by eliminating the energy required to recycle steelwork into new products by melting the material. The research project *PROGRESS* (*Provisions for a greater reuse of steel structures*) focused on single-storey steel buildings, and it identified various reuse scenarios. It also showed how well thought out design measures can facilitate reuse of the structure or its primary components. The scope of the work was to extend from single- to multi-storey buildings within the research project ADVANCE (*Accompanying measure for dissemination, valorisation and collaborative exploitation of circularity of constructional steel products*) and the additional content is included in this second edition of the Recommendations for Reuse of Reclaimed Steel Products.

These recommendations address the key aspects designers need to consider facilitating greater reuse of steel structures and present examples of successful structural reuse. The recommendations outline the requirements for functional reusability, but do not cover in full detail the economic feasibility or environmental benefits of reuse.

The scope of reuse of structural steel is limited here to:

- Members to be reused should not be subjected to damages, inclusive plastic deformations, and reduced cross sections (e.g. through holes, openings, cracks or excessive corrosion),
- All members to be reused should come from a building structure built with elements produced in or after 1970, which is about the time when the Limit State design became common practice,
- All salvaged primary members are rolled steel sections. Welded and built-up members are not included in the scope of this document,
- For the members to be reused, they must be recovered in as much of their original shape as possible, although some additional fabrication and preparation work may be required.

The recommendations are divided into three volumes:

Volume 1: Reusing existing steel products and buildings,
Volume 2: Building design recommendations to facilitate future deconstruction and reuse,
Volume 3: Environmental aspects and practical implementation.

Volume 1 discusses general technical issues related to the structural use of reclaimed steel from existing steel and composite steel-concrete structures. It presents a brief description of the anatomy of single- and multi-storey buildings, classification of different reuse scenarios, an historical review of European codes of practice and product standards, selection and acceptance of materials, and their classification for “new” designs in accordance with the Eurocodes. It also discusses structural design aspects in terms of Limit States principles. The protocol for condition assessment, sampling and testing of reclaimed steel is given in Appendix A. The derivation of the modified partial factor for the buckling resistance of the reused steel members is presented in Appendix B.

Volume 2 covers the design of new buildings with the goals of functionality, ease of fabrication, demountability and future reuse, together with aesthetics. The general principles for the design for disassembly and reuse of steelwork. It defines the loads and combination of actions to be used in design calculations and proposes general improvements in construction details that facilitate future reuse.

Volume 3 presents the assessment of the environmental benefits of reusing reclaimed steel members and offers information on practical aspects of the fabrication and erecting of structures from reclaimed steel. Several case studies are presented in the last section of this volume, which illustrate the use of reclaimed steel structures in various EU countries and some of the technical issues that were overcome.

The members of PROGRESS and ADVANCE project consortia who contributed to the document are (order of the partners in the projects):

Petr Hradil	Finland	Véronique Dehan	Belgium
Ludovic Fülöp	Finland	Francis Grogna	Belgium
Sirje Vares	Finland	Carlos del Castillo	Belgium
Margareta Wahlström	Finland	Helena Gervasio	Portugal
Tiina Vainio-Kaila	Finland	Luis da Silva	Portugal
Michael Sansom	United Kingdom	Ari Ilomäki	Finland
Ana M. Girão Coelho	United Kingdom	Teemu Tiainen	Finland
Ricardo Pimentel	United Kingdom	Timo Koivisto	Finland
Mark Lawson	United Kingdom	Jyrki Kesti	Finland
Viorel Ungureanu	Romania	Břetislav Židlický	Czech Republic
Raluca Buzatu	Romania	František Wald	Czech Republic
Ioan Both	Romania	André Beyer	France
Dan Dubina	Romania	Amor Ben Larbi	France
Markus Kuhnhenne	Germany	Peetu Hirvonen	Finland
Dominik Pyschny	Germany	Maria Carrubba	Germany
Kevin Janczyk	Germany	Jie Yang	Luxembourg
Paul Kamrath	Germany	José Humberto Matias de Paula Filho	Luxembourg

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NOTATION

Lower case

c_{dir}	Directional factor (for wind load calculations)
c_{prob}	Probability factor
c_{r}	Roughness factor
c_{season}	Season factor (for wind load calculations)
c_0	Orography factor
h_{c}	Columns depth
h_{b}	Beam/rafter depth
h_{h}	Haunch depth
m	Group mean value
n	Exponent
p	Probability for an annual exceedance of the 10-minute mean wind velocity
q	Uniformly distributed load
q_{b}	Basic velocity pressure of the wind
q_{p}	Peak velocity pressure
q_{t}	Adapted characteristic value of the variable action for the design working life
q_{t0}	Characteristic value of the variable action for a design working life of 50 years
s	Snow load on the roof
s_{k}	Characteristic value of the snow load on the ground at the relevant site
s_{n}	Ground snow load with a return period of n years
t	Target design working life
t_0	Standard design working life of 50 years
v_{b}	Basic wind velocity
$v_{\mathrm{b,0}}$	Fundamental value of the basic wind velocity
$v_{\mathrm{b,class}}$	Basic wind velocity for European class
v_{m}	Mean wind velocity
z	Height above ground

Upper case

C_{e}	Exposure coefficient (snow load calculations)
C_{esl}	Coefficient for exceptional snow loads
C_{t}	Thermal coefficient (snow load calculations)
G	Permanent action
$G_{\mathrm{k,h}}$	Favourable permanent action h
I_{v}	Turbulence intensity
K	Shape parameter depending on the coefficient of variation of the extreme-value distribution
K_{FI}	Multiplication factor
$K_{\gamma_{M1}}$	Correction factor
L	Span
L_{column}	Columns length
L_{h}	Fabricated haunch segments length
L_{a}	Fabricated apex segment length
P_{n}	Annual probability of exceedance
Q	Variable concentrated load, variable action
$Q_{\mathrm{k,1}}$	Leading variable action
$Q_{\mathrm{k},j}$	Accompanying variable action *i*
S	European snow load class I;
V_{x}	Coefficient of variation
W_i	European wind class i

Greek letters and symbols

α	Coefficient of linear thermal expansion
α_{cr}	Factor by which the design value of the loading would have to be increased to cause elastic instability
β	Reliability index
γ_{M1}	Partial factor for resistance of members to instability
$\gamma_{\mathrm{M1,mod}}$	Modified partial factor for resistance of members to instability
μ_{i}	Snow load shape coefficient
ρ	Air density

ψ_0 Combination factor for variable action

$\psi_{0,i}$ Combination factor for variable action *i*

Subscripts

d Design value

k Characteristic value

Abbreviations

CC	Consequence Class(es)
CEN	European Committee for Standardisation
CEV	Carbon equivalent value
DC1	Low ductility class systems for seismic design according to EN 1998-1
DfD	Design for deconstruction
STR	Design values of actions for strength
EHF	Equivalent horizontal forces
EN	European Norm
ETA	European Technical Assessment
EU	European Union
EXC	Execution Class(es)
hEN	European Harmonised Standard
ID	Identification; Identity.
NA	National Annex
RHS	Rectangular Hollow Sections
SLS	Serviceability Limit State(s)
STC	Steel-timber composite flooring system
STR	Design values of actions for strength
ULS	Ultimate Limit State(s)

Axes

x Longitudinal axis along the member

y Major axis (parallel to flanges)

z Minor axis (parallel to web)

1 Introduction

1.1 Scope of this publication

This second volume of the *Recommendations for Reuse of Steel Products* provides design guidance aimed at facilitating the future deconstruction and reuse of steel elements in building construction. In line with circular economy principles and sustainability goals, the guide addresses the design of new buildings, including single-storey industrial facilities and multi-storey residential or office buildings, with a focus on modularity, demountability, and material longevity.

It presents practical strategies for designing structures that maximise the reuse potential of steel members by promoting standardisation, efficient construction detailing, and the use of accessible mechanical connections over permanent welding.

The publication emphasizes early integration of design for disassembly considerations, careful material selection, and comprehensive documentation practices, including the use of Building Information Modelling (BIM) for long-term traceability.

Additionally, it provides technical recommendations on load combinations, structural analysis, and detailing best practices that enhance the structural integrity and reusability of building components.

By addressing these aspects, the publication aims to minimise environmental impacts, reduce waste, lower embodied carbon, promote the circular use of construction materials and foster a more sustainable and resource efficiency construction industry.

1.2 Terms and definitions

For the purposes of this guide, the following terms and definitions have been used with specific reference to single-storey buildings.

Cladding	Façade and roof elements that cover the structure to form the building envelope and provide the required thermal and air-tightness performance to the building.
Component	Part of a steel structure, which may itself be an assembly of several smaller components, e.g. trusses, sandwich panel.
Consequences classes	Classification based on the consequences of failure or malfunction of the structure; different reliability indices are associated with each consequences class.
Constituent product	Materials or products used in manufacturing with properties which are used in structural calculations or otherwise relate to the mechanical resistance and stability of works and parts thereof, and/or their fire resistance, including aspects of durability and serviceability.
Constructional steel	Generic term to denote the steelwork (primary and secondary) and steel-based cladding.

Deconstruction (or disassembly, or demounting)	Deconstruction is the process of taking a building apart into its components in such a way that they can be readily reused; it minimises the destructive aspects of the process of demolition, by preserving components and materials.
Demolition	Process whereby a building is taken apart with little or no attempt to recover any of its constituent parts for reuse; products of demolition may, however, be recycled.
Design for Deconstruction (DfD)	Designing for deconstruction is a decision-making process at the design stage as to how a building can be taken apart and potentially reused.
Design working life	This is the assumed period for which the component is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.
Distributor	Any natural or legal person in the supply chain, other than the manufacturer or the importer, who makes a construction product available on the market.
Envelope	The enclosure around the building that separates the enclosed space from the external environment and provides a range of structural and building physics functions.
Execution class	Classified set of requirements specified for the execution of the works as a whole, of an individual component or of a detail of a component.
Façade	See cladding
Floor	Part of the structure with the function of providing the useful space in the building. Structurally, it transfers loads to the columns and walls and provides stability in the horizontal plane of the storeys, contributing to the global stability of the structure.
Hazardous waste	Waste that poses substantial or potential threats to public health or the environment.
Importer	Any natural or legal person established within the EU and who places a construction product form a third country in the EU market.
In-situ reuse	The component or structure is reused, without displacement, on the same site. For example, a building structure can be retained and reused during building renovations.
Manufacturer	Any natural or legal person who manufacturers a construction product or who has such a product designed or manufactured and markets that product under his name or trademark.
Notified body	A Notified Body is an independent (non-governmental) third-party body, recognised by the EU/EEA and is authorised to carry out conformity assessments for products that meet the requirements

	of a harmonised standard (hEN) or European Technical Assessment (ETA).
Pre-deconstruction audit	Qualitative and quantitative assessment of construction and demolition waste streams prior to deconstruction, demolition, or renovation of buildings and infrastructures.
Primary structure	The primary steel frame, comprising all main load bearing elements, e.g. columns, beams and bracing.
Purchaser	The company purchasing the steel products; generally, this is a steelwork contractor that executes the structural steelwork.
Reconditioning	Process of returning a product to good working condition by replacing major components that are faulty and making changes to update the appearance of a product, such as by cleaning, painting or refinishing.
Recycling	Process of converting waste materials into new materials and products; recycling steel involves re-melting of scrap to form new semi-finished products.
Refurbishing	This means the process of renovation of an existing building to suit a new use and can involve a range of processes from replacing fitments and fittings to major structural alterations.
Relocated reuse	Requires transport of the structure or component to reuse it on another site.
Remanufacturing	Returning a product or component to the performance specification of the original equipment manufacturer.
Repairing	Fixing a fault but with no guarantee of the product as a whole. In the context of steel structures, this can mean strengthening a component, often by welding plates.
Repurposing	Any operation that changes the function or purpose of a component.
Reuse	Use of old components with little or no reprocessing, largely in their original form; they may be reused for the original function (a conventional reuse scenario), or repurposed.
Reversible joint	Type of mechanical or structural connection that can be disassembled without damaging the components involved.
Secondary structure	The secondary steelwork, consisting of side rails and purlins for the walls and the roof, respectively, used to support the cladding/envelope and to provide restraint to the primary structure.
Structural component	Component used as a load-bearing part of the structure that is designed to provide mechanical resistance and stability and/or fire resistance, including aspects of durability and serviceability.

	The component is often part of a load-bearing steel structure, which may itself be an assembly of several smaller components.
Structural kit	Set of standardised structural components that are assembled and installed on site.
Supplier	The company stocking and supplying the steel products to the market.
Waste	Unwanted or undesired material to be discarded.

2 Recommendations for future buildings

Volume 2 addresses the ways in which new structures can be designed to facilitate greater reuse of steel structures. The focus is on steel members and secondary components used in single- and multi-storey steel buildings such as industrial buildings, large retail units, warehouses, residential and office buildings, and how they may be designed during their first cycle of use to allow easy demounting, enabling their reused in future buildings. Steel, being durable, recyclable, and versatile, offers significant opportunities for reuse if properly integrated into the design of the building. This also covers the connections between structural and non-structural elements, as presented in Section 4.

2.1 General principles of design for disassembly and reuse

The design of steel structures for future deconstruction and reuse is in accordance with the principles of circular economy, reduces waste, and maximises resource efficiency. Integrating these recommendations early in the design process allows the creation of adaptable, resilient, and sustainable buildings. By prioritising modularity, documentation, material selection, and accessible connections, the construction industry can significantly reduce its environmental impact and pave the way for more innovative and sustainable design practices.

The amount of steelwork that can be reclaimed and reused from buildings at the end of their service depends on how they were originally designed and constructed. This section discusses how decisions at the design stage can enable disassembly and therefore increase the quantity of salvaged and reused materials for subsequent use cycles.

In the design for disassembly and potential reuse, the following principles should be adopted [1]:

- Buildings should be built in layers that can be easily replaced throughout the life of the building. The components with the shortest lifetimes should be in the most easily accessible layers,
- The complexity of the building should be reduced as much as possible. Design using simple structural grids with clear support lines leads to use of regular-sized components which maximise their potential reuse with minimum variation. The number of different materials and their specifications should also be kept to a minimum to facilitate reuse,
- Work safety and space for machinery should be considered during construction and demounting. The design should also consider future deconstruction logistics,
- Prefabricated components, or modules, that are installed on site are more easily disassembled for reuse in other locations or even on the same site,
- Connection details should be simple and accessible. This also applies to the connections of the foundations and other components. Welding should be avoided except if the welded components can be reused in their entirety, e.g. portal frame rafters,
- Fittings, fasteners, adhesives and sealants should be selected so as not to damage the secondary components, such as cladding and windows, during their removal as potentially reusable components,
- Designs should maximise reusable materials and avoid composite materials, plaster, reinforced concrete, etc., which are difficult to separate and recycle. Hazardous

materials should be avoided. Also, the effect of coatings and fire protection for steelwork should be considered in a reuse application,

- A building log/data base should also be prepared in the form of a Building Information Model (BIM) that includes information on the design of the original building, the specifications for materials and construction details of any refurbishment work, and also information relevant to dismantling.

By focussing on modularity, material documentation, and smart connection techniques, architects and engineers can create structures that not only serve their immediate purpose but also offer long-term environmental and economic benefits.

2.1.2 Key Design Principles

Modular design stands out as one of the most effective strategies to ensure that steel structures can be disassembled and repurposed with minimal waste. By using standardised components and prefabricated modules, buildings can be assembled in a way that promotes future reuse. Bolted and reversible joints are preferred to welded connections, as they simplify the disassembly process. Prefabricated modules can be easily removed, transported and reintegrated into new projects, extending the lifecycle of steel components beyond the lifetime of the original structure.

Documentation and labelling are essential to facilitate future deconstruction efforts. Maintaining detailed records of all structural components, including steel grades, dimensions, and connection details, allows easier identification and reuse. Digital tools such as Building Information Modelling (BIM) offer robust solutions for tracking and managing materials throughout the building lifecycle. Additionally, physically labelling steel members with durable tags provides quick reference during deconstruction, ensuring that materials are properly identified and reused in the most effective way.

Material selection plays a crucial role in designing for future reuse. Where possible, structural steel should be sourced from steel producers who have defined and are implementing a strategy to reduce greenhouse gas emissions and have made a public commitment to decarbonise in line with national and/or international carbon reduction targets. This includes but is not limited to:

- An emissions reduction pathway compatible with the goals of the Paris Agreement;
- A validated science-based target, for example a target approved by the Science Based Target Initiative (SBTi);
- ResponsibleSteel Certified Steel or steel meeting an equivalent international standard.

Where possible, it is preferable to minimize the use of composite materials or coatings that may hinder recycling, as they can complicate the reclamation process. Standardised steel sections should be prioritised to simplify repurposing and ensure that components can fit perfectly into new projects without extensive modifications.

Connection and fastening techniques significantly influence the ease of deconstruction. Mechanical fasteners, such as bolts, are preferable to permanent methods such as welding or adhesives. Designing connections to be accessible and easily unfastened allows for quick dismantling. This not only accelerates the deconstruction process but also ensures that steel components can be reused with minimal damage.

Structural simplicity and redundancy are vital to creating steel buildings that are easy to deconstruct. Simplifying structural layouts reduces the complexity of connections and minimises overlapping components, making deconstruction more straightforward. Redundancy in design can allow parts of the structure to be disassembled without compromising the overall stability, enabling selective reuse.

2.1.3 Construction Phase Strategies

Designing for future deconstruction requires careful planning and proper training before construction begins. The construction teams should be educated in best practices for assembling structures with future disassembly in mind. Quality control during assembly ensures that connections are correctly installed, facilitating ease of disassembly.

2.1.4 Post-Use and End-of-Life Planning

End-of-life planning for steel structures should begin during the initial design phase, by applying the design strategies outlined above and documenting them. Incorporating deconstruction considerations into the design phase ensures that buildings are created with long-term reuse in mind, minimising waste and contributing to a more circular economy. At the end of a building's service life, deconstruction plans should be developed in collaboration with demolition or deconstruction companies to streamline the process. Identifying potential secondary uses of steel components helps extend their lifecycle and reduce the demand for new materials.

2.2 Standardisation

Most steel components are designed and manufactured for the specific requirements of a particular project to meet the needs of the client. *Value for money* is a client requirement and the whole-life cost of a building should be kept to a minimum consistent with a given quality. Currently, this whole-life cost does not include environmental impact costs associated with products throughout their life cycles and does not include the impacts of disposal or recycling at the end of life, and the associated CO_2 emissions.

If a holistic approach to costs and environmental impacts is required, then reuse of materials becomes an attractive solution, as it can lead to a lower cost as compared to use of new materials but has an additional cost of deconstruction and the subsequent handling and reconditioning of the reclaimed materials.

Standardisation is a way forward to maximise the potential for reuse of structural members as it can help in the selection process and availability of the reclaimed members.

Standardisation can be defined as the extensive use of processes, products, or components in which there is a desire to achieve regularity and repetition. Standard buildings are made to regular dimensions and with multiple identical components that achieve economies of scale in manufacture.

In case of ***single-storey steel buildings***, there are cost-benefits to be realised from dimensional coordination, and the following proposals can be made regarding the dimensional form:

1. The length between member splice points is generally limited by road transportation. The usual stock lengths are 10, 12, 14, 15, 16, 18 and 20 m. Lengths of 12 m are

generally transportable by lorry and lengths up to 18 m are possible depending on the local roads to the site. For containership transport, a total length less than 12 m is usually required,

2. The slope of the roof depends on local snow and rain and building practice in the region. A slope of at least 1:10 (6°) is normally specified for pitch roof portal frames; a slope of 1:20 (3°) is often used for pitch roof trusses, as they are stiffer and deform less than in portal frames,
3. Frame spacing is typically 5 to 8 m, depending on the span. The common dimensions for standardisation can be taken as 7.5 m for low snow regions and 5 or 6 m for high snow regions. In reuse scenarios, it is possible to use different frame spacings compared to the original building,
4. Typical spans and span-to-depth ratios for the primary roof members in single-storey buildings are given in Table 2.1 adapted from [2].

Table 2.1 Typical spans and span-to-rafter depth ratios for single-storey buildings

Forms of framing	Typical span range	Roof beam depth
Simple construction		
Rolled section beams	up to 20 m	span/25 to span/35 based on member sizes and weights
Fabricated beams	up to 30 m	span/20 to span/25
Castellated or cellular beams with web openings	up to 45 m	span/18 to span/30 depending on the size of the openings
Truss roof (pitched)	up to 20 m	span/5 to span/10 based on the height at the top of the truss
Truss roof (flat)	up to 100 m	span/15 to span/20
Continuous construction		
Portal frame	15 m to 50 m	span/50 to span/65 for the rafters (up to span/85 if snow load doesn't govern the design)
Single pitched roof	up to 25 m	
Propped portal	up to 50 m	
Fabricated tapered profiles	up to 70 m	Between span/25 to span/65
Truss roof (flat)	up to 100 m	span/15 to span/20

In the preliminary design of single-storey steel buildings with eaves heights ranging from 6 to 12 m, it is standard practice to adopt frame span-to-column depth ratios between 40 and 50. This guideline provides a balanced approach, ensuring both structural efficiency and economic material usage.

In ***multi-storey steel structures***, load-bearing and load distribution are ensured by a main frame consisting of beams and columns. Optimisation of the number of load points is a question in the design stage and the use of the response must consider the use of the building [3]. As far as the layout of space is concerned, columns are considered obstacles that must be limited as much as possible. Traditional framework structures use spans of the order of 4.5 to 6 m for residential buildings. Large spans of between 12 and 18 m for offices and 15 to 16 m for car parks can free up additional space.

Floor grids define the spacing of the columns in orthogonal directions, which are influenced by:

- The planning grid (normally based on units of 300 mm but more typically multiples of 0.6, 1.2 or 1.5 m);
- The column spacing along the façade, depending on the façade material (typically 5.4 m to 7.5 m);

- The use of the internal space (i.e. for offices or open plan space);
- The requirements for building service distribution (from the building core).

Along the façade line, column spacings are normally defined by the need to provide support to the cladding system (for example, a maximum column spacing of 6 m is normally required for brickwork). This influences the column spacing internally, unless additional columns are used along the façade line.

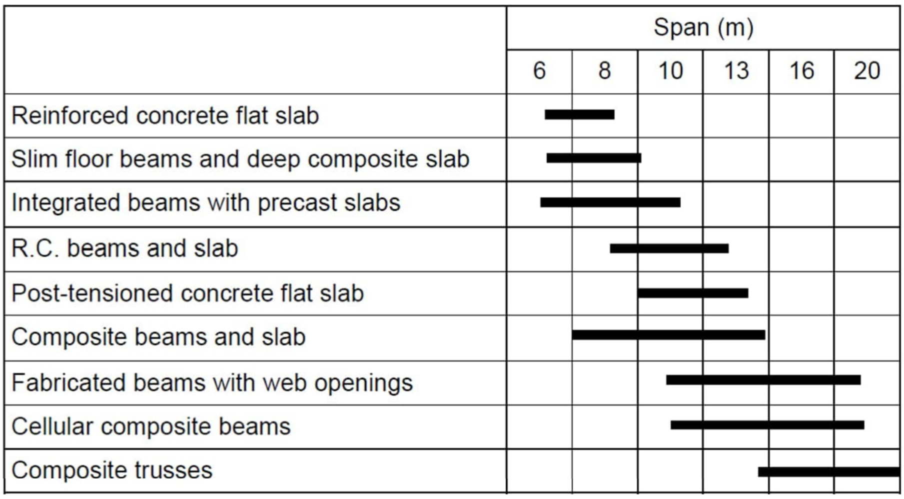
For naturally ventilated offices, a building width of 12 to 15 m is typically used, which can be achieved by two spans of 6 to 7.5 m. A single span can also be provided with deep (400 mm or more) precast concrete hollow core units spanning the full width of the building. Natural lighting also plays a role in the choice of the width of floors. However, in modern buildings, a long-span solution provides a considerable enhancement in flexibility of layout. For air-conditioned offices, a clear span of 15 to 18 m is often used.

The target floor-to-floor height is based on a floor-to-ceiling height of 2.5 to 2.7 m for speculative/usual offices or 3 m for more prestige applications, plus the floor depth including services. The following floor-to-floor depth targets should be considered in the concept design stage:

- Prestige office 4 – 4.2 m,
- Speculative office 3.6 – 4.0 m,
- Renovation project 3.5 – 3.9 m.

The span ranges for the various structural options for floors are shown in Table 2.2.

Table 2.2 Span range of various structural options [3]



The following information on floor grids is provided, based on national practices:

- Germany (according to DIN): planning grid of 100mm, typically multiples of 1.2 m,
- Netherlands: planning grid of 100mm, multiples of 1.8 m for schools, retail, hotels and office buildings,
- UK: planning grid of 300mm; typically, multiples of 0.6, 1.2 or 1.5 m. Schools and medical buildings are designed for multiples of 1.2 m and offices for a floor grid of 1.5 m,

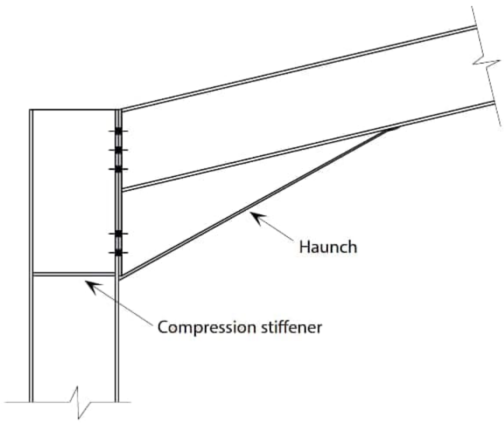
- In general, multiples of 1.25 and 1.35 m are accepted in Europe (1.35 m may be preferred for standardisation of dimensions),
- Low-rise office buildings (2-4 storeys): 6 to 9 m grids are often used to suit the use of precast concrete floor slabs on down-stand beams or possibly slim floor type beams,
- For high-rise office buildings in the UK, rectangular floor grids are preferred, in which the longer span extends 13.5 to 18 m across the building in steps of 1.5 m. A column grid of 16.5 m $\times$ 7.5 m is compatible with basement car parking,
- These long span beams generally have web openings for integration of the building services, such as cellular beams made from rolled sections, which have multiple circular openings.

For high-rise buildings, a planning grid of 1.35 m leads to a column spacing of 16.2 m $\times$ 8.1 m, which is a 2:1 grid that may be useful in planning of buildings for adaptability.

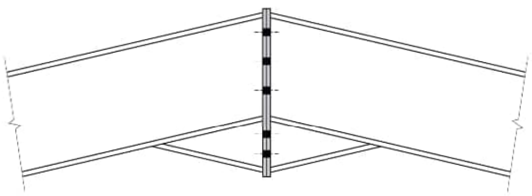
2.3 Best practice for analysis and design of single-storey steel buildings

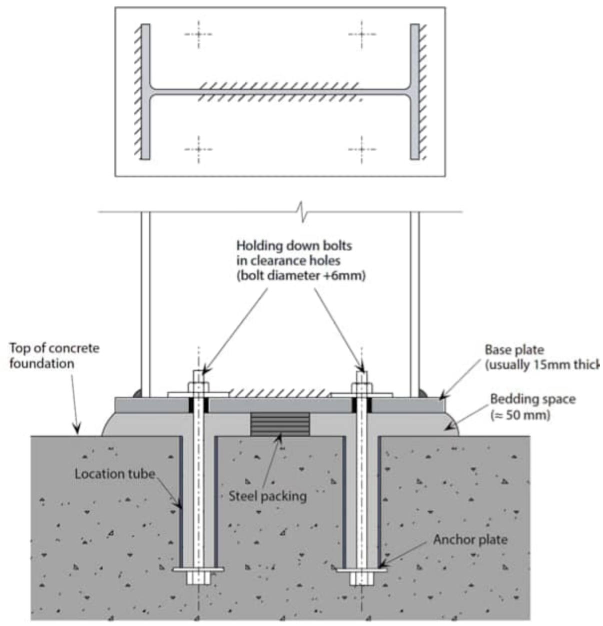
2.3.1 Typical detailing for portal frames

The typical details for a portal frame with nominally pinned base are shown in Fig. 2.1.



Typical eaves connection (UK)





Typical apex connection (UK)   Typical nominally pinned base connection (UK)

Fig. 2.1 Typical detailing for a single bay portal frame with nominally pinned bases [4]

2.3.2 Bracing systems

A portal frame building has two types of primary bracing systems: (i) vertical bracing and (ii) roof bracing. The primary functions of vertical bracing in the side walls of the frame are:

- To transmit horizontal forces to the foundations,

- To provide stability during erection.

The bracing may be located at one or both ends of the building (see Fig. 2.2), within the length of the building or in each portion between possible joints (where these are present). Braced bays or framed bays can be used for this purpose. Their position may also be influenced by the layout of the building. The eave struts ensure that all portal frames are braced in the out-of-plane direction by the vertical bracing system.

The roof bracing system is located in the plane of the roof, typically at both ends of the building between the two first adjacent frames (Fig. 2.3). The primary functions of the roof bracing are:

- To transmit the wind forces from the gable posts to the vertical bracing in the walls,
- To transmit any frictional forces from the wind on the roof to the vertical bracing,
- To provide a stiff anchorage for the purlins that are used to restrain the rafters,
- To provide stability during erection,
- To restrain the tops of internal columns by bracing back to perimeter walls.

The roof bracing system should be arranged to provide support at the top of the gable posts.

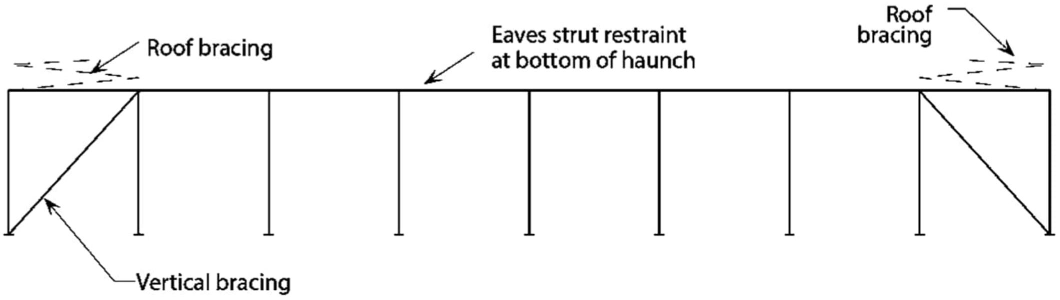


Fig. 2.2 Typical vertical bracing arrangement [5]

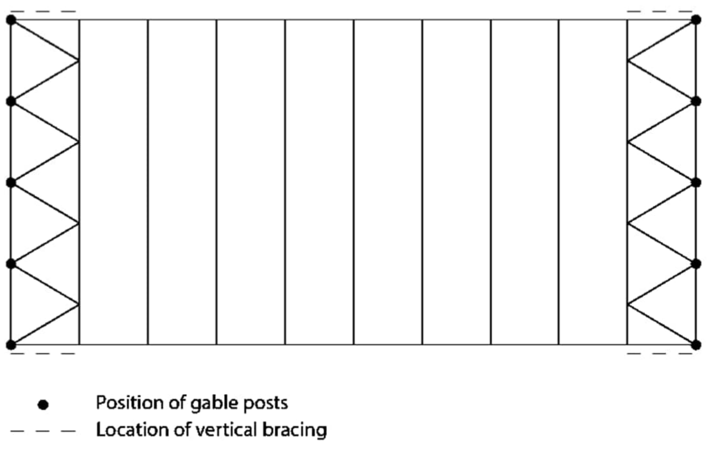


Fig. 2.3 Typical roof bracing arrangement [5]

2.3.3 Gable frames

The end gables are often designed using intermediate supports to the rafters, which are therefore lighter than the main structural frames. Another alternative is to use braced end-gable frames, which may lead to some challenges for cladding performance due to the differential displacements between subsequent frames. It is recommended that the end-gables are the same size as the internal frames to facilitate reuse and also to allow for future building extensions. This practice will also offer an improved performance of the cladding system, since a less significant differential displacement is expected between two adjacent frames.

2.3.4 Global analysis

In Europe, most portal frames are designed using elastic analysis, whereas in the UK they are designed using plastic analysis at the ultimate limit state, but with additional checks on deflections using elastic design. To facilitate the reuse of structures, it is recommended to use an elastic design for the first use and subsequent uses.

The sizes of the members in the plastic design will be smaller than that of the global elastic design, due to the redistribution of the bending moments within the frame. However, the additional cost is minimal, since material costs account for less than 50% of the total installation cost.

2.3.5 Behaviour of column-base joints

Joints/connections can be classified as nominally pinned, semi-rigid, or rigid according to EN 1993-1-8 [6]. For preliminary design purposes, it may be assumed that a nominally pinned connection, following the typical detail presented in Fig. 2.1, may be assumed to provide 10% of the bending stiffness of the frame columns for a global stability analysis and 20% for serviceability checks. For nominally pinned joints, a base stiffness of up to 20% of the stiffness of the column can be assumed [7].

For the final design, if a column is rigidly connected to a suitable foundation, the stiffness of the base connection should be taken as equal to the stiffness of the column for all ultimate limit state calculations. For SLS checks, the base may be treated as rigid [7]. For semi-rigid joints, it is recommended that the rotation spring stiffness be assessed according to the EN 1993-1-8 or by appropriate software.

Common software packages allow for the direct incorporation of a rotational spring stiffness into the model, which facilitates the implementation of the above recommendations. Particularly for frame bases, if the analysis software does not accommodate rotational springs, the base fixity may be modelled by dummy members of equivalent stiffness as shown in Fig. 2.4.

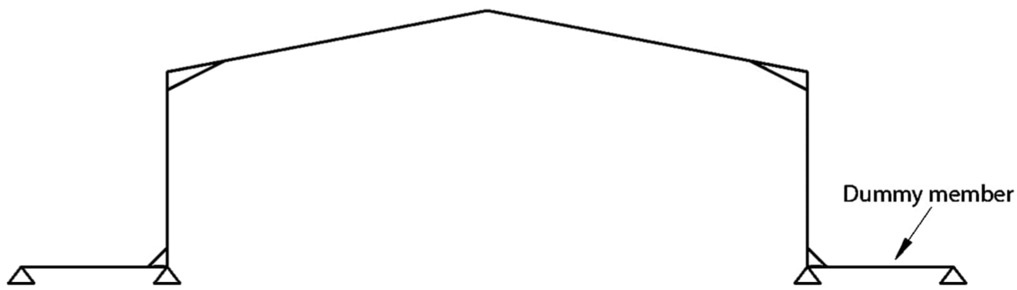


Fig. 2.4 Modelling base fixity by a dummy member [4]

2.3.6 Member buckling design

The member buckling design should follow the procedures in clause 8.3 of EN 1993-1-1.

Secondary steelwork has an important role for the design of portal frames, to provide restraint to flexural, torsional and lateral torsional buckling of members. A typical portal frame design will rely on the minimum torsional restraints proposed in Fig. 2.5. For the wind uplift condition, additional torsional restraints may be necessary to the internal compressed flange of the rafter (see Fig. 2.6) or for columns on a façade subjected to wind suction.

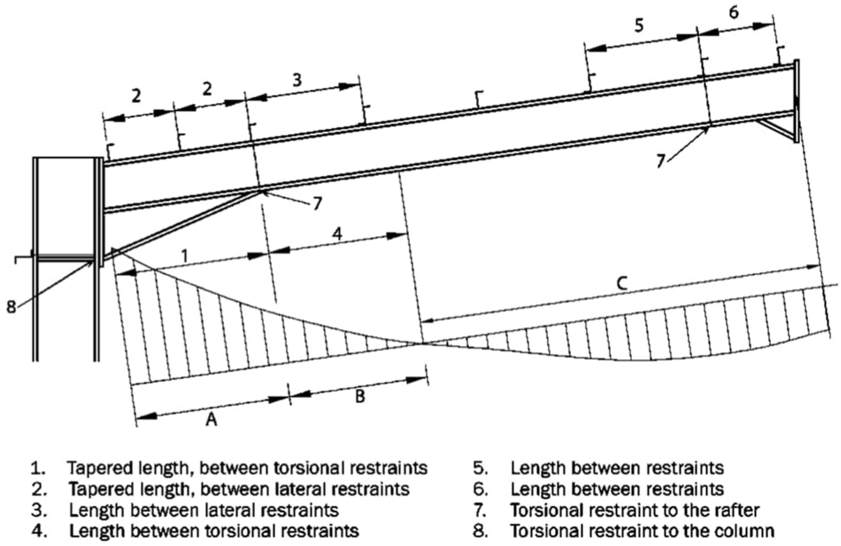


Fig. 2.5 Typical restraints arrangements on a portal frame: gravity load [4]

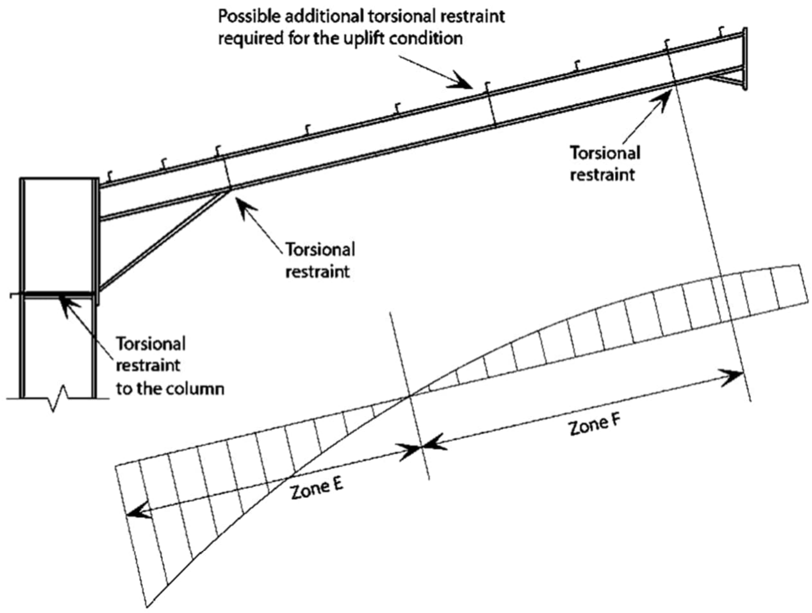


Fig. 2.6 Typical restraints arrangements on a portal frame: uplift [4]

2.3.7 Stress checks

There is no requirement to check the service stresses according to EN 1993-1-1 [8]. However, since the deflection calculations are based on elastic analysis, plasticity should not occur at the serviceability limit states (SLS). If the structure is designed according to global plastic analysis, it is recommended that a stress check is performed for the characteristic serviceability load combinations according to EN 1090 [9].

2.3.8 Deflection checks

Deflection check criteria may be established for a specific project or local practice. The recommendations from Section 6.5.1 of Volume 1 may be followed.

2.3.9 Truss solutions

By using truss-type structures, a comparatively high stiffness and load bearing resistance structure can be achieved, while minimising material use (the savings tend to increase with increasing span). Especially in Nordic countries, rafters with trussed solutions are usually used, as heavy roof loads result from snow.

In addition to the ability to create long spans, lattice structures are attractive and enable simple service integration. Trusses often use hollow sections for columns and rafters, but an open section may also be used. In a lattice structure, the high buckling resistance of hollow sections enables the use of long spans and larger spacing between diagonals. Due to the superior torsional stiffness of the closed section, lattice structures made of hollow sections have good resistance to lateral torsional buckling. For hollow sections, the flexural buckling of the members typically governs the design of the members. The fabrication of standard joint details is cost efficient, while rounded corners and easily accessible joints facilitate pretreatment.

For long span claddings (such as built-up deep sandwich panels), the top chord of the truss may be assumed to be restrained. For wind-induced uplift loads on the roof, the bottom chord needs to be restrained by a longitudinal roof bracing. For continuous roof trusses, bracings may also be necessary to restrain the compressed bottom chord close to the columns.

In Nordic countries, welded hollow section trusses represent the most typical truss girder. In long spans, the girders are usually manufactured and transported in two pieces with bolted connections in the upper and lower chords.

2.4 Best practice for analysis and design of multi-storey steel buildings

2.4.1 Typical structural systems

The structural system required for stability is mainly influenced by the building height [10]. For buildings up to 8-storey high, the steel structure may be designed to provide stability, but for taller buildings, concrete or braced steel cores are more efficient. The following structural systems can be considered for stability.

For buildings up to 4-storey high, rigid frames may be used in which multiple beam-to-column connections provide bending resistance and stiffness to resist horizontal loads. This is generally only possible where the beams are relatively deep (400 mm to 500 mm) and where the column size is increased to resist the applied bending moments. Full-depth end-plate connections generally provide the necessary rigidity.

For buildings up to 12-storey high, braced steel frames are commonly with X, K, or V bracing in the walls; generally, within a cavity in the façade, or around stairs or other serviced zones. Cross bracing is designed in tension only (the other member being redundant). Cross bracing is often simple flat steel, but angle sections and channel sections may also be used.

When bracing is designed to work in compression, hollow sections are the most popular, although angle sections and channel sections may also be used.

Steel braced frames have two key advantages:

- A single construction team is responsible for ensuring temporary stability,
- As soon as the steel bracing is connected (bolted), the structure is stable.

Concrete cores are a practical system for stabilising buildings up to 40-storeys high. The concrete core is generally constructed in advance of the steel framework. In this form of construction, the beams often span directly between the columns on the perimeter of the building and the core. Special structural design considerations are required for:

- The beam connections to the concrete core,
- The design of the heavier primary beams at the corner of the concrete core,
- Fire safety and robustness of the long-span construction.

Special attention must be paid to the connections between the steel beams and the concrete cores, allowing adjustment, anticipating that the core may be out of position. The connection itself may not be completed until *in situ* concrete has cured or until elements have been welded, so attention to temporary stability is important.

Columns in multi-storey steel frames are generally H sections, predominantly carrying axial load. When the stability of the structure is provided by cores, or discreet vertical bracing, the beams are usually designed as simply supported. The generally accepted design model is that nominally pinned connections produce nominal moments in the column, calculated by assuming that the beam reaction is 100 mm from the face of the column.

For ease of construction, columns are usually erected in two-, or sometimes three-storey sections (i.e., approximately 8 m to 12 m in length). Column sections are joined with splices, typically 300 mm to 600 mm above the floor level.

There is a wide range of floors [3]. Although steel solutions are appropriate for short spans (typically 6 to 9 m), steel has an important advantage over other materials at long spans (between 12 and 18 m). This gives key advantages of column-free space, allowing future adaptability, and fewer foundations.

Floor slabs spanning over the steel beams will normally be either precast concrete units, or composite floors. The steel beams can be located below the floor slab, with the floor bearing on the top flange (often known as “downstand” beams). The beams may be non-composite or composite. In composite construction, shear connectors are welded to the top flange of the beam, transferring load to the concrete floor. Shear connectors are often welded on site to the top flange of the beam that has been left unpainted, through the steel decking (known as “through-deck” welding). The slab may also be placed within the depth of the beams to reduce the overall height of the floor; such beams are known as slim floor beams, or integrated beams. The available depth of the slabs is often the determining factor when choosing a floor solution. For instance, in high-rise buildings, the total depth of the 30-40 floor slabs can add up to a significant unutilised construction volume.

2.4.2 Global analysis

Low-rise buildings (of two- or three-storeys) are only subject to modest horizontal forces and may easily be constructed with robust enough bracing systems such that second-order effects are minimised, to the extent that sway stability effects need not be considered explicitly in design. The bracing may be provided either by triangulated bracing or by reinforced concrete core(s). The floors act as diaphragms to tie all columns to the bracing or cores.

Medium-rise steel frames are defined as frames where neither resistance to horizontal loads, nor achieving sufficient sway stability, has significant impact on either the plan arrangement of the floors or the overall structural form. Concrete/steel cores or braced sections are usually used to provide horizontal stability. This limit is normally regarded as twelve storeys.

An unbraced frame is a frame that does not have either a concrete core or a complete system of vertical triangulation, provided primarily for resisting horizontal loads. In unbraced frames, at least some beam-to-column connections must be moment resisting in order to transmit horizontal forces to the foundations and to provide frame stability.

A braced frame has structural components explicitly provided for the purpose of transmitting horizontal forces to the foundations. These components increase the horizontal stability of the frame. They may be one or more concrete cores, which will usually contain the vertical services, lifts and stairs. Alternatively, they may be complete systems of triangulated steel bracings in vertical planes (acting in conjunction with floor diaphragms or horizontal bracing). In a braced frame, the beams may be designed as pinned at the ends. The columns carry axial loads and (generally) minimal moments. The beam-to-column connections can be designed as nominally pinned, and hence not attracting bending moments; sufficient rotation capacity must be provided.

The general design procedure is as follows:

1. Determine the ULS vertical actions,
2. Calculate the equivalent horizontal forces (EHF) to allow for imperfections,
3. Determine the ULS horizontal loads,
4. Determine the total horizontal loads (from 2 and 3 above),
5. Choose bracing configuration and choose bracing members (i.e., if any), based on the total horizontal loads,
6. Carry out 1st order analysis to determine the internal forces and the sway stiffness of the frame,
7. For each floor of each braced bay, determine the α_{cr} from the most demanding combination of vertical loads,
8. Determine the governing α_{cr} as the lowest value obtained from the analysis above,
9. Based on the α_{cr} obtained, decide if the 1st order analysis is sufficient, or additional analysis is needed to consider 2nd order sensitivity,

where α_{cr} is a factor by which the design value of the loading would have to be increased to cause elastic instability

First-order analysis can be used if the EN 1993-1-1 requirement of $\alpha_{cr} \geq 10$ for the whole frame and therefore for each storey of a multi-storey building is fulfilled.

Buildings have to be designed for the combinations of actions set out in EN 1990. Design for the Ultimate Limit State (see EN1993-1-1), i.e., verification of the strength of all the structural components of the building to resist the actions identified by the global analysis, remains the core of the detailed design process. EN 1993-1-8 [6] gives design methods for the design of joints subject to predominantly static loading using steel grades S235, S275, S355 and S460.

EN 1990 and EN 1993-1-1 require structures to satisfy the Serviceability Limit State. Criteria relevant to multi-storey buildings are: (1) Horizontal deflections; (2) Vertical deflections on floor systems; (3) Dynamic response.

2.5 Durability

Metallic coatings (hot-dip galvanised solutions) are less common compared to conventional paint coatings because they tend to be more expensive. When specifying paint coating systems for reusable buildings, designers may wish to consider a high or very high durability class for the paint system according to ISO 12944-1 [11]. However, as a paint coating system tends to be weaker than a galvanised solution, the latter is preferable for structures with possible multiple assembling and disassembling cycles. Hot-dip galvanised solutions should follow ISO 1461 [12] and ISO 14713 [13] to [15].

2.6 Documentation, identification and traceability for reuse

The main challenges for reuse are the uncertainties in the product and material properties and the consequent testing requirements. If material, fabrication and construction records are efficiently stored for future consultation, costs related to testing may be avoided. To facilitate the reuse of building structures, this information has to be documented, maintained throughout the life of the structure, updated when necessary, and clearly linked to the particular building components to allow future identification. The efficiency of the reuse process can further increase if the information is stored in a machine-readable form, such as Building Information Model (BIM). This section explains the basic principles of building information management and component identification.

2.6.1 Building memo

In order to facilitate future reuse of building structures that are currently being fabricated and erected, it is helpful to establish a building memo that would contain design information, declared and/or certified nominal properties such as:

- the steel characteristics (e.g., mill certificates, CE markings, Environmental Product Declarations),
- specification and drawings that meet what was offered in the tender,
- fabrication and erection drawings and documents,
- all connections between members and splices.

Results of measurement, assessment, testing, or inspections should be recorded, such as:

- identification of non-conformities, e.g. dimensional variations,
- reports of regular maintenance, changes and renovation work,

- pre-deconstruction audits if existing steelwork is being used,
- photo documentation.

The structural design documents for a building are based on consideration of the design loads and forces to be resisted by the structural steel frame, for any cycle of use, and clearly show and describe all elements of structural steelwork. They should also include the standards and codes that govern the design and construction, including bolting and welding. Any revision to these documents should be added to the originals, e.g. design modifications during erection.

It is recommended that the building owner maintains the building memo because it contains details of all products that constitute the building and is usually required to keep detailed maintenance records. This will ensure that the products within the building are properly maintained and, when replaced or deconstructed for reuse, fully comply with the new requirements.

The building memo can be linked to a digital representation of the building, for instance, its 3D architectural model, building information model (3D model with functional characteristics), or digital twin (3D model with functional characteristics and dynamic processes). Since the Building Information Modelling (BIM) is becoming widespread in the construction sector, it is discussed in more detail in the following section.

2.6.2 Building Information Modelling

For achieving mainstream reuse, digital information plays a key role in the process, as all relevant building data can be stored in a 3D digital model with Building Information Modelling (BIM) approach. The level of information that a 3D BIM model needs to accommodate is a responsibility of all project stakeholders. The ISO standards EN ISO 19650-1 [16] and EN ISO 19650-2 [17] introduce the concept of level or information need (LOIN), for which it is suggested that each project actor must define the relevant information to be stored for the purpose of the element in a specific project. The key concepts from a structural engineering point of view are proposed in Table 2.3 and Table 2.4.

Table 2.3 Proposed information categories for the definition of the LOIN: general definition

Category	Description
Context	For each life cycle: the context/time where/when the structural member has been used;
Project actors	For each life cycle: actors involved from relevant disciplines;
Purpose	For each life cycle: the purposes of the member;
Identification	For each life cycle: the identity of the structural steel member and its traceability to the digital information;
Structural design	For each life cycle: relevant design conditions and design outcome for the building and element;
Fabrication & erection	For each life cycle: records from fabrication and procedures and the quality of those procedures;
Provenance & characteristics	Full traceability of the member material, including records and certificates;

Table 2.4 Proposed information categories for the definition of the LOIN: possible relevant data

Category	Description
Context	For each life cycle: project description, site details, construction date, etc.
Project actors	For each life cycle: architects, engineers, contractors, etc.
Purpose	Features such as load bearing or non-structural, structural function (beam, column bracing), condition (permanent, temporary) etc.
Identification	For each life cycle: member identification number (ID), location (say floor number, bloc number), other relevant visual property; section serial size etc.;
Structural design	If the element belongs to a primary or secondary structural system (say according to prEN 1998-1-2), ductility class according to prEN 1998-1-2, fire rating, critical temperature, utilization factor and/or resistances, studs detailing, floor/element frequency/response factor/OS-RMS90, in-service deflections, project loading (loads on floors, wind action, snow load, etc.), type of connections (pinned, fixed or elastic – specify stiffness), maximum bending moment and shear forces on member and for connection design, tying forces etc.
Fabrication & erection	For each life cycle: fabrication company, fabrication date, standard for execution (say EN1090-2), execution class, fabrication records (project number), erection company, erection date, coating/galvanizing details (class, durability, thickens/mass)
Provenance & characteristics	New steel: producer, mill certificate number/ID, material product standard, delivery condition (EN 10204), steel grade, sub grade and Z quality, heat treatment delivery condition, geometry product standards, etc. Reclaimed steel: stockholder, reference standard (say EN 10025-2 or EN 10219-1), grade and subgrade, relevant properties according to EN 1090-2 section 5.1 (measured/determined values and design values), stockholder internal documentation reference, product standards (say EN 10365 and EN 10034) etc.

The proposals in Table 2.4 may be used as a reference to define the level of information stored for steel members as part of a BIM model. References [18] to [20] may be used to help establish the level of information need of the BIM model. The CWA 17316 guidance form [21] may be used to facilitate information exchange.

2.6.3 Traceability of steel products for future reuse

In order to avoid expensive verification of product and material properties, the connection between the physical products and their digital information (the tracking system) should be created. Typically, a component tracking system is implemented during the fabrication and erection processes. However, it is mostly not preserved during the lifetime of the building. It is recommended that a more durable system should be provided for the whole lifetime of the building and is linked with a digital model where the relevant information about the building and members can be kept. This measure will facilitate steelwork reuse without the need for further testing.

For this purpose, permanent labelling should be established, and the marks should be applied directly to the structural steel members. The marks should be unique for each group of the members with the same nominal characteristics, but it is recommended that the marks be different for each component, in order to link this component to the specific results of measurement, assessment, testing, or inspections.

Examples of permanent labels are laser-engraved plates with visible information or radio frequency identification (RFID) tags with information readable by RF scanner. Both methods should have a unique identifier that can be linked to the digitally stored information; for example, the laser-engraved identifier can be a QR code, a barcode, or just a simple identification code. Both methods can optionally contain the most essential information (such as CE marking) directly on the label. This can take the form of an engraved table of essential characteristics or data stored on the memory chip connected to the RFID antenna. An example of a QR code for a reclaimed element is presented in Table 2.5.

It is essential that the digital information (Building memo, BIM, etc.) is available throughout the lifecycle of the building and its components for the facility owner and relevant building authorities issuing demolition, renovation and building permits. The reliability of the information contained in declarations and certificates can be guaranteed for instance by using independent traceability systems (such as Tracimat in Belgium), databases managed by the building authorities, steel fabricators responsible for re-certification or blockchains.

Table 2.5 Proposed information to be stored on a permanent physical label

Example of possible QR code for component tracking: (Try me)	Type: Reclaimed Origin: UK, Ascot Steel Age: 1975 ID: C10 Fabricator: Name Designer: Name Stockholder: Name Stockholder Certificate: AA001 Steel Designation: S355JR Material Standard: EN1090-2 cl. 5.1 Design Yield (MPA): 355 Design Tensile (MPA): 470 Measured Yield (MPA): 405 Measured Tensile (MPA): 520 Measured Elongation (%): 23 Measured CEV: 0.45 Profile: IPE500 Dimensions: EN 10365 Tolerances: EN 10034

3 Loading and combination of actions for new buildings

Single-storey steel buildings are loaded by their self-weight, building service, wind and snow loads. They can be designed to support mezzanine floors and cranes, and often have additional attached office spaces, large canopies and other architectural features.

For multi-storey steel buildings, the imposed loads must be added and are represented by uniformly distributed loads, line loads or point loads applied on roofs or floors, or a combination of these loads.

Snow loads and wind loads are site specific (location, altitude). They are also influenced by the geometry of the structure and the local topography in the immediate vicinity of the building. In the case of single-storey steel buildings, these loads influence the reuse of a building with the same layout and frame spacing.

3.1 Characteristic values of actions

3.1.1 Loads on roofs

Permanent loads on roofs include the self-weight of the cladding, and service loads like ventilation and light fixtures, etc., in addition to the self-weight of steelwork. Typical self-weights of roofing components are shown in Table 3.1. A minimum of 0.10 kN/m^2 allowance is recommended for the secondary elements. For service loads, a nominal value of 0.30 kN/m^2 should be allowed for lighting units, pipes for sprinkler systems, air-conditioning ducts, etc., but also to allow for solar panels. Table 3.2 presents the recommended self-weights for different types of roof cladding.

Table 3.1 Self-weights of the roofing components

Type of secondary steelwork	**Weight (kN/m^2)**
Single skin roof sheeting (short span and long span decking)	0.04 – 0.20
Insulation (mineral wool per 100 mm thickness)	0.04 – 0.08
Insulation boards, per 25 mm thickness	0.07
Insulation glass fibre, per 100 mm thickness	0.01
Liner trays (0.4 mm – 0.7 mm thickness)	0.09 – 0.13
Sandwich panels (40 mm – 150 mm thickness)	0.10 – 0.15
Steel purlins/rails (distributed over the roof area)	0.04 – 0.07
Steel purlins	0.03 – 0.08

Table 3.2 Recommended self-weights for different types of roof cladding

Type of roof cladding	**Weight (kN/m^2)**
Lightweight sandwich panels (short spans, up to 100 mm thick)	0.15
Heavyweight sandwich panels (long spans, up to 200 mm thick)	0.35
Profiles steel sheeting, insulation and membranes (long spans)	0.60

The imposed loads on roofs are specified in Clause 6.3.4.2 (1) of EN 1991-1-1 [64] and the countries NAs. These loads are required for access only, for cleaning or maintenance (category H). The recommended values are q_k = 0.4 kN/m^2 and Q_k = 1.0 kN. They should not be added to snow or wind loads. The values generally adopted in European countries are summarised in Table **3.3** for roof slopes not greater than 6°.

Table 3.3 Imposed loads for maintenance of roofs (category H)

Country	q_k (kN/m^2)	Q_k (kN)
Belgium	0.80	1.5
Czech Republic	0.75	1.0
Finland	0.40	1.0
France	0.80	1.5
Germany	-	1.0
Ireland	0.60	1.0
Italy	0.40	1.0
The Netherlands	1.00	1.5
Norway	0.75	1.5
Portugal	0.40	1.0
Romania	0.50	1.0
Slovakia	0.75	1.0
Spain	0.40	1.0
Sweden	0.40	1.0
United Kingdom	0.60	0.9

3.1.2 Loads on mezzanine floors and floors in multi-storey buildings

For lightweight floor solutions, a self-weight load of 1 kN/m^2 is recommended to allow future adaptability. For rolled beams supporting heavyweight flooring solutions, such as precast planks, this self-weight may be 3 to 4.5 kN/m^2 depending on the slab span and thickness. A minimum allowance of 1.75 kN/m^2 is recommended for finishes and services. Table 3.4 presents the typical weights for the building elements of multi-storey buildings.

Table 3.4 Typical weights for building elements [22]

Element	Weight [kN/m^2]
Precast units (spanning 6 m, designed for a 5 kN/m^2 imposed load)	3.5 to 4.5
Composite slab (Normal weight concrete, 140 mm thick)	2.8 to 3.5
Composite slab (Light weight concrete, 130 mm thick)	2.1 to 2.5
Services (lighting)	0.25
Ceilings	0.1
Steelwork (low rise 2 to 6 storeys)	0.35 to 0.50
Steelwork (medium rise 7 to 12 storeys)	0.40 to 0.70

The imposed loads on the floors are given in Clause 6.3.1.2(1) of EN 1991-1-1 [64] and the NAs of the countries for office and residential areas and are summarised in Table 3.5. A value of 3.0 kN/m^2 is recommended for office areas and 2.0 kN/m^2 for residential areas, as standard values.

Table 3.5 Imposed loads for residential/offices and for mezzanine floors

Country	*Residential* (Category A)*		*Office (Category B)*	
	q_k (kN/m^2)	Q_k (kN)	q_k (kN/m^2)	Q_k (kN)

Belgium	2.00	2.00	3.00	3.00
Finland	2.00	2.00	2.50	2.00
France	1.50	2.00	2.50	4.00
Germany	1.50	2.00	2.00	2.00
Ireland	1.50	2.00	3.00	4.50
Italy	2.00	2.00	3.00	2.00
The Netherlands	1.75	3.00	3.00	3.00
Norway	2.00	2.00	3.00	2.00
Portugal	2.00	2.00	3.00	4.00
Romania	1.50	2.00	2.50	4.50
Spain	2.00	2.00	3.00	4.00
Sweden	2.00	2.00	2.50	3.00
United Kingdom	1.50	2.00	2.50	2.70
* floors only				

3.1.3 Snow loads

Snow loads are a function of local climate, terrain, roof slope, roof type, and building geometry. EN 1991-1-3 [23] specifies that snow loads have to be determined in *normal conditions* (persistent design situation), and *exceptional conditions* (persistent and accidental design situations). The snow loads on roofs, as they appear in Clause 5.2(3), are provided below:

- For persistent design situations:

$$s = \mu_i C_e C_t s_k \tag{3.1}$$

- For accidental design situations of exceptional snow load:

$$s = \mu_i C_e C_t C_{esl} s_k \tag{3.2}$$

- For accidental design situations of exceptional snow drift:

$$s = \mu_i s_k \tag{3.3}$$

where
μ_i snow load shape coefficient,
s_k characteristic value of snow load on the ground (50-year return period),
C_e exposure coefficient that varies with the topography,
C_t thermal coefficient,
C_{esl} coefficient for exceptional loads.

Recommended values for these various coefficients for a roof slope not greater than 6° are:

$$\mu_i = 0.8 \qquad C_e = 1.0 \qquad C_t = 1.0 \qquad C_{esl} = 2.0\,.$$

Variations in snow load are mainly due to s_k. EN 1991 Part 1-3 divides Europe into nine different climatic regions and defines zones to compute s_k as a function of the altitude. Four different snow classes are proposed. For each class, a recommended value of s_k is proposed to allow reuse in the same and lower snow regions (Table 3.6 and Fig. 3.1).

Table 3.6 Proposed snow classes S1 to S4 for design of roofs

Country	s_k (kN/m^2)			Snow Class
	Min. [a]	Country average [b]	Min. European value	
Finland	2.00	2.75	2.00	S1
Romania	1.50	2.00		
Norway	1.50	3.50		
Sweden	1.50	2.50		
Germany	0.45	0.85	1.00	S2
Italy	0.60	1.00		
United Kingdom	0.45	0.65	0.70	S3
France	0.45	0.65		
Ireland	0.40	0.55		
The Netherlands	0.70	0.70		
Belgium	0.50	0.70		
Portugal	0.10	0.30	0.40	S4
Spain	0.30	0.40		
[a] Assuming the average altitude for the less critical zone of the country				
[b] Assuming the average altitude for the zone representing most area of the country				

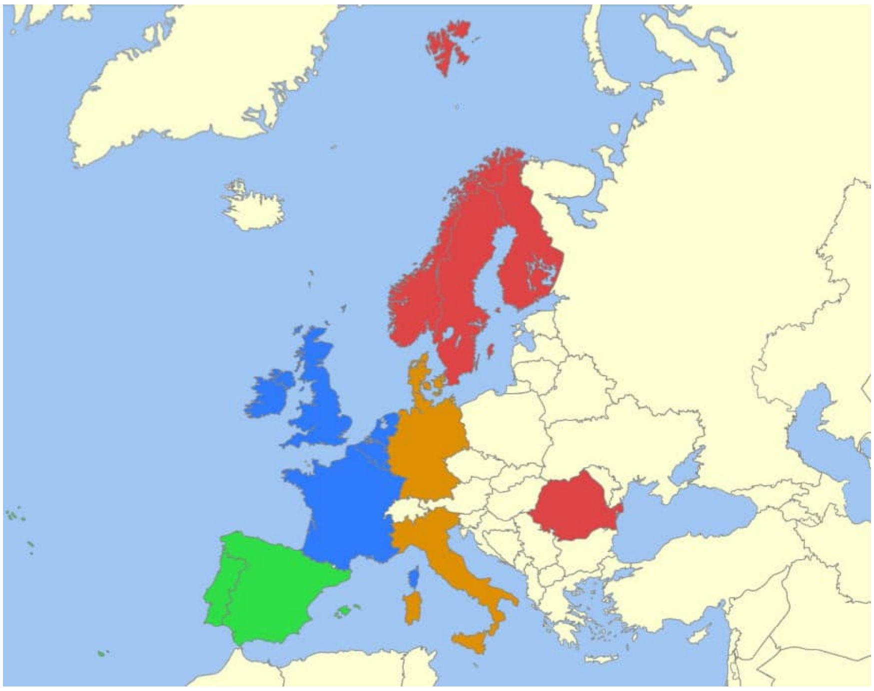


Fig. 3.1 Proposed design classes for the snow load map based on Table 3.6

3.1.4 Wind loads

EN 1991-1-4 [24] treats wind pressures as equivalent static loads. The basic wind velocity is based on a 10-minute mean wind speed for the geographical location considered. This is corrected for the effects of orography, terrain roughness, and length of exposure to the wind for which a dynamic pressure is calculated. This is then converted into a force on the surface using pressure or force coefficients, which depend on the shape of the building.

The basic wind velocity v_b is defined in Clause 4.2 of EN 1991-1-4 as a function of the wind direction factor c_{dir}, and the season factor c_{season}, which modify the basic wind velocity $v_{b,0}$, as follows (for terrain roughness category II):

$$v_b = c_{dir} c_{season} v_{b,0} \tag{3.4}$$

The value of $v_{b,0}$ is a national choice for a return period of 50 years. Table 3.7 presents the limits for this parameter and the average in various European countries [25]. Based on these values, a minimum European value for $v_{b,0}$ is proposed and, four different wind classes are defined (Fig. 3.2 and Table 3.7). Therefore, the basic velocity pressure for each European class $v_{b,class}$ can be obtained from Eqn. (4.10) in EN 1991-1-4, as follows:

$$q_b = \frac{1}{2} \rho v_b^2 \tag{3.5}$$

where ρ is the air density and the proposed value is 1.25 kg/m^3.

The velocity pressures are then used to obtain the peak velocity pressure at a specific height.

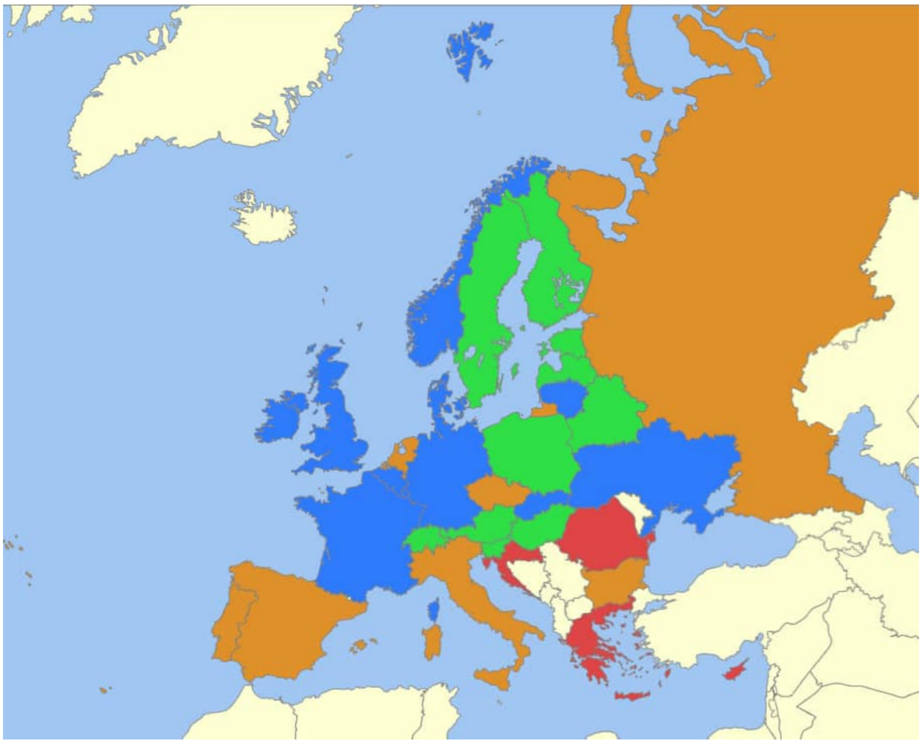


Fig. 3.2 Proposed design classes for the wind load map based on Table 3.7

Table 3.7 Basic wind velocity according to European countries: wind classes

Country	**v_{b0} (m/s)**			**$q_{b0,mean}$ (kN/m^2)**	**$v_{b0,class}$ (m/s)**	**$q_{b,class}$ (kN/m^2)**	**Wind Class**
	Min.	**Max.**	**Average**				
Croatia	20	48	29	1.05	>28	1.20	W1
Cyprus	24	40	29	1.05	>28		
Greece	27	33	29	1.05	>28		
Romania	27	35	31 a)	1.20	>28		

<table>
<tr><td>Bulgaria</td><td>24</td><td>36</td><td>27</td><td>0.91</td><td>28</td><td rowspan="7">0.98</td><td rowspan="7">W2</td></tr>
<tr><td>Czech Republic</td><td>23</td><td>36</td><td>27</td><td>0.91</td><td>28</td></tr>
<tr><td>Italy</td><td>25</td><td>31</td><td>27 [a)]</td><td>0.91</td><td>28</td></tr>
<tr><td>The Netherlands</td><td>25</td><td>30</td><td>27 [a)]</td><td>0.91</td><td>28</td></tr>
<tr><td>Portugal</td><td>27</td><td>30</td><td>27 [a)]</td><td>0.91</td><td>28</td></tr>
<tr><td>Russia</td><td>20</td><td>44</td><td>27</td><td>0.91</td><td>28</td></tr>
<tr><td>Spain</td><td>26</td><td>29</td><td>27 [a)]</td><td>0.91</td><td>28</td></tr>
<tr><td>Belgium</td><td>23</td><td>26</td><td>24</td><td>0.72</td><td>26</td><td rowspan="11">0.85</td><td rowspan="11">W3</td></tr>
<tr><td>Denmark</td><td>24</td><td>27</td><td>25</td><td>0.78</td><td>26</td></tr>
<tr><td>France</td><td>22</td><td>28</td><td>24 [a)]</td><td>0.72</td><td>26</td></tr>
<tr><td>Germany</td><td>23</td><td>30</td><td>25 [a)]</td><td>0.78</td><td>26</td></tr>
<tr><td>Ireland</td><td>25</td><td>28</td><td>26</td><td>0.85</td><td>26</td></tr>
<tr><td>Lithuania</td><td>24</td><td>32</td><td>26</td><td>0.85</td><td>26</td></tr>
<tr><td>Luxemburg</td><td>24</td><td>24</td><td>24</td><td>0.72</td><td>26</td></tr>
<tr><td>Norway</td><td>22</td><td>31</td><td>25</td><td>0.78</td><td>26</td></tr>
<tr><td>Slovakia</td><td>24</td><td>26</td><td>24</td><td>0.72</td><td>26</td></tr>
<tr><td>United Kingdom</td><td>22</td><td>32</td><td>25 [a)]</td><td>0.78</td><td>26</td></tr>
<tr><td>Ukraine</td><td>24</td><td>31</td><td>26</td><td>0.85</td><td>26</td></tr>
<tr><td>Austria</td><td>18</td><td>28</td><td>21</td><td>0.55</td><td>23</td><td rowspan="10">0.66</td><td rowspan="10">W4</td></tr>
<tr><td>Belarus</td><td>22</td><td>24</td><td>22</td><td>0.61</td><td>23</td></tr>
<tr><td>Estonia</td><td>21</td><td>21</td><td>21</td><td>0.55</td><td>23</td></tr>
<tr><td>Finland</td><td>21</td><td>26</td><td>22 [a)]</td><td>0.61</td><td>23</td></tr>
<tr><td>Hungary</td><td>24</td><td>24</td><td>23</td><td>0.66</td><td>23</td></tr>
<tr><td>Latvia</td><td>21</td><td>27</td><td>23</td><td>0.66</td><td>23</td></tr>
<tr><td>Poland</td><td>22</td><td>26</td><td>23</td><td>0.66</td><td>23</td></tr>
<tr><td>Slovenia</td><td>20</td><td>30</td><td>23</td><td>0.66</td><td>23</td></tr>
<tr><td>Sweden</td><td>21</td><td>26</td><td>22</td><td>0.61</td><td>23</td></tr>
<tr><td>Switzerland</td><td>20</td><td>24</td><td>21</td><td>0.55</td><td>23</td></tr>
<tr><td colspan="8">[a)] Usual value from the NA/local standard</td></tr>
</table>

For design purposes, it is necessary to calculate the peak velocity pressure $q_p(z)$ at height z, which includes mean and short-term velocity fluctuations, can be calculated according to the following expression (EN1991-1-4, Clause 4.5):

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m(z)^2 = c_e(z) \cdot q_b \qquad (3.6)$$

where:
$I_v(z)$ – turbulence intensity at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity,
ρ – is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms,
$c_e(z)$ – is the exposure factor,
q_b – is the basic velocity pressure,

$v_{\mathrm{m}}(z)$ – the mean wind velocity at a height z above the terrain which depends on the terrain roughness and orography and on the basic wind velocity, v_{b}, and should be determined using the following expression:

$$v_m(z) = c_{\mathrm{r}}(z) \cdot c_0(z) \cdot v_{\mathrm{b}}, \tag{3.7}$$

$c_{\mathrm{r}}(z)$ – is the roughness factor,
$c_0(z)$ – is the orography factor, taken as 1.0 for the cases where orography (e.g., hills or cliffs) increases wind velocities less than 5% or when the average slope of the upwind terrain is less than 3°. The upwind terrain influencing the wind may be considered up to a distance of 10 times the height of the isolated orographic feature. The orography effects can be calculated according to EN1991-1-4 Annex A.3. National annexes may impose modification to this procedure.

Orography and roughness factors are too dependent of the building location to allow for specific recommendations for reuse. However, as a rule of thumb, it is recommended to keep the ratio between basic velocity pressure peak velocity pressure at 15 m of: $q_{\mathrm{p}}(15)/q_{\mathrm{b}} \geq 1.15$ while using the wind load classes proposed in Table 3.7 and Fig. 3.2.

3.1.5 Guidance for the use of wind and snow classes

The proposed design process for new buildings will essentially require the following steps:

- Define snow and wind loads according to the building location, which will need to consider nationally defined parameters,
- Compare the snow and wind loads according to the building location with the defined values for the different design classes proposed in Table 3.6 and Table 3.7. Designers may choose to adopt the proposed country average values or the European load class values,
- Engineering judgement is required to assess the cost implications of increasing the design loads to the ones proposed in Table 3.6 and Table 3.7.

This approach is relevant in cases where future relocation and reuse of the entire building is anticipated.

Engineering judgment must consider the final outcome of the design process, and not a simple comparison between the load according to the building location and the proposed values in Table 3.6 and Table 3.7. In practical design scenarios, the available section sizes may lead to a unoptimized utilisation factor for the structural members, which can allow for an increase of the design loads without increasing the solution costs. Designers may wish to document the ULS capacity of the structure, i.e., which characteristic snow and wind loads are admissible for the structure.

Despite using the values proposed for the in the design classes, differences between design outcomes based on different national annex defined parameters are likely to occur. The use of mean national values or the proposed European load classes values together with the documentation of the structure load bearing capacity will increase the reuse opportunities.

3.1.6 Adapting characteristic load values based on return period

The characteristic values of the variable actions according to EN 1991 are calibrated to a design working lifetime of 50 years. For a lower design working life, designers may wish to reduce the characteristic values of the variable actions (only recommended for existing buildings or to a whole structure relocation scenario). For a design working life greater than 50 years, the characteristic value of the variable actions shall be corrected.

Snow Load

According to Annex D of EN1991-1-3, for return periods greater than 5 years, if the available data show that the annual maximum snow load can be assumed to follow a Gumbel probability distribution, then the relationship between the characteristic value of the snow load on the ground and the snow load on the ground for a mean recurrence interval of *n* years is given by the formula:

$$s_{\mathrm{n}} = s_{\mathrm{k}} \left\{ \frac{1 - V_{\mathrm{x}} \frac{\sqrt{6}}{\pi} ln\left(-ln(1 - P_n)\right) + 0.57722}{(1 + 2.5923\, V_{\mathrm{x}})} \right\} \tag{3.8}$$

where:

s_k – Is the characteristic snow load on the ground (with a return period of 50 years, in accordance with EN 1990),

s_n – Is the ground snow load with a return period of *n* years,

P_n – Is the annual probability of exceedance (equivalent to approximately 1/*n*, where n is the corresponding recurrence interval (years),

V_x – Is the coefficient of variation of annual maximum snow load, that can be defined by the relevant national authority. Values between 0.2 and 0.6 are suggested as informative values according to the Annex D of EN1991-1-3. For reducing the building lifetime, values between V_x= 0.20 and 0.30 are recommended.

Values for s_n / s_k for different coefficient of variation are given in Table 3.8.

Table 3.8 Adjustment of snow load according to the return period (EN1991-1-3)

Return period	s_n / s_k					
	***V_x*=0.20**	***V_x* =0.30**	***V_x* =0.40**	***V_x* =0.50**	***V_x* =0.60**	***V_x* =0.70**
15	0.87	0.84	0.81	0.79	0.78	0.76
30	0.95	0.93	0.92	0.91	0.91	0.90
50	1.00	1.00	1.00	1.00	1.00	1.00
75	1.04	1.05	1.06	1.07	1.07	1.08
100	1.07	1.09	1.11	1.12	1.13	1.14
125	1.09	1.12	1.14	1.16	1.17	1.18

The coefficient of variation for the snow load may vary between 0.30 and 1.15. Values between 0.30 and 0.70 are suggested as the lower boundary and mean value, respectively [26]. The value may be defined for specific countries by national annexes or other valid references.

Wind Load

The mean wind velocity having the probability *p* for an annual exceedance is determined by multiplying the basic wind velocity v_b by the probability factor, c_{prob} , given by the following expression (Note 4 in Clause 4.2 of EN 1991-1-4):

$$c_{\text{prob}} = \left(\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K\, ln(-\ln(0.98))} \right)^{n} \tag{3.9}$$

where:
K – is the shape parameter depending on the coefficient of variation of the extreme-value distribution,
n – is the exponent.

According to the core Eurocode, the recommended values are 0.2 for *K* and 0.5 for *n*. The probability *p* can be obtained based on the return period, i.e., for a return period of 50 years, p = 1/50=0.02, leading to $c_{prob} \approx 1.00$. The values of c_{prob} for different return periods can be found in Table 3.9.

Table 3.9 Adjustment of wind load according to the return period (EN1991-1-4)

c_{prob}								
Standard	***K***	***n***	**Return Period**					
			15	**30**	**50**	**75**	**100**	**125**
Core Eurocode	0.20	0.50	0.93	0.97	1.00	1.02	1.04	1.05
Germany NA	0.10	1.00	0.91	0.96	1.00	1.03	1.05	1.07
France NA - p>0.02	0.15	0.50	0.94	0.98	1.00	1.02	1.03	1.04
France NA - p≤0.02	0.20	0.50	0.93	0.97	1.00	1.02	1.04	1.05
Neverlands NA - Zone I	0.20	0.50	0.93	0.97	1.00	1.02	1.04	1.05
Neverlands NA - Zone II	0.234	0.50	0.92	0.97	1.00	1.02	1.04	1.05
Neverlands NA - Zone III	0.281	0.50	0.91	0.96	1.00	1.03	1.05	1.06
United Kingdom NA	0.20	0.50	0.93	0.97	1.00	1.02	1.04	1.05
Portugal NA	0.11	1.00	0.91	0.96	1.00	1.03	1.05	1.07

Imposed loads

As shown in the previous sections, the adaptation for snow and wind is discussed in the relevant parts of EN 1991 by means of a probability factor (c_{prob}). For imposed actions on floors, the characteristic values of the variable action may be adapted according to the procedure proposed by the Dutch National Annex to EN 1990, clause A1.1(2) [27] as follows (see Table 3.10):

$$q_{\text{t}} = q_{\text{t0}} \left(1 + \frac{1 - \psi_0}{9} \ln \frac{t}{t_0} \right) \tag{3.10}$$

where:
q_{t} – is the adapted characteristic value of the variable action for the design working life,
q_{t0} – is the characteristic value of the variable action for a design working life of 50 years,
Ψ_0 – is the factor for combination value of a variable action (EN 1990),
t – is the target design working life,

t_0 – is the standard design working life of 50 years.

Table 3.10 Adjustment of imposed load on floors according to the return period

q_t/q_{t0}						
ψ_0	**Return Period**					
	15	**30**	**50**	**75**	**100**	**125**
0.50	0.93	0.97	1.00	1.02	1.04	1.05
0.60	0.95	0.98	1.00	1.02	1.03	1.04
0.70	0.96	0.98	1.00	1.01	1.02	1.03
1.00	1.00	1.00	1.00	1.00	1.00	1.00

3.1.7 Thermal action

Changes in the temperature of a steel structure cause thermal strains in the steel elements. The magnitude of the thermal strain is equal to the thermal expansion coefficient which is stated in EN 1993-1-1 as $\alpha = 12\times10^{-6}$/°C for temperatures lower than or equal to 100°C multiplied by the temperature increase. This corresponds to 1.2 mm expansion per degree of temperature rise per 100 m of building. The result of the thermal strain can be a free expansion of the element if there is no constraint, or if the expansion is fully restrained, an axial stress is induced.

In portal frames, with vertical bracing in the side walls at the ends of the building, axial forces will appear in structural elements that are continuous between the braced bays, from thermal expansion. The axial force depends on the temperature change since construction of the building and the stiffness of the restraints.

In practice, axial stresses may be relaxed by slippage at bolted connections or elastic buckling of secondary elements to relieve axial load. Continuous longitudinal elements such as crane runway beams, crane rails, valley girders and eaves beams should be considered carefully and designed for axial loads due to temperature change. The guidelines provided by EN 1991-1-5 [28] should be followed to define the thermal action. Very stiff elements such as crane runway beams may potentially develop large forces due to thermal expansion.

As a rule of thumb, it is suggested that if expansion joints are provided at 150 m centres for typical portal frames without cranes, longitudinal members need not be designed to resist stresses due to restraint of expansion. Positioning vertical braced bays midway between expansion joints will allow unrestrained expansion away from the braced bay.

In the transverse direction, changes in temperature will result in the members of the changes in length of the portal frame. Even for a 4-bay portal frame, elastic analysis shows that the effects of thermal action in the in-plane direction are small enough to be neglected for internal steelwork.

Steel frames in multi-storey buildings also expand and contract with changes in temperature. Often, the temperature change of the steelwork itself is much lower than any change in the external temperature, because the steelwork is protected. It is recommended that expansion joints are avoided, if possible, as they are expensive and can be difficult to detail correctly to maintain a weathertight external envelope. Instead of using expansion joints, the frame may be analysed including the effects of the temperature change. The temperature actions may be determined from EN 1991-1-5, and combinations of actions verified in accordance with EN 1990. In most cases, the members will be found to be adequate.

In absence of calculations, common practice for multi-storey buildings in Northern Europe is that expansion joints are needed when the length of the building exceeds 100 m for simple (braced) frames and 50 m in continuous construction. In warmer climates, common practice is to limit the length to around 80 m. These recommendations only apply to the steel frame – expansion joints must be provided in stiff external cladding such as brickwork. When expansion joints are provided, they are commonly located at significant changes in shape in the building plan, or at changes in number of floors, or to separate building parts on different foundations.

3.2 Combinations of actions

Combinations of actions for a given limit state are presented in Clause 8.3.4 of EN 1990 for different design situations: persistent, i.e. final use of the structure, transient, accidental, and seismic design cases. This section focuses on the first use and therefore only the fundamental load combinations are considered. For the strength (STR) limit state, the combinations of actions can be obtained from Eqn. 8.12, or Eqns. 8.13 and 8.14 in EN 1990.

When a structure is designed for a working life of 100 years, the target reliability must be increased to reflect the longer period of exposure. EN 1990 indicates reliability indices in the order of $\beta \approx 5.2$ for a 1-year reference period and $\beta \approx 4.3$ for 50 years. Although these β-values are often associated with Consequence Class 3 (CC3), a CC2 structure with an extended design life can achieve the required reliability by adopting the reliability differentiation measures described below.

Option A: Amplify Partial Factors ($K_{FI} = 1.1$)

Table A.1.9 of EN 1990 allows the use of a global reliability differentiation factor K_{FI}. Applying $K_{FI} = 1.1$ to the partial factors for unfavourable actions in the fundamental combinations raises reliability to the required level. When K_{FI} is used, load combinations should be assessed with Eqn. (8.12) from Clause 8.3.4.2(2) of EN 1990; Eqns. (8.13) and (8.14) would give lower reliability. Normal supervision and inspection (EXC2) may be sufficient only if the risk assessment confirms that the reliability target has been met. Otherwise, higher supervision (EXC3) should accompany the use of $K_{FI} = 1.1$.

The fundamental load combinations are then obtained from the following general expression, in which "+" implies "to be combined with":

$$\underbrace{\sum_{j\geq 1} 1.5G_{k,j}}_{\text{Unfavourable permanent actions}} \text{"+"} \underbrace{\sum_{h\geq 1} G_{k,h}}_{\text{Favourable permanent actions}} \text{"+"} 1.65Q_{k,1} \text{"+"} \sum_{i>1} 1.65\psi_{0,i}Q_{k,i} \qquad (3.11)$$

Option B: Increase Execution Class and Supervision

An alternative is to specify a higher execution class (e.g., EXC3 or EXC4) together with enhanced inspection and maintenance. This approach improves the realised reliability without altering the partial action factors. However, Annex A.5 of EN 1993-1-1 stipulates that a higher execution class must not be used to justify reducing resistance partial factors.

Designing with $K_{FI} = 1.1$ (or with EXC3) provides additional capacity that may facilitate future relocation or reuse of the structure in regions where design parameters differ. The initial conservatism thus preserves compliance with the standard CC2 50-year reliability requirements even after significant changes to national annex parameters.

Either amplifying partial factors by $K_{FI} = 1.1$ or upgrading execution class can achieve the reliability targets for a CC2 structure with a 100-year design working life. The designer should document the chosen strategy, verify the reliability index using EN 1990 Equation (8.12), and ensure that supervision and inspection regimes are commensurate with the adopted reliability differentiation measures.

4 Reuse through design and better construction details

4.1 Structural design for single-storey steel buildings

Three single-storey steel building forms are identified for the study of the opportunities to facilitate reuse through design. They comprise rafters (simple or continuous hot-rolled beams, or lattice structures) and columns. The beams, columns, joints and column bases are the structural elements of the building frame. In terms of reuse of the structural members or entire frames, general principles apply which are explored below.

The building frame is first designed globally as an assembly of members considering the characteristics of the joints. To facilitate reuse, the characteristics of the joints should be clearly defined, and the general recommendation would be to use either notionally pinned or rigid connections in the first use, if the same members are used in their entirety in a second application.

From the structural analysis, the internal forces and moments are obtained for each load combination and are used to verify the resistance and stability of the members taking into account the secondary elements. For service loads, the displacements are calculated to be able to assess the acceptability of the structural system in terms of its effect on the cladding, cranes and the general use of the building.

The basic structural systems are presented in simple terms in Fig. 4.1.

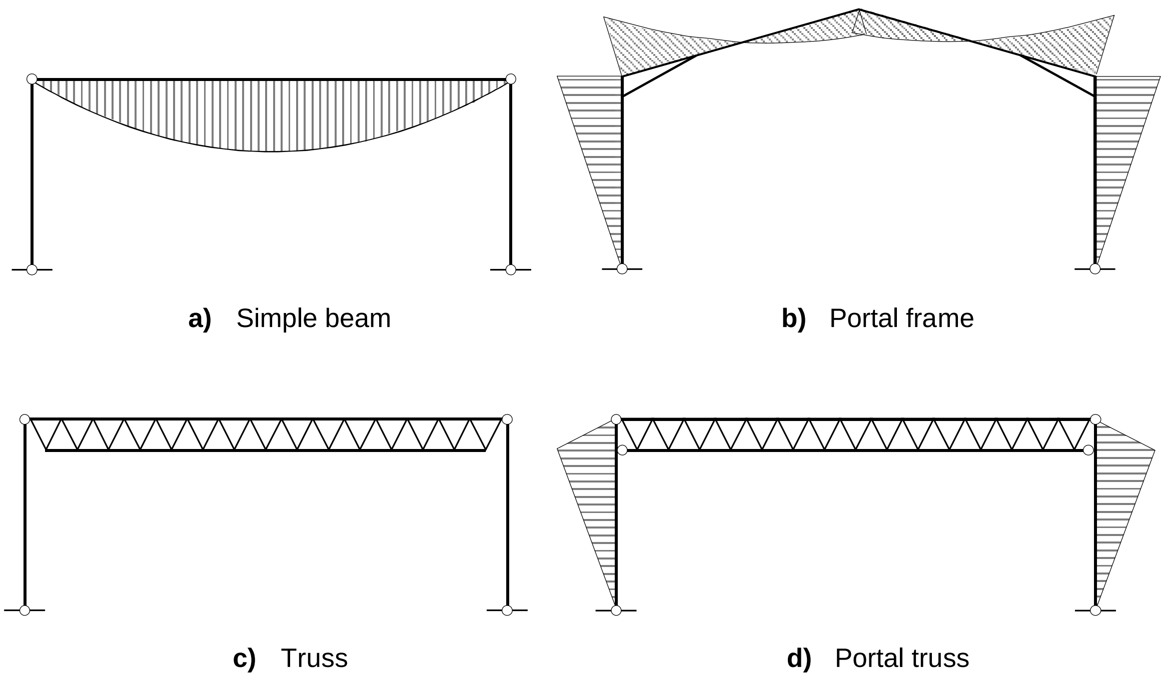


Fig. 4.1 Structural concepts for primary steelwork and bending moment diagrams from global elastic analysis (after [29])

4.2 Standardisation in single-storey steel structures

The ability to achieve a sensible level of standardisation of the primary structure depends on:

- The form of construction, as portal frames have inclined rafters and trusses have well defined lattice forms,
- The dimensional requirements for the structure and the spacing of frames,
- Column heights depend on the application of the space and the need for a mezzanine floor, etc.,
- Loading applied to the structure, which is likely to be similar within one region and building type,
- The minimum use of different structural and secondary components for given member lengths and loading,
- Design of connections using bolts, and standardisation of the components in these connections,
- Design of end gables (end frames) to be of the same form as the internal frames,
- Design and detail columns to act as edge or internal columns in a possible multi-bay portal frame scenario,
- Eaves connections must try to avoid the use of hunched segments; the solution with simple end plates, with possible lines of bolts over and below the rafter, is recommended; for longer spans, a hunched solution must be required for strength and/or connections stiffness. The designer must keep in mind that little influence on overall member design and overall frame stability is achieved by introducing a hunched apex.

4.2.1 Opportunities for reuse in portal frame structures

A conventional portal frame system offers the possibility of reuse of its individual components, as most primary members are long, with a span-to-depth ratio of 40 to 50 for columns and 50 to 65 for rafters (identified in green in Fig. 4.2). The lengths in green may be separated from the more critical zones (identified in red in this figure) by cutting to obtaining beam and column segments.

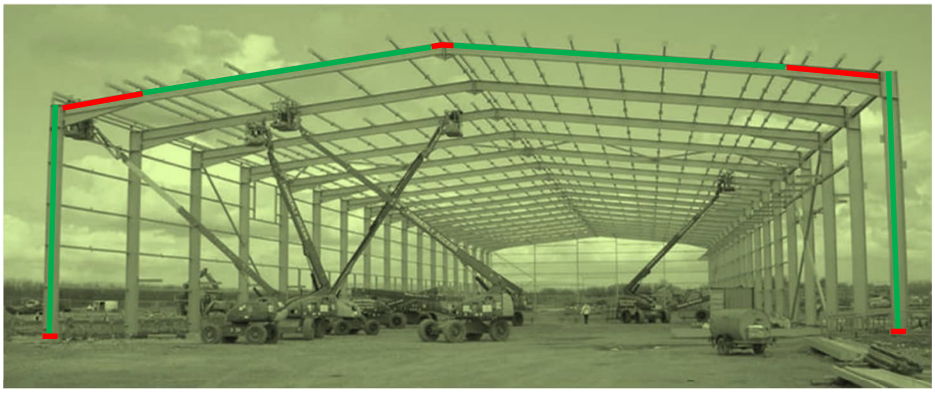


Fig. 4.2 Opportunities for reuse in portal frame: segments with minor modifications

It is recommended to design portal frames for the following standardised dimensions:

- Span increments of 3 m. Typical spans are 30 m, 36 m and 42 m using rolled sections,
- Roof slope of 6^{o} to the horizontal,
- Frame spacing of 6 m or 7.5 m with 7.5 m being preferred for purlin and side rail systems,

- Columns with a height to the top of the column of 7.5 m as standard (6 m may be used for portal frame spans less than 30 m and 7.5 m for longer spans). The height to the underside of the haunch may be up to 1 m less than the column height,
- Design the columns for the additional load from a mezzanine floor on a 7.5 m square grid with the floor level at 4 m above the ground floor slab, which would require 7.5 m long columns; the square grid approach ensures that the columns can be used in a possible multi-bay future application,
- Haunch length of (10 to 12)% of the span and of a depth equal to twice that of the rafters, being 10% recommended as the standard dimension,
- End gables should be the same as the internal frames to facilitate building expansion,
- Column bases with 4 bolts that may be treated as nominally pinned at the ultimate limit state, but which may offer some rotational stiffness for sway deflection calculations,
- Bracing in the form of circular or square hollow sections with a typical range of cross section sizes (diameter/width) between 130 mm and 200 mm in diameter with lengths between 3 m up to (but excluding) 12 m length between the frames (with 7.5 m spacing); avoid using "x" bracing arrangements; it is preferable to use few robust members that can be reclaimed without modifications.

4.2.2 Standard portal frame with welded eaves and apex segments

Using the guidance presented in the previous section, a portal frame may be composed of standardised components to facilitate the reuse of the beams and columns either in a similar portal frame or in general building construction. These components are shown in Fig. 4.3 and are:

- Fabricated haunch segments (2 no.) of length $L_h \approx 0.1L$ to $0.12L$, where L is the overall span of the portal frame,
- Fabricated apex segment (1 no.) of length $L_a \approx 0.1L$,
- Beams (2 no.) of length, $L_b = 20h_b$, where h_b is the beam depth,
- Columns (2 no.) of overall length, $L_c = 20h_c$, where h_c is the column depth.

The overall span of the portal frame is given by:

$$L = 2\,(L_b + L_h)\cos\theta + L_a \qquad (4.1)$$

where θ is the slope of the rafter to the horizontal = 6^o.

The haunch depth, h_h, is approximately $2h_b$. It may be manufactured from steel plates. The end plate to the haunch is typically 15 mm or 20 mm thick. A total of 4×2 or 6×2 M20 to M24 bolts act in tension at the top of the connection and 2×2 M20 to M24 bolts act in shear at the base of the connection.

Based on the shape of the bending moment diagram close to the apex, the size of the fabricated apex segment can be reduced to a minimum to allow for connections between the rafters.

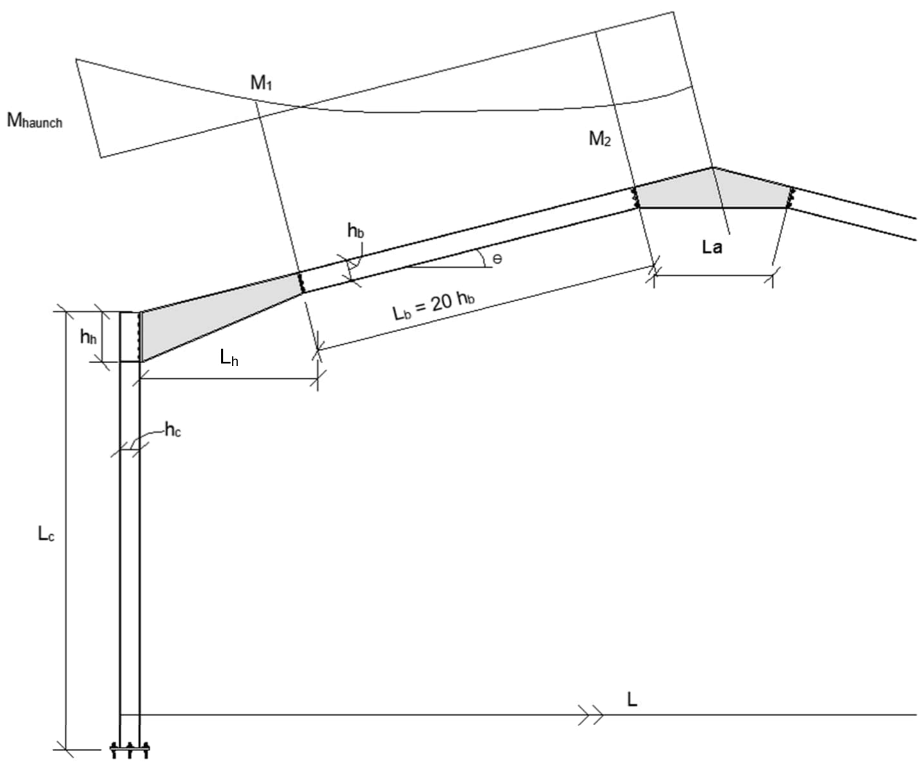


Fig. 4.3 Reusable components in a portal frame [5]

4.2.3 Portal frame rafter with bolted haunch

Other opportunities to increase the component reuse of a portal frame are shown in Fig. 4.4, which shows a bolted haunch detail. In this case, friction grip bolts are used between the separate haunch section and the inclined rafter. A similar example of this haunch segment is shown in Fig. 4.5 with a fabricated profile with a tapered section. The same system may be used for the apex connection in Fig. 4.6.

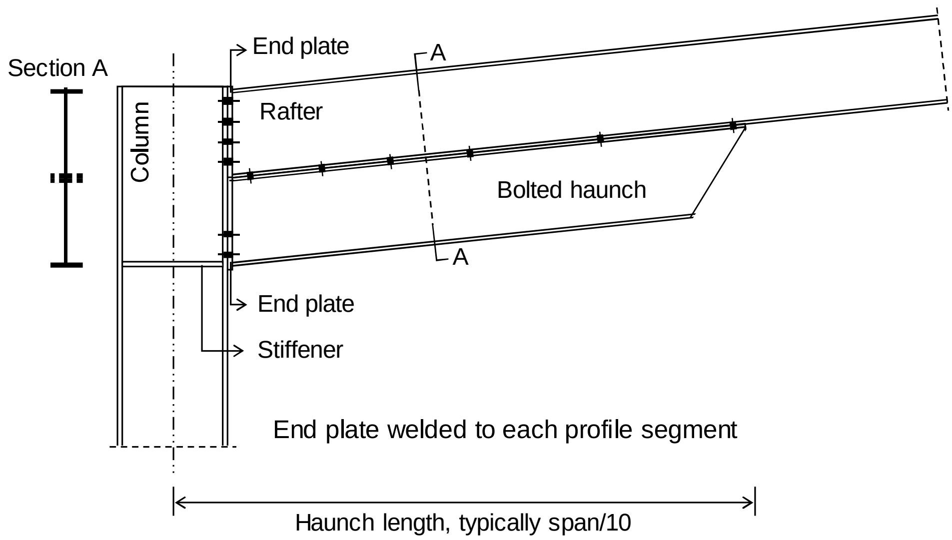


Fig. 4.4 Bolted haunch components in a portal frame [5]

Both the bolted haunch and the rafter have individually welded end-plates, allowing the connection behaviour to that of common eaves portal frame connection. Fig. 4.5 and Fig. 4.6 presents alternatives for eaves and apex connections.

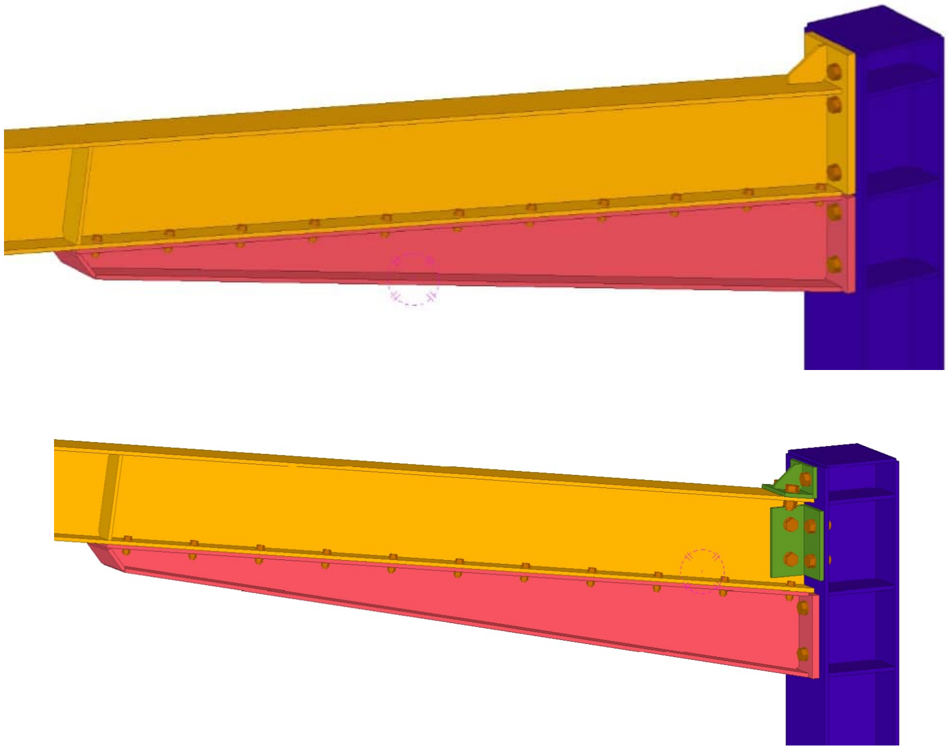


Fig. 4.5 Bolted tapered haunch segment in a portal frame

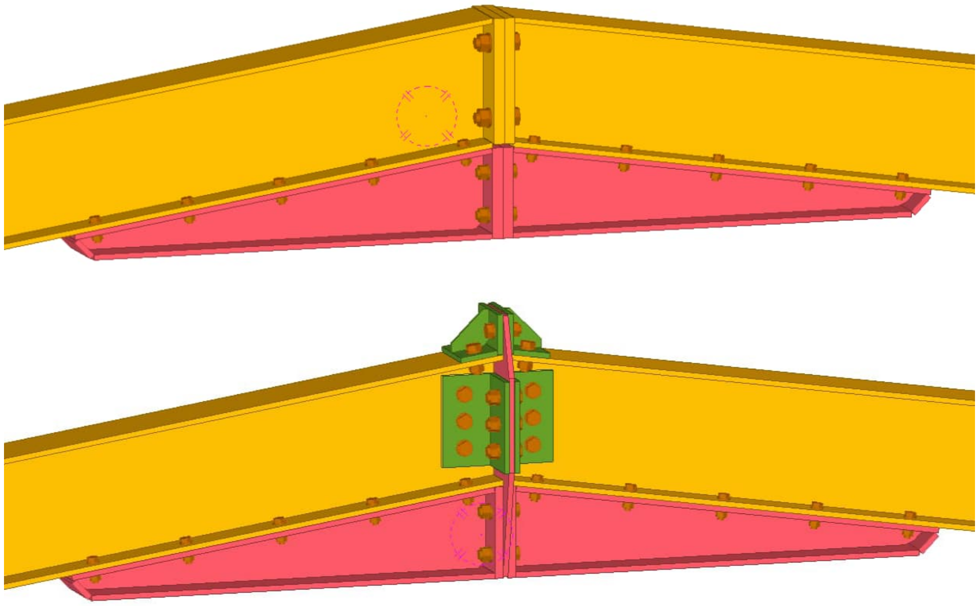


Fig. 4.6 Bolted apex components in a portal frame

4.2.4 Portal frame with corner strut

For short and medium span portal frames, the haunch detail can be replaced by a strut using a Square Hollow Section (SHS) with 4 bolts connected to the rafter and column flange (see Fig. 4.7). These bolts act in shear and tension depending on the applied bending moment. In the example, the inclined strut is at an angle of 45+3 = 48^o to the horizontal, so that the ends of the strut have the detail at the same angle to the member axis. The strut will generally be a SHS member, located approximately 1…1.5 m below the axis of the member connection, so it may interfere with the use of the space. The web of the column and rafter would have to be stiffened locally by a half-web stiffener. To reuse the full length of the rafter, a bolted end plate compatible with use in general applications should be used.

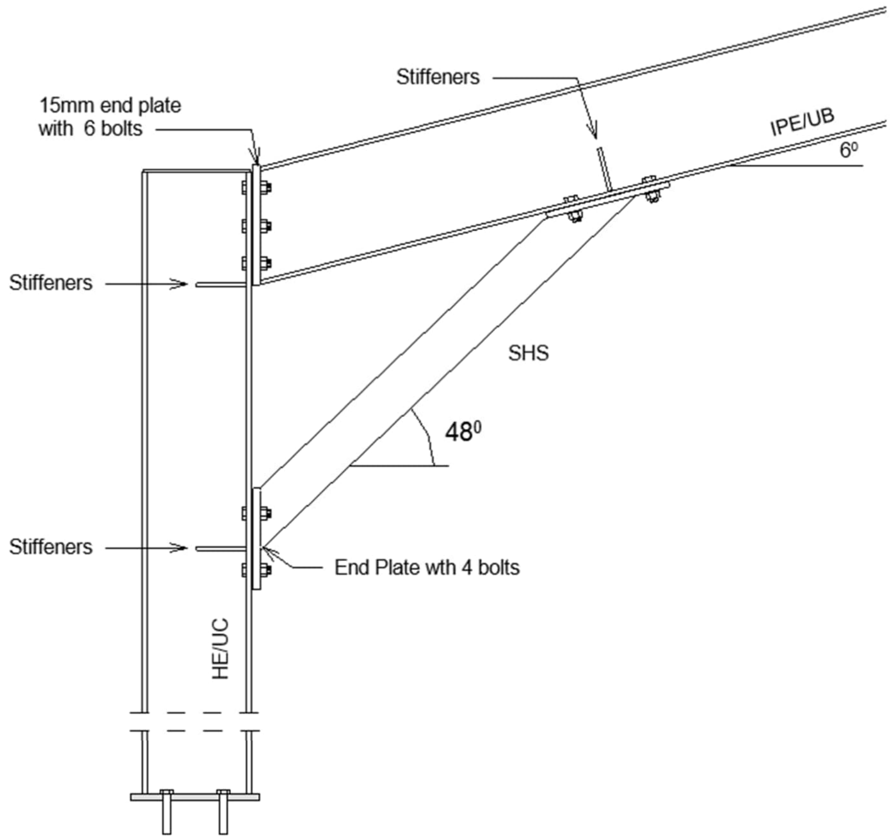


Fig. 4.7 Strut-type haunch in a portal frame [5]

The same strut detail may be used at the apex, but in this case the strut is horizontal. It is less efficient at resisting bending than the haunch due to the shallow inclination of the rafter.

4.3 Column connection to foundations

There are three generic forms of connections to the foundations that may be considered depending on the scale of the structure, i.e.:

- Pinned connections using bolted angles for relatively simple and short span portal frames. An example is shown in Fig. 4.8a. To facilitate grouting operations, the proposed solution can be bolted to a base plate which can provide a constant soffit,

- End plate connections that have some bending stiffness, but are normally treated as pinned, unless the end plate extends outside the column. The use of additional welded stiffeners is not recommended to facilitate the reuse of the columns,
- A column 'shoe', which is bolted to the column by friction bolts, as shown in Fig. 4.8b can transfer a high moment to the foundation. The column shoe is prefabricated for a particular column size. It may also be combined with the use of a bolted connection for inclined bracing, as also shown.

a) Pinned b) Fixed

Fig. 4.8 Examples of demountable base connections to columns

It is also recognised that the column bases should be accessible to enable them to be demounted easily without damage and without the need for major demolition works. An example of a detail that achieves this accessibility is shown in Fig. 4.9.

Fig. 4.9 Example of accessible base connection [30]

4.4 Reusable truss solutions

In principle, the re-usability of welded steel trusses is good because the trusses are connected using bolted connections to the columns. Truss elements are strong on the major axis of bending and are easy to dismantle. Welded trusses are designed and manufactured to specific span and loading requirements and it is difficult to make modifications to the truss span.

A typical truss configuration is shown in Fig. 4.10. It is usually manufactured in two pieces that are connected with sleeve joints in the ridge of the truss and in the middle of the bottom chord. These connections act in tension or compression, depending on the load direction. The connection to the column can be either pinned or fixed (e.g., the truss in Fig. 4.10 acts as fixed to the column, while the one in Fig. 4.11 is pinned).

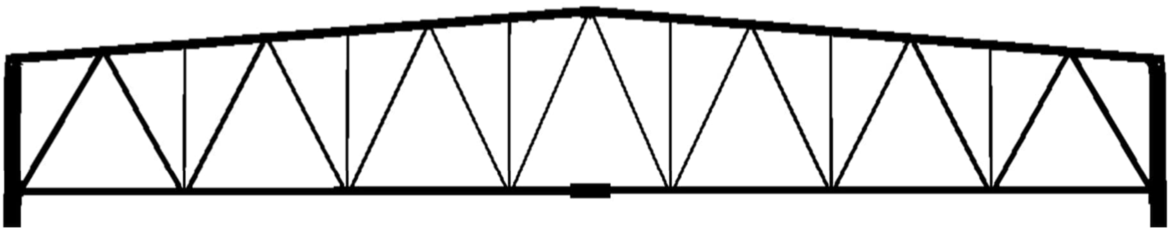


Fig. 4.10 Typical truss configuration with roof slope of 1:20 (~3°) [31]

The principles that guide the reuse of truss systems are the following.

- Trusses should be considered for spans exceeding 30 m and are preferred where additional loads are applied to the truss including suspended services or walkways,
- Trusses should have a span: depth ratio of about 20 and so a truss may be 2m deep at its apex for 40 m span. Therefore, a span: depth ratio of 20 may be adopted as a standard for reuse and this may be expected to be very stiff in bending,
- Due to their higher fabrication costs, the trusses should ideally be placed at 7.5 m or even 9 m apart depending on the form of the secondary elements,
- A truss should be fabricated from Structural Hollow Sections (SHS) in which the chords and bracing members are of the same width so that high axial forces can be transferred without bending of the face of the SHS,
- For trusses of more than 50 m span, it may not be possible to deliver the trusses in two equal segments and so intermediate splices may be required. These splices should be at the quarter span positions,
- The width of the truss chords is normally chosen so that they are stable in compression under their own weight when being lifted at two points at an assumed angle of 45°. For a 20 m long truss segment with lifting points at 12 m apart, the slenderness in the transverse direction should ideally not exceed 200, in which case the minimum width would be 150 mm (i.e. 150×150 SHS or 150×100 RHS),
- The top chord is relatively stable in compression when restrained by the roof purlins, but the bottom chord would generally have to be stabilised at mid-span in the case of wind uplift on the roof. This can be achieved by an inclined strut to a beam at ridge level,
- Trusses can be designed to transfer significant bending moments to the columns by ‘push-pull’ action and so structures using trusses are efficient in terms of their resistance to horizontal forces, provided that that the bottom chord is stabilised by in-plane bracing in the transverse direction.

One option to improve reusability of welded steel trusses is to change the typical truss configuration so that the truss would be always delivered in two pieces having uniform height with parallel top and bottom chords as shown in Fig. 4.11. The system should enable installation of roof truss into different slopes and thus enabling small range of variation in truss spans. The truss halves are connected in the middle with project specific connector parts to accommodate different roof slopes in the range of 3º to 10º. The connectors are designed with friction grip bolts acting in shear and they are shown in Fig. 4.12.

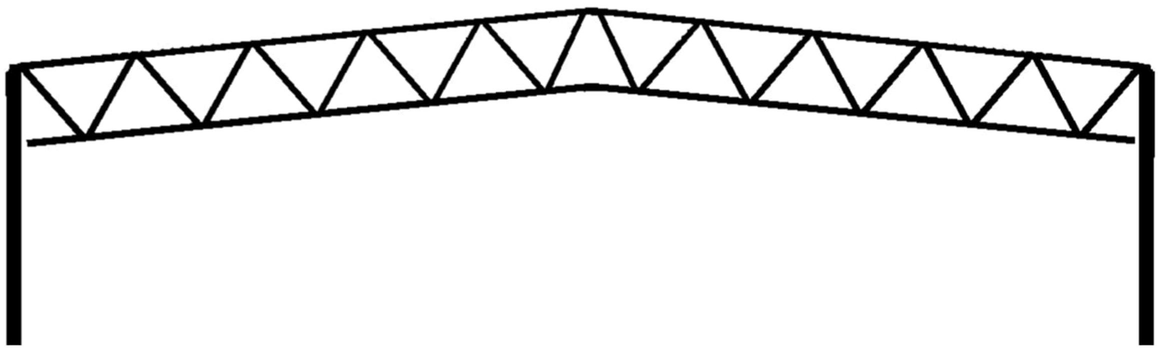


Fig. 4.11 Steel truss system for better reusability [31]

In order to maintain the portal frame action of these trusses with the columns, a connector piece would have to be used between the ends of the bottom chord and the flange of the column.

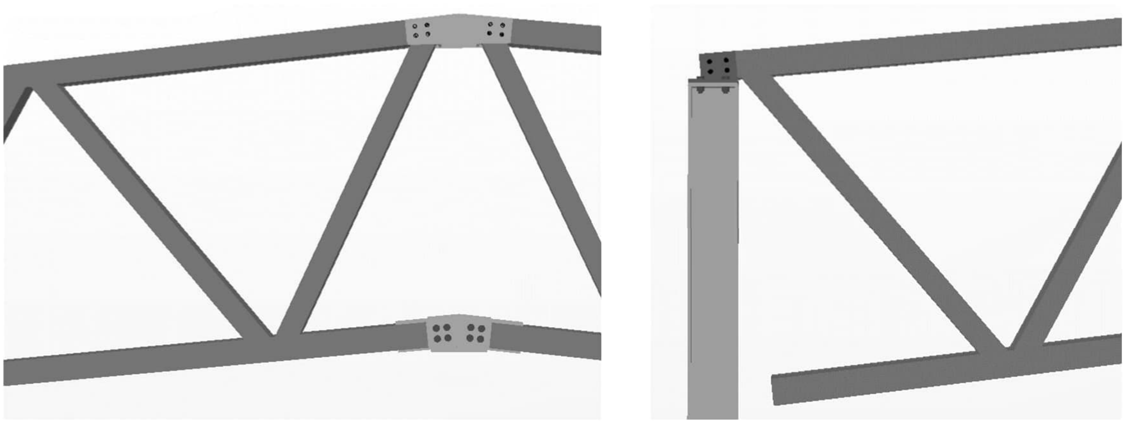


Fig. 4.12 Connectors enabling truss assembly to different slopes [31]

4.5 Braced box type structures

Braced box-type structures offer good opportunities for reuse as the structural components can be easily standardised. All connections between members can be pinned, which requires simple detailing and a minimum number of bolts. The solution requires a bracing system in the longitudinal direction of the building and a roof truss bracing supported by the braced gable frames, which provide in-plane stability of the frames (see Fig. 4.13). This concept is widely used in temporary structures and may be considered for single-storey buildings with short spans.

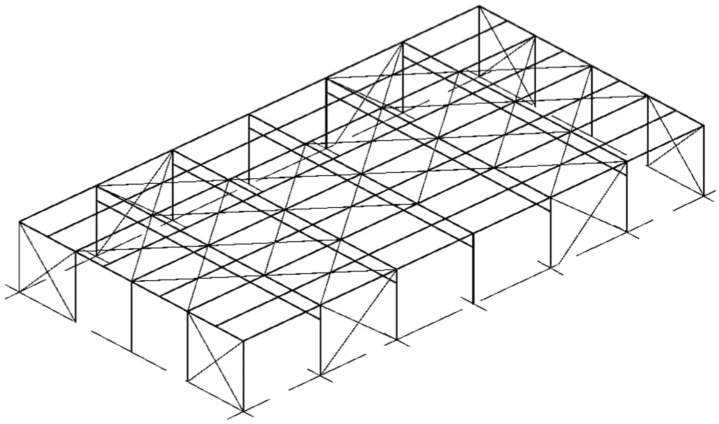
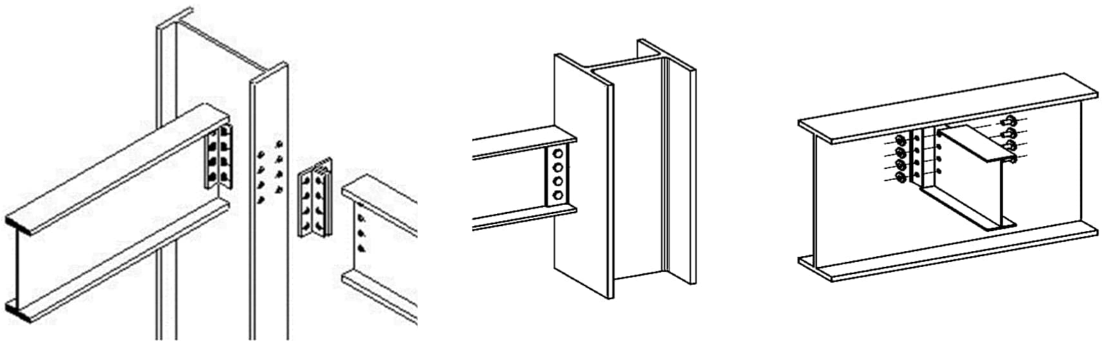


Fig. 4.13 Examples of braced box type structures [5]

4.6 Mezzanine floors

It is recommended that mezzanine floors rely on a column grid with multiples of 1.5 m, and typically 6 m. It is proposed to use a value of 4 m as a standard height above the ground floor for a portal frame with 7.5 m eaves height. Mezzanines should use a grid of pin-ended beams with cleated connection (see Fig. 4.14a). Members with equal cross section should have the same detailing. While connecting beams to the minor axis of a column, cleated connections may not be possible. For such cases, a fin plate detail can be used (see Fig. 4.14b). The general recommendation is that welding on floor beams shall not be used. If hollow section columns are used, Hollo-Bolt or Blind-Bolt solutions are recommended (see Fig. 4.15).



a) Cleated connection with angles

b) Fin plate connections

Fig. 4.14 Recommended connections for mezzanine floor beams

a) Hollo-Bolt

b) Blind-bolt

Fig. 4.15 Hollo-Bolt and Blind Bolt solutions [32]

An alternative to the cleated and fin plate connections is the system presented in Fig. 4.16, which can speed up the erection and deconstruction processes. Guidance on this concept can be found in reference [33].

Fig. 4.16 Quicon connections [34]

One of the critical details that hinders the reusability of mezzanine floors is the permanent attachment between the floor plate and the beams, such as the traditional solution with composite floor with welded studs. To increase the reusability of floor beams, it is encouraged the use of detachable floor solutions as those presented in Fig. 4.17 to Fig. 4.20. The demountable composite floor solution proposed in Fig. 4.20 offers the benefit of demountability together with higher stiffness and resistance provided by the composite action between the steel beam and the concrete topping over the steel sheet. Guidance on the analysis and design of such systems can be found in reference [35].

Fig. 4.17 Demountable floor system using precast units and floor bracing [29]

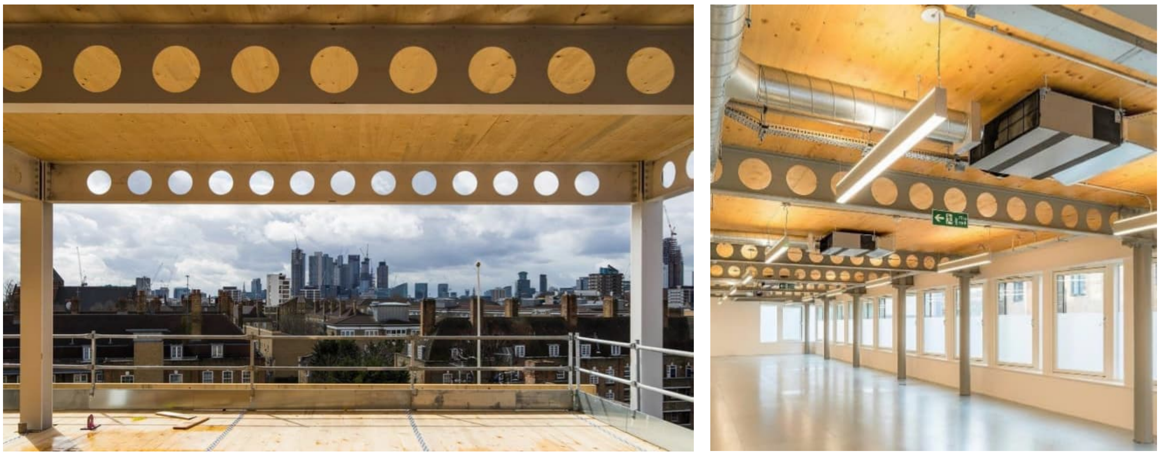


Fig. 4.18 Demountable floor system using cross laminated timber (CLT) [36]

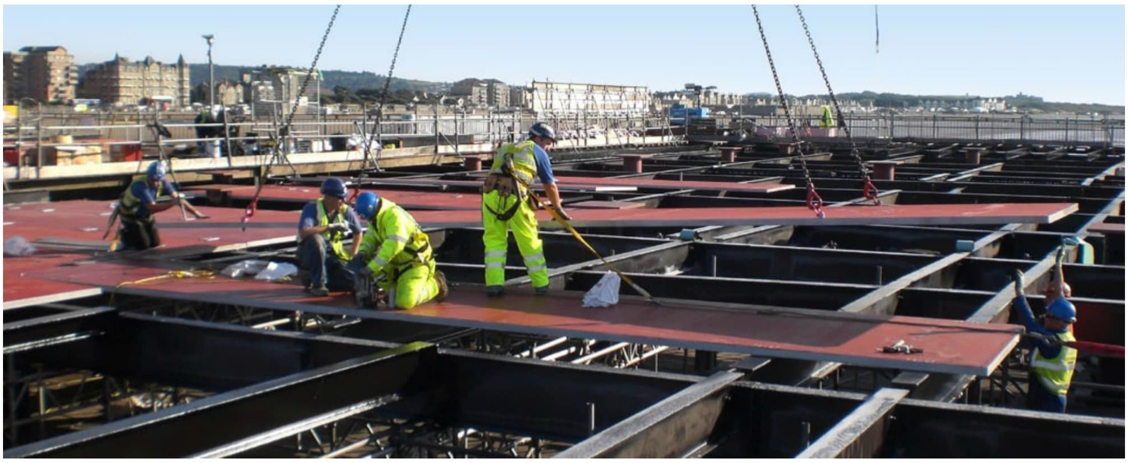


Fig. 4.19 Demountable floor system SPS panels – two steel plates with polymer core [37]

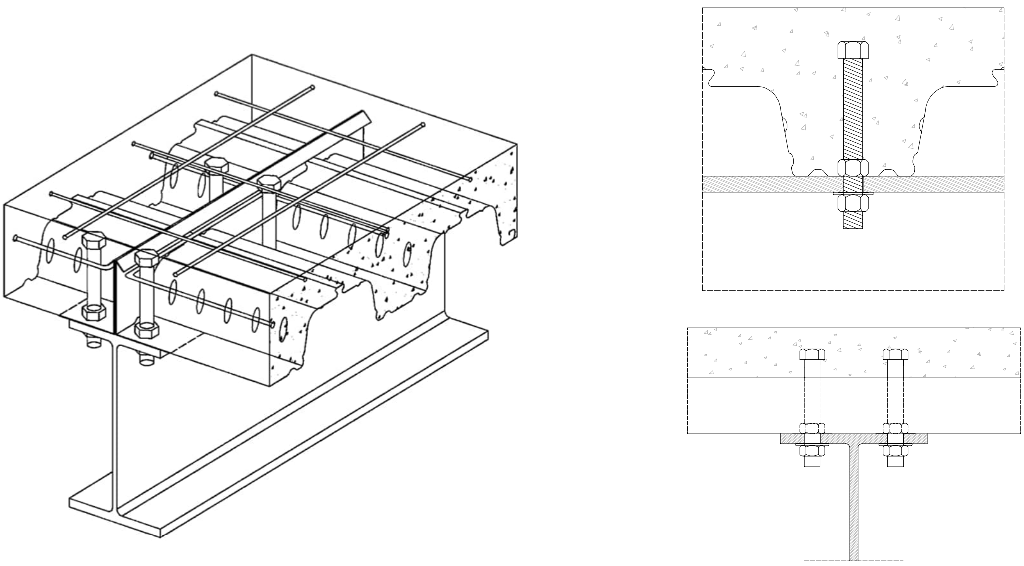


Fig. 4.20 Demountable composite floor system with a cast-in edge trim to form a cut line to allow reuse of slab segments [35]

4.7 Secondary steelwork and cladding

Secondary steelwork and cladding are the two most critical building layers for an efficient reusable single storey building. This is related to the considerable number of connections and attachments that are used between the primary structure and secondary structure, as well as the cladding. Removing one of these two layers will facilitate the deconstruction and reuse process. Therefore, whenever possible, it is recommended to use long-span cladding systems that avoid the on-site erection of secondary steelwork. For the cases where secondary structure is used, the number of such elements should be reduced to a minimum, which will contribute to a reduction of the connections between those layers.

4.7.1 Secondary steelwork

Secondary elements are typically in the form of Z-section purlins for roofs and C-section side rails for walls. Z-sections are often designed with sleeves or overlaps over the supports to benefit from continuity, the purlins bolted being via angle cleats to the top flange of the rafters. To facilitate reuse of the rafter beams, these cleats should be bolted using 2 bolts at a minimum spacing of 1.8 m along the rafter (therefore, a minimum purlin spacing of 1.8 m is recommended). For a span of 7.5 m, sleeved purlins are often 250 mm deep.

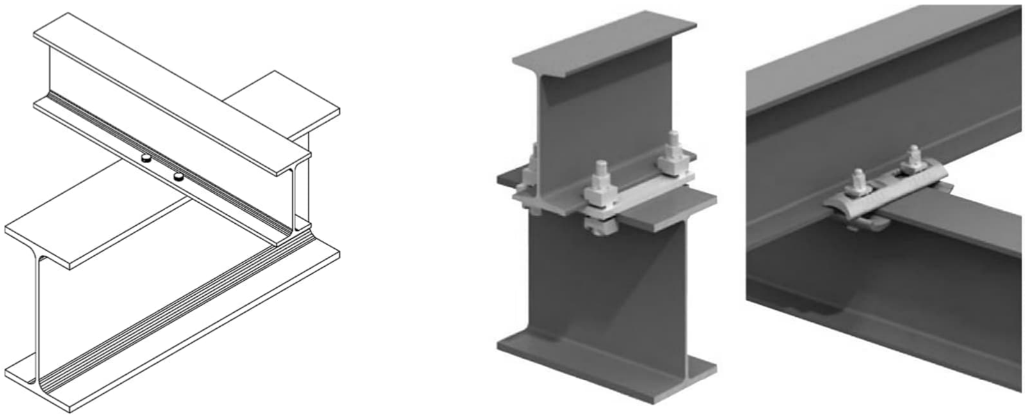
In many countries, cassette systems are used as an alternative to C section side rails, as they support the external cladding and insulation in the cassette tray. Cassettes are more efficient for column spacings up to 6 m.

Omega-shaped purlins should be considered for reuse applications as they do not require additional cleats as they are directly screwed to the rafters. However, they are generally not suitable for long-span applications (spans > 6 m), unless sleeves or overlaps are used.

Anti-sag bars should be avoided as much as possible, as they require significantly more on-site work. They introduce holes in the secondary structure that may affect reusability. This may lead to a heavier solution, although the assembly process is faster. Side rails usually have anti-sag bars to keep them in place while cladding elements are installed. With appropriate analysis and design, regarding in-plane and out-of-plane actions, anti-sag bars may also be avoided.

4.7.2 Alternative systems for secondary steelwork

The common solution for secondary steelwork relies on cold formed purlins and side rails that typically provide an economic solution for a spacing of 1.8 m to 2 m between members. A possible measure to reduce the number of connections between the building layers would be to rely on larger purlin/side rail spacing (e.g., 3.5 to 4 m). This will require stronger and stiffer purlins, for which a hollow section (typically rectangular) or hot rolled section may be used. The benefit of continuity for secondary steelwork can be achieved by using the well-known *Gerber* system, which used simple connection in continuous elements located at the points of expected counter-flexure (bending moment equal to zero). For an open section, the system would not require welding of auxiliary steelwork to the purlins/side rails. For open section, cleats and other fittings are not required, as the secondary steelwork can be directly bolted to the primary structure (Fig. 4.21a). As an alternative, the clamped solution may be explored, as it avoids drilling holes in the steel elements (Fig. 4.21b).



a) Bolted connection between hot rolled purlin and primary structure

b) Clamped connection between hot rolled purlin and primary structure [38]

Fig. 4.21 Connections between hot-rolled purlins and primary steelwork

4.7.3 Cladding

Cladding systems are usually fixed using self-drilling self-tapping screws, which affects the reusability of the cladding. In addition to fixings between the cladding and the secondary steelwork, additional screws are needed between the cladding panels, increasing the labour required for disassembly. These fixings are made at the top of the roof profile rib to prevent water ingress. The need for flashing elements at the building edges increases the complexity of the system, as well as the number of required fixings. The details of eaves and gable parapets, ridge detail, or simply wall junctions are examples.

The key aspect for a more reusable cladding system is related to the number and type of fixings that are used between the cladding elements and between the cladding and the secondary/primary structure. Screw type connections are recommended for all cladding operations, including flashing elements. The screw locations should ideally be hidden using standing seam-type connections between the panels.

With standardised roof slopes, it may be possible to develop standard details for eaves, parapets, apex, etc., with as few screwed connections as possible, which could reduce the on-site labour effort for assembly and disassembly.

As noted above, the use of long-span cladding is encouraged, as no purlins/side rails are needed. With the reduction of a building layer, a considerable reduction in the number of connections would be achieved, which may allow for an increase in the reusability of the cladding system.

A potential measure to make current practice in single storey buildings more efficient for reuse is to improve the fixings of the cladding systems to the secondary steelwork. Sandwich panels or roof cassettes could have pre-attached rails/trays that could allow the adjustment of clamps according to secondary structure position. This may lead to a more complex cladding installation. However, with such a system, the number of connections could be reduced to a minimum, increasing the potential reusability of the secondary steelwork and cladding.

A clear area of interest is to investigate new long-span roof and wall panelised solutions (spans of 6-10 m). If an appropriate connection system is developed (bolted or clamped with clear disassembly points), these elements could be easily disassembled and reused (see section 4.7.5).

The most common practice for long-span cladding panels (of 6 to 8 m span) is based on deep trapezoidal sheeting (Fig. 4.23) or deep sandwich panels in roofs (Fig. 4.24, Fig. 4.25) and horizontally installed sandwich panels on walls spanning between frames (Fig. 4.22). Such solutions are common practice in Finland and Sweden, where thermal insulation requirements usually require thick panels that are consequently structurally strong, enabling horizontal installation between primary structural elements without the need for purlins.

A possible solution for wall sandwich panels with clamped connections is shown in Fig. 4.22.

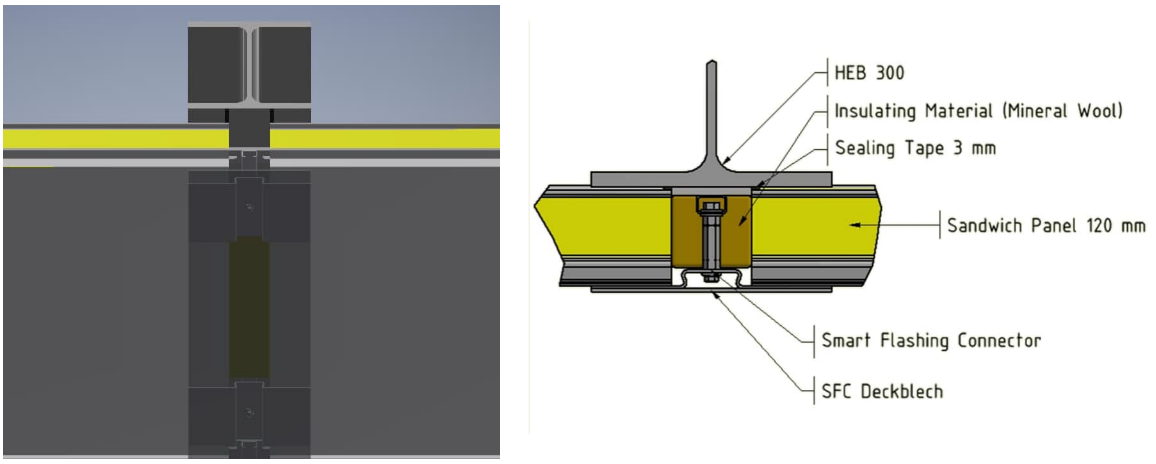


Fig. 4.22 Clamped fixing system for long span wall cladding

Fig. 4.23 Example of roof system with deep decking [31]

The use of deep decking is suitable for the proposed standard roof slopes of 3° and 6°, for hot rolled rafters or trusses. The roof is made up of a built-up solution typically comprising a trapezoidal sheeting, vapour barrier, insulation layer (mineral wool or PIR) and membrane (PVC or bitumen). The deep decking can benefit from continuous behaviour over the primary structure by means of an overlap. A typical overlap of 0.10 × Span over the supports can be assumed. As an alternative, the principles from the *Gerber* system may be used, for which an overlap of 150mm may be assumed. A minimum top flange width to support the steel sheet of 150 mm is recommended.

Fig. 4.24 Example of roof system with long span composite panels [31]

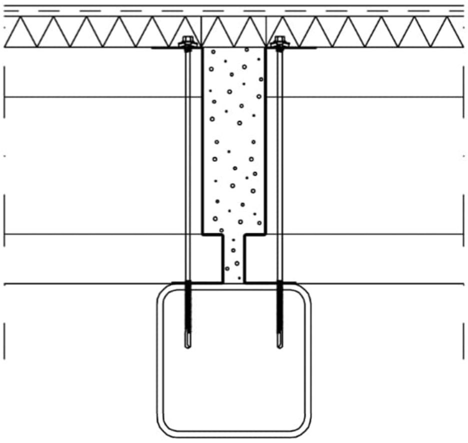


Fig. 4.25 Connections between long span composite panel and primary structure [31]

4.7.4 Use of prefabricated light steel cassettes in portal frames

Prefabricated cassettes are often used in floors and roofs in light steel framing on residential building construction. The cassettes consist of cold formed C-sections that span between Z- or U-sections at their ends. On floors, the spacing of the C-sections is 400 or 600mm and the maximum width of the cassettes is 2.4 m to be suitable for transportation and mechanical lifting.

The same form of cassette may also be used for roofs (accessible only for maintenance) and walls in portal frames for spans of 6 to 7.5 m. The cassette may be suspended from the top flange of a rafter beam by an edge Z-section, as shown in Fig. 4.26.

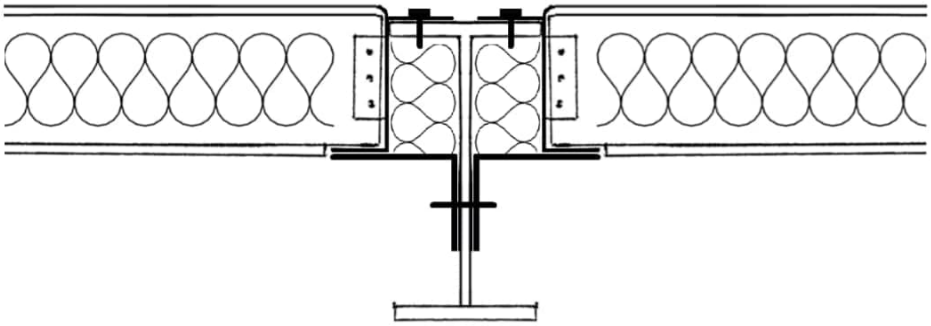


Fig. 4.26 Use of pre-fabricated insulated suspended roof or floor cassette [5]

An alternative would be to place the cassette on the top of the flange, in which case, U-sections are used at its ends (Fig. 4.27 and Fig. 4.28). A gap between cassettes may be provided to facilitate erection, which can later be filled with a thin layer of insulation. A minimum top flange width of 150 mm is recommended. Self-drilling screws with sealing washers can be used to fix the cassette to the primary steelwork. The solution is also suitable for a trussed solution with hollow sections.

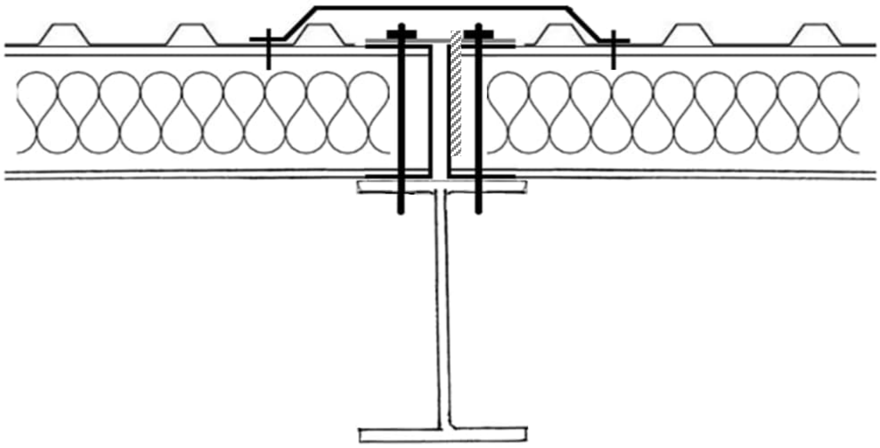


Fig. 4.27 Use of pre-fabricated insulated roof cassettes supported on the top flange [5]

For snow load not exceeding 1 kN/m^2, 200 mm deep C-sections are suitable for a 6 m span and 250 mm C-sections for a 7.5 m span. The cassette is boarded/sheeted and insulated off-site, providing weather protection during construction.

The details of a typical roof cassette are shown in Fig. 4.28 and Fig. 4.29. To allow for tolerances, the Z-section should be 75 mm wide and 2 to 3 mm thick, positioned to allow for a flat soffit levelled with the cassettes surface. The C-sections are 1.2 to 2 mm thick and are placed at 600 to 900 mm spacing depending on roof/floor loads and cassette span. The C-sections may have perforated webs to improve the thermal performance of the system (potential heat loss via cold bridging of the element by 70 to 80% - Fig. 4.29) [31].

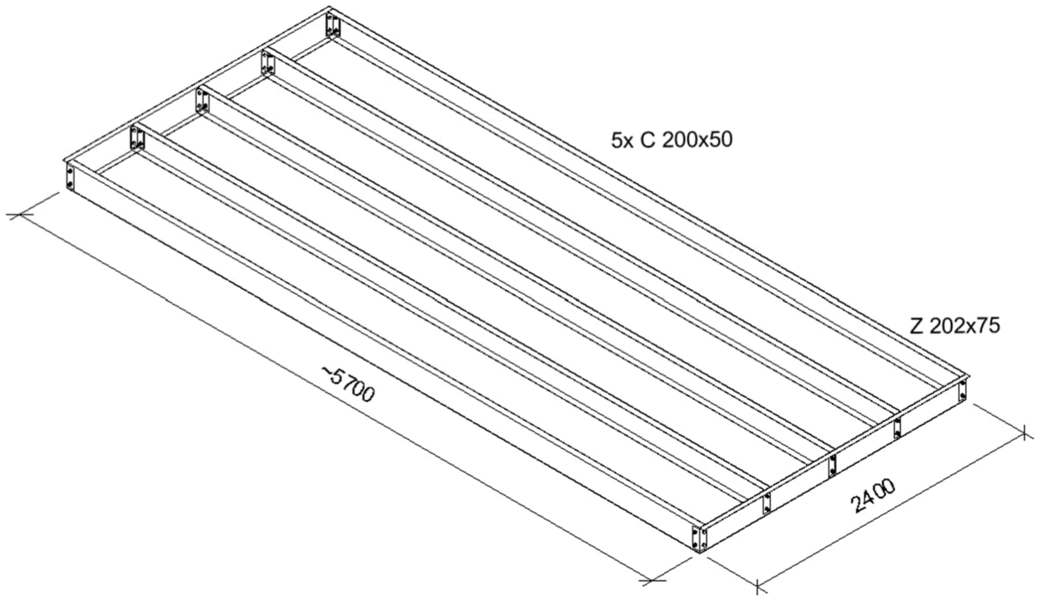


Fig. 4.28 Light steel cassette used in pre-fabricated roof construction for 6m beam spacing (less the beam width of nominally 300mm) [5]

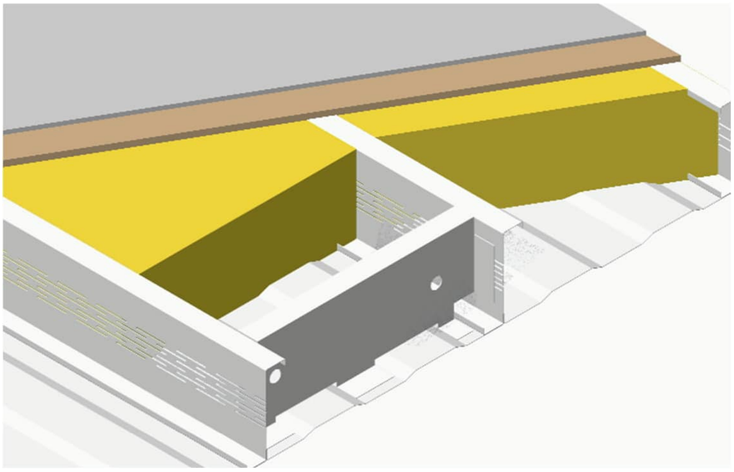


Fig. 4.29 Light steel cassette with perforated webs [31]

The C-sections have cleated connections with 3 screws to the perimeter Z/U-sections. Possible projected screw threads are cut off to avoid interfering with placement of the cassette over the beam. In some cassette systems, clinching or pneumatically pierced rivets are preferred. All connection between cassettes, cassettes and rafter of flashing elements must be done using screws.

Sheeting or OSB boards can be used to confine the insulation layer. Boards should be durable if the cassette is left exposed before the final layer of roof sheeting is attached. The board to the underside should be sufficiently robust and aesthetic so that it can be left exposed. Calcium silicate boards also provide a passive fire resisting system to the C-sections and beams.

This prefabricated system means that a weather-tight insulated building envelope is created. The cassette with its mineral wool insulation achieves a U value of 0.15 W/m^2K. The building volume may also be reduced due to the suspended floor/roof cassettes and the absence of purlins above the rafters. This will lead to cost savings for the cladding system.

The diaphragm stiffness is also provided by the boarding/sheeting applied to the floor cassettes and the screws to the beams. Tie cap plates can be used to connect the two roof cassettes placed over a rafter (as shown in Fig. 4.27), allowing for improved diaphragm action.

In the system shown in Fig. 4.26, the upper part of the beam may be insulated between the roof cassettes to reduce thermal bridging. This can be done from the inside of the building. Additional continuous angles may be bolted to the web of the beam, so that their tension-compression action provides torsional restraint to prevent lateral torsional buckling of the beams under negative bending.

The final roof covering may be provided by shallow roof sheeting. A wide range of different solutions are available on the market for this purpose. The bottom face of the cassette may be covered with sheeting or boards depending on the application of the cassette or as a client requirement. For instance, for a floor cassette to which a celling will be hanged, an ordinary OSB board will suffice; for an exposed bottom surface on a roof, a steel sheet or a board with the desired appearance may be used.

The final over cladding layer of the building can be incorporated into the cassette itself (as done on a typical sandwich panel) or installed on site over the top ‘hat’ spacer sections that were previously attached off-site to the board. The benefit of the latter is that the roof sheeting can be easily replaced in the future and the cassettes are retained without affecting the use of the building or damaging the cassette while replacing the sheeting.

Services such as lighting may be introduced as the C-sections can be manufactured with 150 mm diameter service openings. The cassettes can be reused on other roofs, where the boarding can be removed and replaced if it has deteriorated over time. The same cassette system may be used for mezzanine floors supported on steel I section beams.

The cassette systems are an alternative to long-span sandwich panels, which may not be available on certain markets for a competitive price. Cassettes allow for a more optimised solution for a certain load and span, as they are not constrained by catalogued sizes.

4.8 Structural design for multi-storey steel buildings

4.8.1 Standardization in multi-storey steel buildings

Standardisation in multi-storey steel structures is an important approach aimed at enhancing the efficiency and sustainability of building construction and its future reuse. Here are some key aspects to consider:

1. **Modular Design**: Implementing modular design principles allows components to be prefabricated and standardised, making them easier to assemble, disassemble, and reuse in different configurations or projects,
2. **Design Flexibility:** While standardisation promotes uniform components, it can be integrated with modular design principles, allowing buildings to be easily reconfigured or expanded,

3. **Material Standardisation**: The use of standardised steel sections and connections can simplify the design process, reduce fabrication costs, and ensure compatibility when reusing parts in new structures,
4. **Connection Systems**: Standardising the connection methods can improve the ease of disassembly and reassembly, which is vital for future reuse. Using bolted connections instead of welded ones can aid in this process,
5. **Documentation and Labelling**: Proper documentation and labelling of structural elements can assist in tracking components for future reuse, ensuring they can be easily identified and repurposed,
6. **Digital Technologies**: Leverage Building Information Modelling (BIM) and other digital tools for design, allowing for better visualisation and coordination of standardised components,
7. **Life-cycle Assessment**: Incorporating life-cycle assessment tools during the design phase can help assess the environmental impacts and sustainability of materials, promoting the use of reusable components.

4.8.2 Guidance for multi-storey steel buildings

The existing solutions for multi-storey steel buildings, most of which have steel-concrete composite structures, present a difficult challenge for reclaiming and reusing structural steelwork. The steel beams are connected to the concrete composite floor through shear connectors with very limited accessibility. Due to the permanent bonding of steel and concrete, composite steel floor decking cannot be reused in its original form after construction. However, both steel and concrete can be recycled, allowing for an environmentally responsible disposal method, even if direct reuse is not possible.

Thus, ordinary multi-storey steel buildings are generally reusable only in-situ due to their complex construction, composite materials, and site-specific engineering considerations. Moving them to a new site would require extensive reengineering, making it impractical for most projects.

The recommendations provided in this guide may be used for multi-storey steel buildings located in non-seismic areas or areas where DC1 (i.e., formal Ductility Class Low or DCL) concept is assumed in seismic design (low seismic action class, see prEN 1998-1-2 [39]).

Structures designed according to the dissipative structural behaviour concept shall belong to the structural ductility classes DC2 or DC3. These classes correspond to the increased ability of the structure to dissipate energy in plastic mechanisms. Depending on the ductility class, specific requirements in one or more of the following aspects shall be met: class of steel sections and rotational capacity of connections. For such cases, it is only recommended to allow for reclaimed steel elements if those elements are used at least under one of the following conditions: (i) as members of the gravity or secondary load resisting systems (not part of the lateral load resisting system, such as pin-ended floor beams), or (ii) as elements that are part of a DC1 structure.

Consequently, in the case of multi-storey steel buildings located in non-seismic areas or DC1 seismic areas, the biggest challenge is the composite action of the floor, making them impossible to reuse.

Hot rolled steel frames with precast floors are a common structural solution for a variety of multi-storey building types [10]. It could result in non-composite steel beams with precast concrete floor slabs that rest on top of the steel beams or on shelf angles connected to the web, beyond the beam flange, but do not have a structural connection to them, meaning they do not work. This contrasts with composite steel beams where a structural connection, such as shear studs, is used to create a unified load-bearing resistant element. The slim floor system can be adopted to minimise the structural floor, where precast units may be supported on a wide bottom flange or a wider plate welded to the bottom flange of a standard H section beam.

The steel beam is designed to carry the load on its own, without relying on the concrete slab for added strength or stiffness. The precast units primarily act as a floor surface and a formwork for the in-situ concrete topping (if used) and also can provide lateral support to the steel beams during construction. The precast concrete units are either in the form of hollow core units, normally 150 – 400 mm deep, or solid planks of 75 mm to 100 mm depth. Hollow core precast concrete units can be used to span up to 15 m (400 mm or deeper).

Structural design guidance for this structural solution is covered in several SCI publications, *P287: Design of composite beams using precast concrete slabs* [40], *P342: Design of asymmetric Slimfloor beams with precast concrete slabs* [41] and *P334: Design of multi-storey braced frames* [42].

Peikko [43] implemented a similar solution. DELTABEAM® is a slim-floor composite beam that is integrated into the floor. The beam is completely filled with concrete on-site. The infill concrete and DELTABEAM® form a composite structure after the concrete has hardened. DELTABEAM® acts as a steel beam before the infill concrete has reached the required strength. It can be used with several types of timber floors, including mass timber slabs (CLT, GLT, NLT, DLT) and composite timber slabs or beam decks. Precast composite units can also be used.

Non-composite steel beams used with precast concrete units offer significant advantages for reuse because their assemblies can be dismantled with minimal damage, allowing both beams and slabs to be salvaged intact. With only minor modifications, these components can be efficiently repurposed in new structures, supporting more sustainable and circular construction practices.

The RFCS REDUCE project [44] developed and tested new demountable composite floor systems, as well as providing design guidance and practical information on the fabrication and detailing of such systems, for the first and second cycles of use. The goal was to facilitate circular economy as well as the serial production of structural elements and the suitability to be added to BIM or other digital tools. Demountable beam and floor elements and adjustable steel connections form the basis of a ‘plug and play’ [45] structural system (see Fig. 4.30).

Push-out and beam tests have been carried out on new, demountable shear connection systems. The tests have been conducted on precast and composite decking using friction bolts and embedded couplers, with some systems using injected resin to reduce initial slip.

In addition, a large-scale demountable car park trial has been conducted at the Delft University of Technology comprising four (7.2m $\times$ 2.6m) prefabricated concrete decks on three 14.4m tapered, steel beams. Composite action was achieved using resin-injected embedded couplers. The tests were designed to understand the practicalities and costs of assembling, demounting and reassembling composite car park floors.

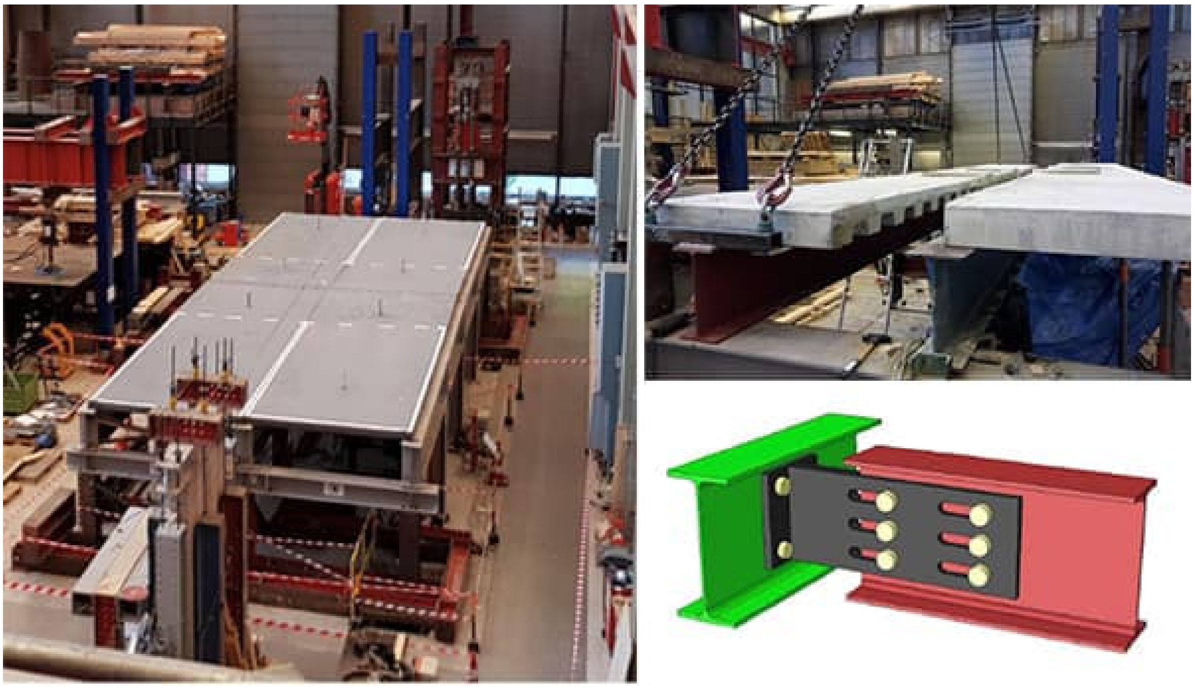


Fig. 4.30 Overview of experimental set-up [44], [46], [47]

Composite beam tests with different demountable shear connections and in different arrangements, have also been conducted. After elastic, first cycle testing, some slabs were cut longitudinally, demounted, reassembled and tested to failure. The key structural component for achieving a demountable and reusable structure is the type of shear connectors (see Fig. 4.31) used for the connection between the steel beam and the floor (for cast-in-situ and prefabricated slabs) [44], [46], [47].

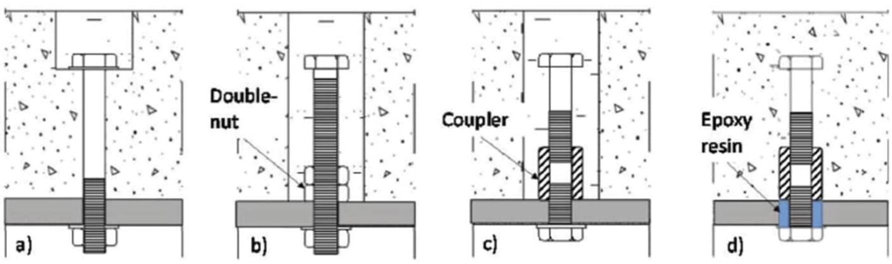


Fig. 4.31 Types of demountable bolted connectors a) friction grip bolts, b) double-nut, c) coupler system connectors, d) injected bolted shear connectors with a coupler system [44]

In addition, tests have been performed on standardised connections that facilitate the deconstruction and reuse of the beams. These included a slotted beam-to-beam connector and a block connector [45].

The design guide P428 [48] presents a design procedure and worked examples for composite beams using demountable shear connectors that are based on the principles of Eurocode 4 (EN 1994-1-1). The design methodology considers the different characteristics of the demountable shear connections, in terms of their shear resistance, stiffness, and ductility. Design data on the performance of two types of demountable shear connectors, using high-strength structural bolts and couple systems, were presented. Several solutions for buildings, hybrid or composite members and joints for multi-storey buildings were presented. However, there are no established technical standards for the structural design of demountable composite structures. Therefore, extensive research is essential to build a comprehensive knowledge database and develop robust design methodologies.

Romero and Odenbreit [49] also introduced an innovative demountable steel-timber composite (SCT) flooring system with potential for reuse. The flooring system consists of downstanding hot-rolled I-shaped steel profiles and laminated veneer lumber (LVL) slabs connected to the steel beams with novel shear connectors. This STC flooring system offers an alternative to traditional steel-concrete flooring systems. The novel shear connections implemented in the STC beams facilitate demountability, adaptability, reconfiguration, and relocation of components (see Fig. 4.32).

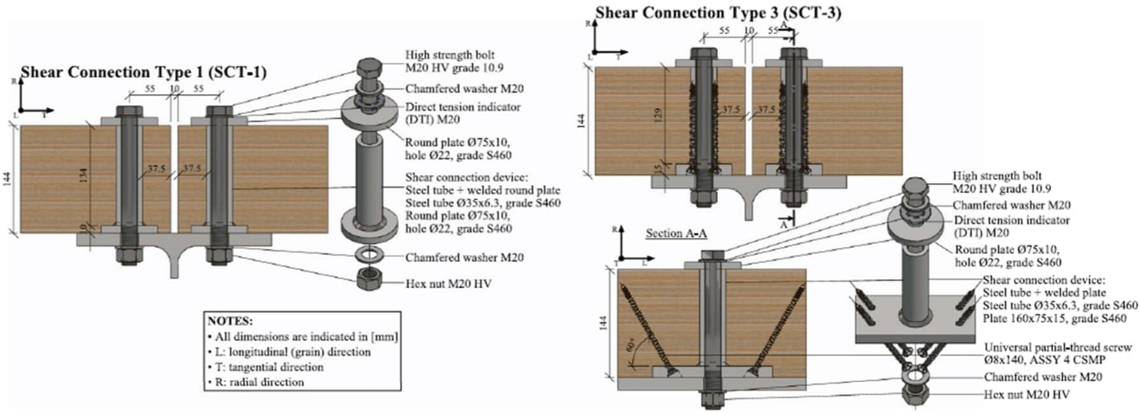


Fig. 4.32 Details of the shear connections SCT-1 and SCT-3 implemented in the demountable steel-timber composite beams tested in this experimental investigation [49]

In addition, this flooring system exhibits significant promise for modularisation, standardisation, and off-site serial production, making it ideal for prefabrication in standard sizes and modules. The structure can be relocated, or individual structural components can be used in different buildings. Hence, it brings both environmental and economic benefits.

Two full-scale STC beams, each with a span of 10 m, an LVL slab width of 2.51 m and thickness of 144 mm connected to a steel profile IPE 400, were simply supported and tested in six-point bending to assess their flexural response (see Fig. 4.33). The results of the two full-scale bending tests demonstrate their significant load-bearing and deformation capabilities, as well as their potential for reuse. Furthermore, the results indicate the efficacy of the novel shear connections and the overall structural integrity of the beams.

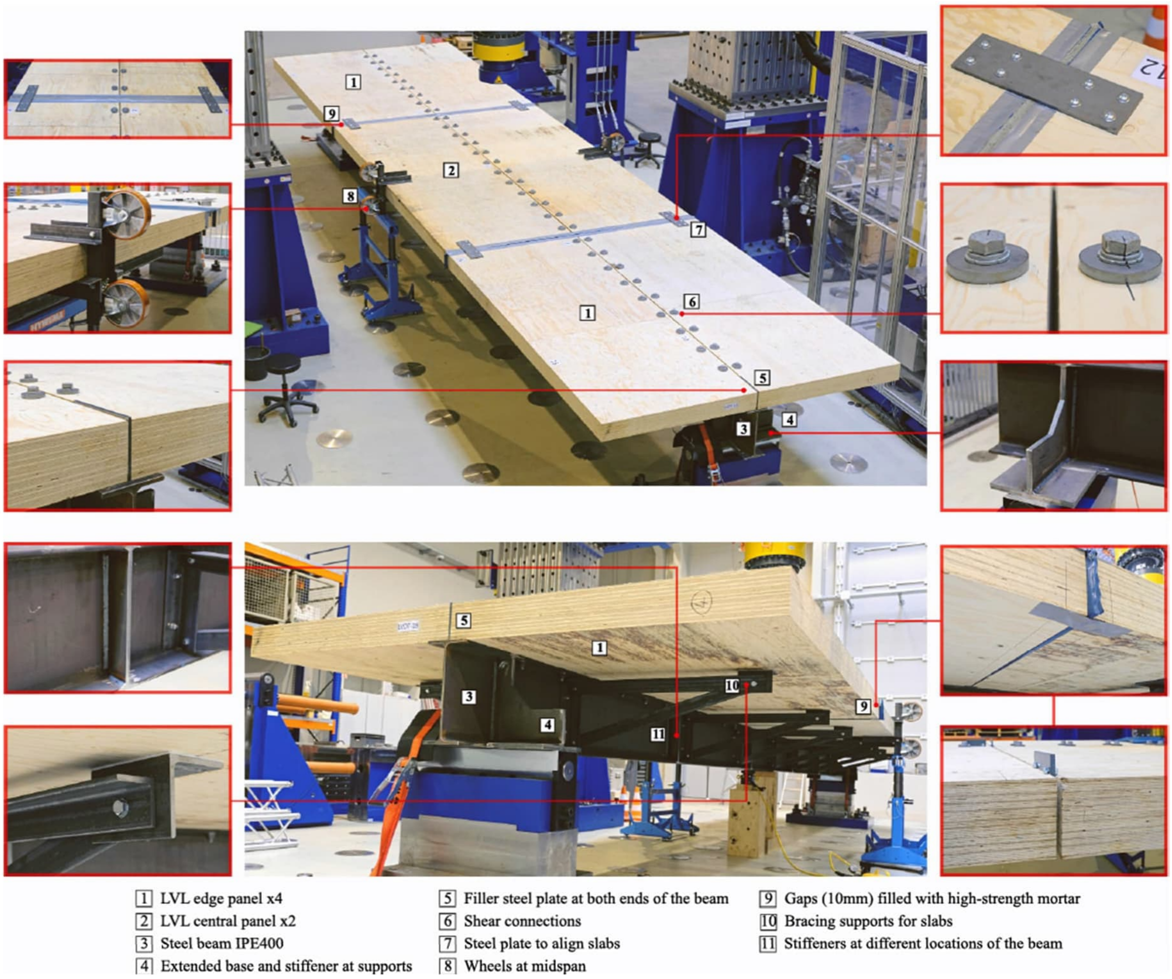


Fig. 4.33 Components of the SCT beam specimens [49]

Bompa et al. [50] evaluated the disassembly capability and the reuse potential of steel-timber shear connections. Experiments involving double shear configurations with coach screws of three diameters were used. Monotonic tests were first performed for each configuration to evaluate the stiffness, strength, and ductility (see Fig. 4.34). The counterparts were then tested under ten loading-unloading cycles, at 40% of the capacity obtained from monotonic tests, to evaluate stiffness degradation, screw deformations, and cross-laminated timber (CLT) panel damage.

After disassembly and measurements, the specimens were reassembled and tested up to failure. After disassembly, the screws had permanent deformations, and the timber panels indicated limited damage during cyclic loading. The reassembled specimens had similar stiffness, strength, ductility, and failure modes similar to those of the monotonic tested specimens. These observations suggest that both the steel section and the CLT panels have full structural reusability and that the test measurements can be adopted for a structural reuse index for building circularity indicators.

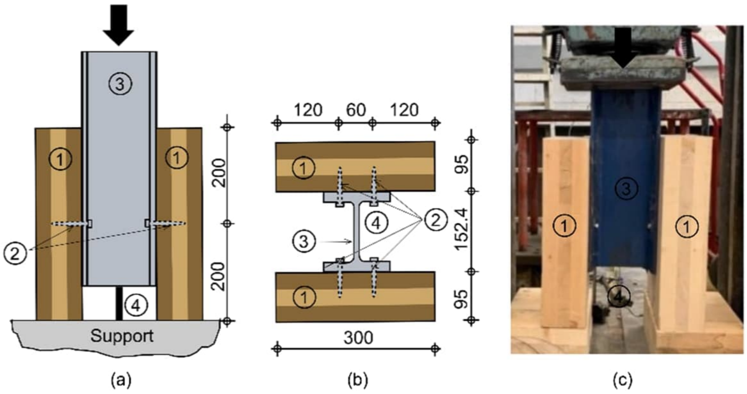


Fig. 4.34 Specimens' details: a) schematic front view, b) top view, c) view of a test specimen (legend: (1) CLT panel, (2) screws, (3) steel profile, (4) transducer, arrow indicates reaction point)

Design Guide 37, *Hybrid Steel Frames with Wood Floors* [51], encourages the use of mass timber floor systems in construction. The guide provides comprehensive context for this new building typology, detailing strategies in order to accelerate the use of hybrid timber and steel in multi-storey residential and commercial construction. Mass timber is lightweight, and steel provides strength to structures and may better meet buildings' vibration and span requirements. Design Guide 37 and the solutions presented are adapted for design for disassembly and future reuse.

The state-of-the-art findings of the ECCS Nº145 [52] publication serve as a solid future guidance on research topics to help finalise a design guideline for structures having steel timber composite/hybrid structures. The proposed recommendations are intended to outline future research topics and provide technical knowledge so that a comprehensive design code for steel-timber composite structures can be developed. The recommendations are separated into the following items: (1) timber material and products; (2) shear connections; (3) joints; (4) beams and slabs; (5) columns; (6) walls, diaphragms and braces; (7) fire design and (8) sustainability.

Ferdous et al. [53] presented a state-of-the-art review, that systematically investigated the recent advances, mechanical performances, challenges and prospects of modular buildings.

Loss et al. [54], [55] developed a multi-storey prefabricated modular building system in which the main structural components, made by combining timber with steel, are highly engineered and can be produced in the factory. The research demonstrated the potential of steel-timber hybrid structures in terms of sustainability, providing lightweight modern seismic-resistant constructions and easy disassembly. The articles present tests on several innovative connections and provides prototypes of new highly industrialised steel-timber hybrid shear wall and floor components. Fig. 4.35 shows the 3D view of the reference building, which combine steel frames and beams with cross-laminated timber panels (CLT).

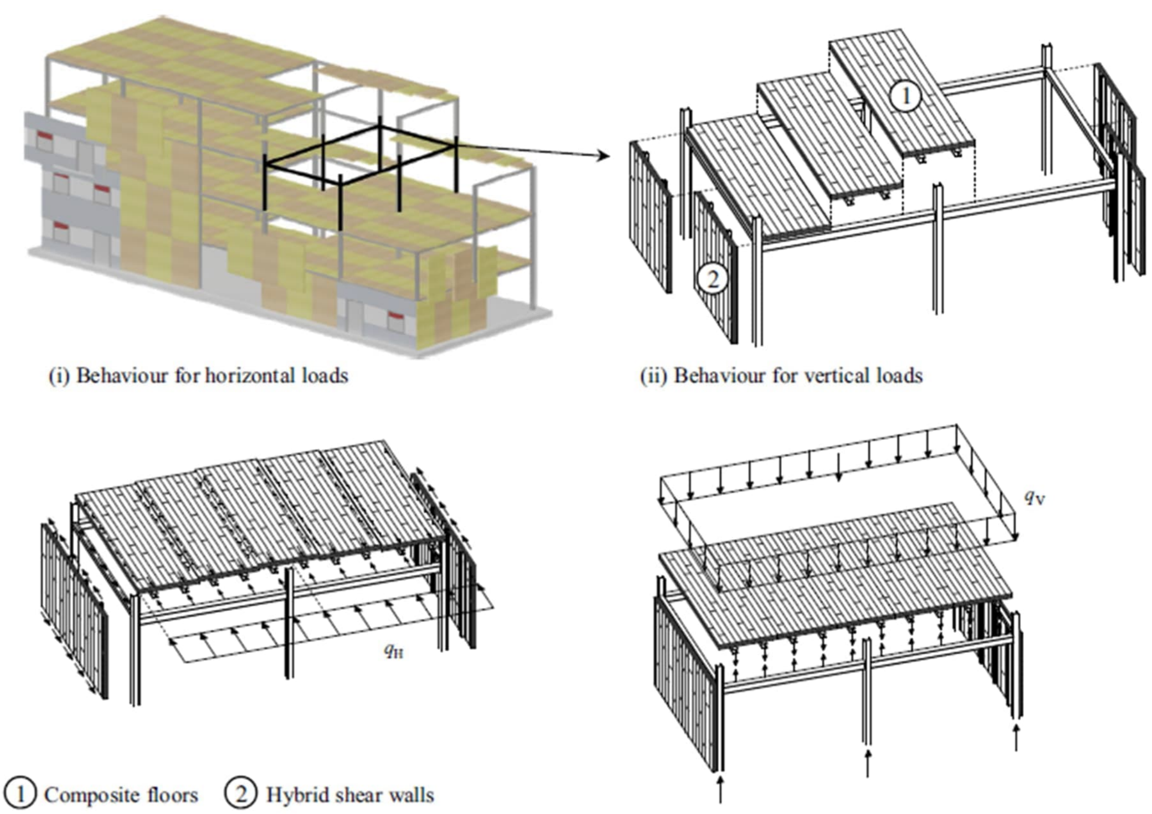


Fig. 4.35 Axonometric view of the building and exploded view of the hybrid structure with load path depicted for the vertical and horizontal loads [54]

Fig. 4.36 and Fig. 4.37 present Set I of steel–timber connections that covers 20 different configurations, while Set II includes 4 distinctive arrangements of timber–timber connections. The research has demonstrated that it is simple to obtain composite floor elements that provide excellent flexural behaviour under vertical loads.

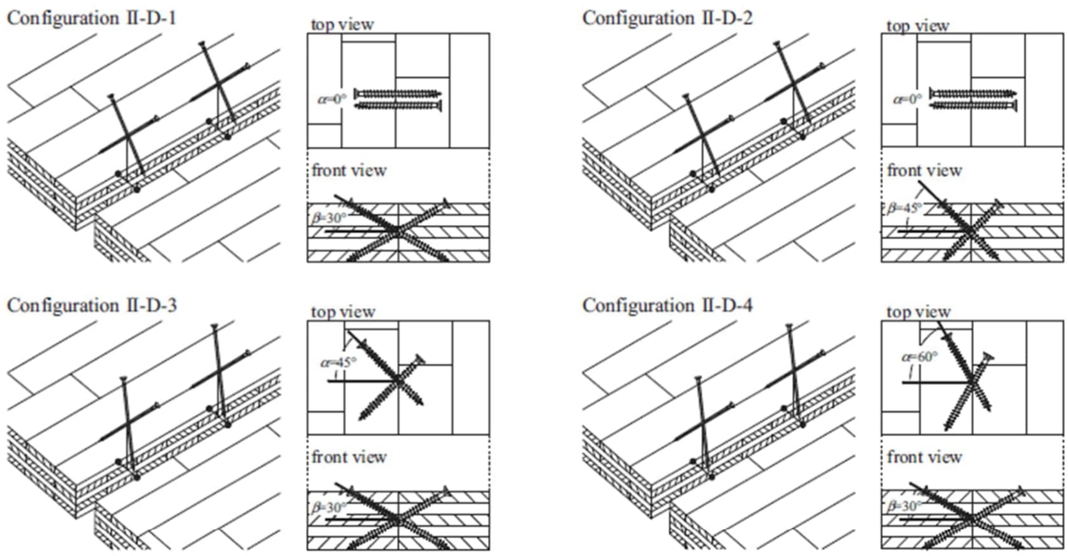


Fig. 4.36 Panel-to-panel edge connections for hybrid steel-CLT floors and shear walls [54]

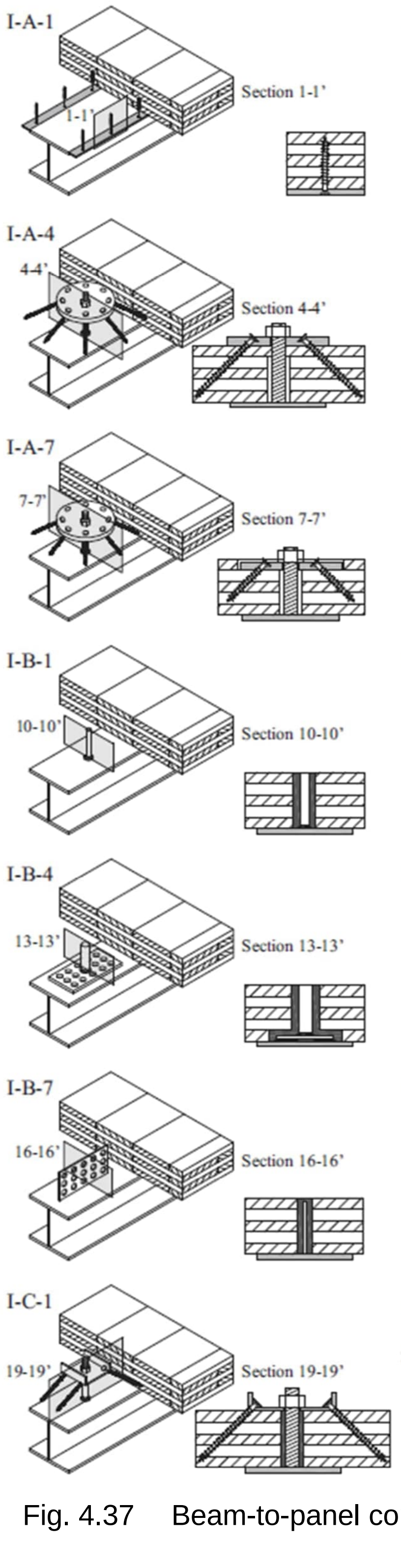


Fig. 4.37 Beam-to-panel connections for hybrid steel-CLT floors and shear wall [54]

In case of multi-storey buildings, if demountable shear connectors are used for the floor and accessibility to the beam-to-column connections is ensured, the bolted connections can be easily dismantled.

However, significant limitations are identified in the case of structures designed in accordance with dissipative structural behaviour concepts (structural ductility classes DC2 or DC3 in EN1998). The European research project “Equaljoints plus” [56], [57], [58] was aimed to provide prequalification criteria of steel joints for the next version of EN 1998-1-2 [39]. The research activity covered the standardisation of design and manufacturing procedures with reference to a set of bolted joint types and a welded dog-bone with heavy profiles designed to meet different performance levels. One of the objectives of the project was the development of a loading protocol for joint European prequalification, based on representative seismic demand in Europe.

Four bolted beam-to-column joint typologies were investigated within the project, namely (a) unstiffened extended end-plate bolted joints, (b) stiffened extended end-plate bolted joints, (c) hunched bolted joints, and (d) dog-bone welded joints (see [57], [58]) designed to meet different performance levels. The bolted joints were designed according to a design procedure in the framework of prEN 1993-1-8. The design of dog-bone welded connections was compliant with US building code ASCE 7-10 (Minimum Design Loads for Buildings and Other Structures) [59] and to the steel buildings specific standards AISC 341-16 (Seismic Provisions for Structural Steel Buildings) [60], AISC 358-16 (Prequalified Connections for Seismic Applications) [61] and AISC 360-16 [62].

The joints were proposed to be used for DC2 and DC3 ductility classes with the following performance objectives:

- Full strength joint: all the plastic demand is concentrated in the connected beam, leaving the connection and the web panel free from the damage;
- Equal strength joint: the plastic demand is balanced between the joint and the connected beam;
- Partial strength joint: all the plastic demand is concentrated in the joint.

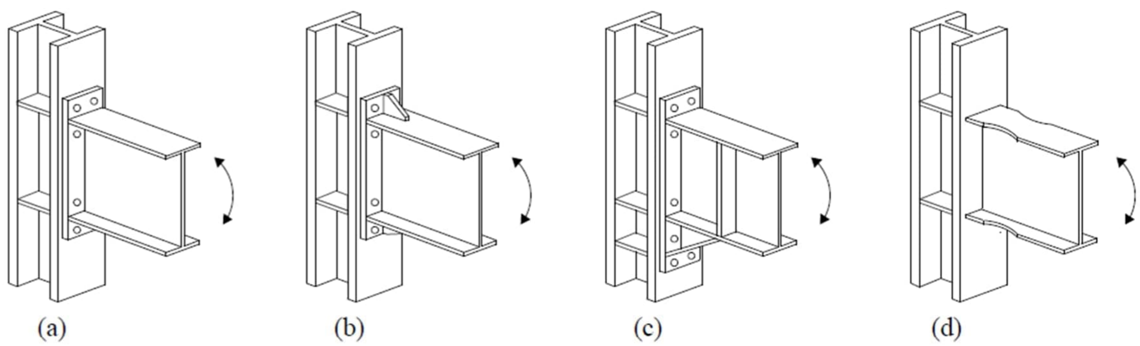


Fig. 4.38 Beam-to-column joints prequalified in the framework of EQUALJOINTS project: a) Bolted haunched joint b) Bolted extended stiffened end-plate joint c) Bolted extended unstiffened end-plate joint d) Welded dog-bone joint

Other innovative solutions for DC2 and DC3 are based on beam-to-column joints equipped with friction dampers [63]. The beam is connected to the column with a classical fixed T-stub that fastens the upper flange and a friction damper located at the lower flange of the beam (see Fig. 4.39). The friction damper is composed of a stack of steel plates conceived to assure symmetric friction. The friction damper is designed to slide at a load level lower than the nominal flexural resistance of the connected beam. In this way, it is possible to obtain connections able to dissipate the seismic input energy almost without any damage to the steel beams.

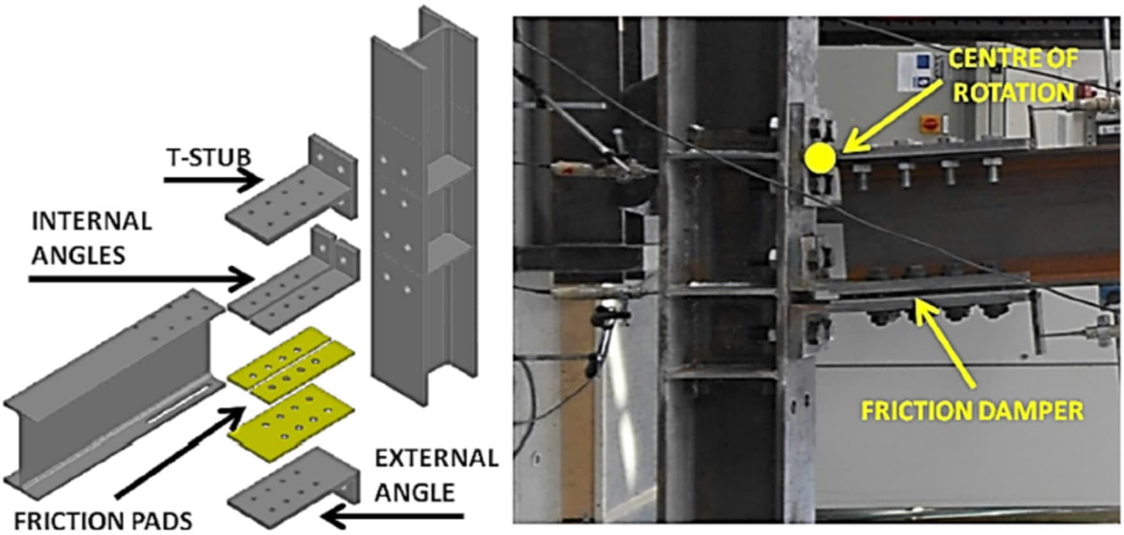


Fig. 4.39 Beam-to-column connection equipped with friction dampers [63]

In order to maximise the exploitation of the connected beam, the use of a beam end haunch to increase and calibrate the lever arm can also be suggested (see Fig. 4.40). The results of the experimental program on the friction damper have shown that maintaining the connected beams without damage is possible, even in case of repeated cyclic loading induced by earthquake loads.

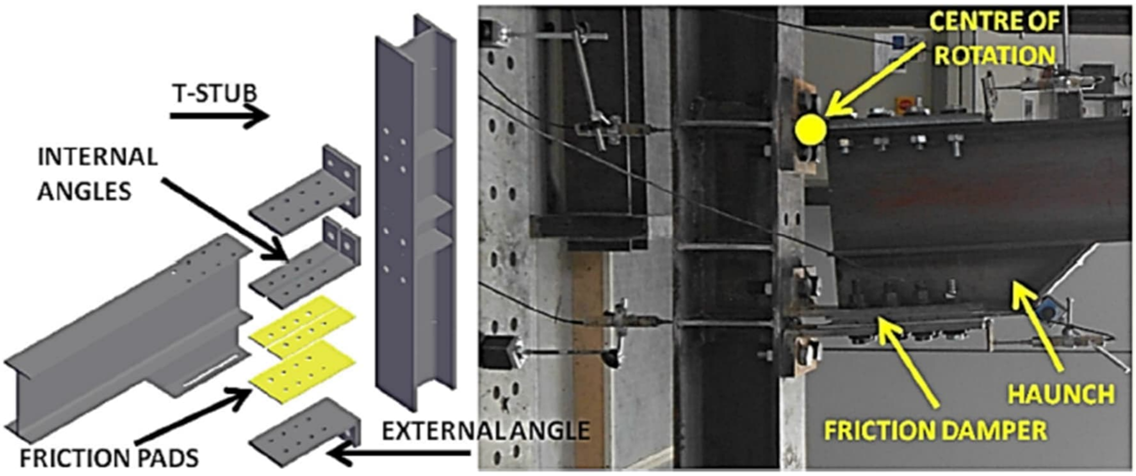


Fig. 4.40 Beam-to-column connection equipped with friction dampers and additional haunch [63]

5 Concluding Remarks

The growing need to mitigate environmental impacts from the construction sector has led to significant interest in the reuse of building materials, particularly structural steel. This publication addresses this need by focusing on design strategies that facilitate the future disassembly and reuse of steel elements. A key point discussed is the importance of integrating design for disassembly (DfD) principles early in the design process, ensuring that structures are not only functional and durable but also adaptable to future needs. The emphasis is placed on modular design, standardised components, and accessible mechanical connections, which collectively enhance the ease of deconstruction and extend the life cycle of materials.

Furthermore, the document underlines the need for traceability and comprehensive documentation throughout the life of a building. Implementing Building Information Modelling (BIM) and maintaining detailed building memos ensure that critical information, such as material properties, design specifications, and fabrication details, is retained, greatly facilitating future reuse efforts. The guide also discusses the loading and structural analysis requirements to ensure that buildings designed for deconstruction maintain high performance standards during their initial and subsequent service lives. By coupling technical rigour with practical strategies, the guide presents a holistic approach to sustainable construction practices.

These recommendations mark a pivotal shift from traditional design practices toward a circular economy model, where materials are kept in use for longer periods. The integration of environmental considerations into structural design, without compromising functionality or safety, highlights the maturity and practicality of the proposed strategies. The publication provides a solid foundation for advancing sustainability in the construction sector, supporting designers, engineers, and policymakers to achieve long-term environmental goals.

The following key conclusions can be drawn from this volume:

- **Design for Reuse:** Integrating design for disassembly (DfD) principles early in the design process enables the future reuse of structural steel elements, supporting a circular economy in construction.
- **Modularity and Standardisation:** Modular building designs and standardised steel components enhance the ease of deconstruction and increase the potential for material recovery and reuse.
- **Reversible Connections:** Using bolted and mechanical connections, rather than permanent welded joints, facilitates the dismantling and repurposing of structural elements.
- **Traceability and Documentation:** Detailed record keeping, including Building Information Modelling (BIM) and building memos, ensures that essential material and fabrication data are preserved for future reuse.
- **Structural Performance:** Designs must balance the need for reusability with rigorous performance standards, ensuring durability, safety, and adaptability over multiple life cycles.
- **Environmental Impact Reduction:** By facilitating the reuse of reclaimed steel products, the recommendations contribute to significant reductions in construction waste and embodied carbon emissions.

- **Industry Advancement:** The guidelines promote a shift in the construction industry toward sustainable practices without compromising functionality, economic feasibility, or safety.
- **Policy and Practice Alignment:** The publication supports the development of industry standards and regulations that can drive broader adoption of reuse-oriented design strategies.

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The ADVANCE project's ambition is to contribute to greenhouse gas reduction and circular economy goals by addressing these challenges in both deconstruction and reuse of existing steel buildings, and in the design of new buildings, their construction and documentation to facilitate future reuse. Its scope includes reuse of constituent products, fabricated components, and reuse of component assemblies. The reused material may originate from primary structures, secondary structures and envelopes. The reduction of greenhouse gas emissions of steel industry became essential in the recent years with the major focus on construction products, the largest contributor of its environmental footprint. The construction sector comprises the opportunity to establish steel-based technologies in a leading position for the decarbonisation of other relevant industries dependent on steel solutions.

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TC14 is the Technical Committee within the European Convention for Constructional Steelwork (ECCS) for Sustainability and Eco-Efficiency of Steel Construction. The committee aims to promote developments in industry, research and teaching communities that strengthen knowledge and capabilities in relation to sustainable steel construction. The broad area of the issues includes, for example, the following aspects: Management of overall building performance during the whole lifecycle; Techniques for the improved environmental performance; Techniques for a high quality and comfort of the indoor environment; Energy efficiency; Minimization of resources and use of raw materials.