ECCS
CECM
EKS

Funded by
the European Union

RECOMMENDATIONS FOR REUSE OF STEEL PRODUCTS

— Volume 1: Reusing existing steel products and buildings —

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

ADV1-EN | 2025

ECCS TC14
Sustainability & Eco-Efficiency of Steel Construction

Recommendations for Reuse of Steel Products

Volume 1: Reusing existing steel products and buildings

2nd Edition, 2025

ECCS
CECM
EKS

Recommendations for Reuse of Steel Products

Volume 1: Reusing existing steel products and buildings

Nº 146, ADV1-EN, 2nd edition, 2025

Published by:
ECCS – European Convention for Constructional Steelwork
publications@steelconstruct.com
www.eccspublications.eu

ISBN: 978-92-9147-202-4

2023-2025
ADVANCE
ACCOMPANYING MEASURE FOR DISSEMINATION, VALORISATION AND COLLABORATIVE EXPLOITATION OF CIRCULARITY OF CONSTRUCTIONAL STEEL PRODUCTS

This work was funded by the European Union under the grant no. 101112269. Views and opinions expressed are however those of the author(s) only and do not necessarily reflect those of the European Union. The European Union cannot be held responsible for them.

Funded by
the European Union

https://www.steelconstruct.com/eu-projects/advance/

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 Gervásio	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

TABLE OF CONTENTS

NOTATION

Lower case

f_u	Tensile strength
f_y	Yield strength
$f_\mathrm{y}(t)$	Yield strength based on plate thickness
k_n	Value taken from Table D1 of EN 1990
m	Group mean value
n	Exponent
w	Deflection

Upper case

E	Elastic modulus
F	Action
G	Shear modulus, permanent action
$G_{\mathrm{k},j,\mathrm{sup}}$	Upper characteristic (superior) value of permanent action *j*
$G_{\mathrm{k},j,\mathrm{inf}}$	Lower characteristic (inferior) value of permanent action *j;*
H_v	Vickers hardness value
$K_{\gamma_{M1}}$	Correction factor
P_f	Probability of failure
Q	Variable action
$Q_{\mathrm{k},1}$	Leading variable action
$Q_{\mathrm{k},j}$	Accompanying variable action *i*
R_eH	Yield strength from testing or relevant product standard
R_m	Ultimate strength from testing or relevant product standard
$R_{\mathrm{p}0,2}$	0.2% Offset Yield Strength
S_x	Standard deviation
V_x	Coefficient of variation
X	Material or product property;
$\bar{X}$	Mean value of a material or product property
X_d	Design value of interest
X_k	Characteristic value of interest

Greek letters and symbols

α	Coefficient of linear thermal expansion
α_{R}	Importance factor of a material property
β	Reliability index
γ_{G}	Partial factor for actions (generic)
γ_{m}	Partial factor for a material property
γ_{M}	Partial factor for resistance (generic)
γ_{M0}	Partial factor for resistance of cross-sections
γ_{M1}	Partial factor for resistance of members to instability
$\gamma_{\mathrm{M1,mod}}$	Modified partial factor for resistance of members to instability
γ_{M2}	Partial factor for resistance of cross-sections in tension to fracture
γ_{Rd}	Partial factor covering uncertainty in the resistance model
ε_{f}	Elongation after fracture
ν	Poisson’s ratio
ρ	Air density
ξ	Reduction factor for unfavourable permanent actions
ψ	Combination factor
ψ_{0}	Combination factor for variable action
$\psi_{0,i}$	Combination factor for variable action *i*
Φ	Normal distribution

Subscripts

ad	Adjusted
d	Design value
inf	Inferior
k	Characteristic value
mod	Modified
nom	Nominal
sup	Superior

Abbreviations

CEN	European Committee for Standardisation
CEV	Carbon equivalent value

CFC	Chlorofluorocarbon
CHS	Circular Hollow Sections
CoV	Coefficient of variation
CPR	Construction Products Regulation
$D_{0\text{-}A}$	In situ reuse scenario
$D_{I\text{-}B}$	Reuse scenario: same configuration and same site
$D_{I\text{-}C}$	Reuse scenario: different configuration and same site
$D_{I\text{-}D}$	Reuse scenario: same configuration and different site
$D_{I\text{-}E}$	Reuse scenario: different configuration and different site
DCL	Low ductility class systems for seismic design according to EN 1998-1-2
DoP	Declaration of Performance
DT	Destructive Testing/Test
EN	European Norm
ETA	European Technical Assessment
EU	European Union
EXC	Execution class(es)
FEM	Finite element method
H-CFC	Hydrochlorofluorocarbon
hEN	European Harmonised Standard
LCA	Life-cycle assessment
LCC	Lifecycle cost assessment
LSD	Limit states design method
MSB	Multi-storey steel building
NA	National Annex
NAD	National Application Document
NDT	Non-Destructive Testing/Test
$P\text{-}\Delta$	Global second order effects
$P\text{-}\delta$	Local second order effects
PU	Polyurethane
RHS	Rectangular Hollow Sections
SHS	Square Hollow Sections
SLS	Serviceability Limit State(s)

SSB	Single-storey steel building
STR	Design values of actions for strength
ULS	Ultimate Limit State(s)
Z	Zinc coating by immersing the prepared strip in a molten bath of zinc
ZF	Zinc-iron coating by immersing the prepared strip in a molten bath of zinc and a subsequent annealing
ZA	Zinc-aluminium coating by immersing the prepared strip in a molten bath of zinc-aluminium
ZM	Zinc-magnesium coating by immersing the prepared strip in a molten bath of zinc-aluminium-magnesium
AZ	Aluminium-zinc coating by immersing the prepared strip in a molten bath of aluminium-zinc-silicon
AS	Aluminium-silicon coating by immersing the prepared strip in a molten bath of aluminium-silicon

Axes

x	Longitudinal axis along the member
y	Major axis (parallel to flanges)
z	Minor axis (parallel to web)

1 INTRODUCTION

The construction industry needs to develop more sustainable construction practices that lead to a lower carbon footprint and contribute to the circular economy. The 9R circularity strategies (Refuse-Rethink-Reduce-Reuse-Repair-Refurbish-Remanufacture-Repurpose-Recycle-Recover) may be applied in structural engineering to help develop new design approaches and systems that reduce environmental impacts and improve the overall structural efficiency of construction. In the design, construction and maintenance of steel structures, the 9R framework can be understood as follows:

- *Refuse* producing unnecessary steelwork. If there is a way to achieve the same goals with already existing infrastructure, it should be preferred,
- *Rethink* the end-of-life options for your steel building. Demolition is not the only solution,
- *Reduce* CO_2 emissions and energy demands associated with steel production and/or recycling, *reduce* waste, and *reduce* material use by developing more efficient structural systems,
- *Reuse* reclaimed steel products, where possible, to substitute the use of new steel.
- *Repair* the damaged components. Design the new steelwork so that the components can be accessed, inspected and repaired,
- *Refurbish* the existing structure, verify its integrity, stability and serviceability to extend its design life,
- *Remanufacture* recovered obsolete components to comply with the current specifications instead of recycling them,
- *Repurpose* the steelwork if it is no longer needed. It may serve well in a less demanding environment or in a different industrial application,
- *Recycle* steel components that cannot be repaired, refurbished, remanufactured, or repurposed to minimise the depletion of primary resources and minimise environmental impacts,
- *Recover* what is left and find the best circular solution to it. Steel is not the only material in the building.

This publication focuses on the reuse of constructional steelwork, but the concepts of its refurbishment, remanufacturing, repairing and repurposing are addressed as well, because they are often overlapping. Refusing and rethinking strategies are not covered in this publication, because they are related to the adaptive reuse and sharing of architectural spaces, rather than technical solutions for dismantling and/or physical modifications of existing steel structures.

The reduction of carbon emissions associated with the production of materials and the reduction of waste are important drivers in construction. As part of the philosophy of circularity in construction, Kibert [1] presented some basic steps necessary to obtain the use and recovery of closed-loop materials and to reduce waste at the end of the life of a building. This means that the building should be designed for flexibility in use and that at the end of its life, its materials must be reusable or recyclable.

In the context of the reuse of steel structures, new steel sections are supplied with a certificate that guarantees their properties. Reused steel sections need an equivalent guarantee of their performance, and in the absence of other information, material testing is required for reuse of these sections.

There are significant challenges that must be addressed, particularly concerning the assessment of the adequacy and reliability of reclaimed steel to ensure that:

- The reclaimed steel members meet the performance requirements for the mechanical, physical, dimensional, and other relevant properties to ensure their adequacy in design to EN 1993,
- The salvaged materials meet the quality requirements of the nominal specifications to ensure their reliability for use. For structural steel, the relevant standard for structural design is EN 1993 with its various parts,
- Structures made from reclaimed steel must have long-term integrity and long-lasting durability in their subsequent use.

These are key aspects to solve in order to show that reclaimed steel can be an economically and structurally viable alternative to the use of new steel in buildings. Reuse can be considered at all structural levels, i.e. individual members, structural components, such as a truss system, a frame or a sandwich panel and the whole structure or part of it.

The main purpose of this publication is to provide recommendations and practical information on the fabrication and detailing of buildings made from reclaimed steel and on the design of buildings for future demounting and reuse.

Other purposes of this guide are as follows:

- To establish acceptability criteria in terms of geometry, member condition, and material properties to enable the potential reuse of steel products, according to CEN / TS 1090-201 [2],
- To address the identified barriers to steelwork reuse [3], in particular the sourcing and procurement of reused steel, the cost implications for reuse of structural steel and recertification of steel members for reuse.

1.1 Scope of this publication

These recommendations provide design guidance on the improvement of existing procedures for designs using reclaimed steel products, and provide information on design for future adaptability, demountability and reuse. The recommendations are presented as guidelines for steel reuse in the context of Eurocode design. For a specific location, the relevant National Annexes may require use of country-specific design parameters that may also affect the reuse of steelwork.

The main target audience of this guidance is structural engineers and architects who are interested in reusing reclaimed structural steel today and designing new steel buildings that can be deconstructed and reused more easily in the future. Mainstream reuse of structural steel will require action by all parts of the steel construction supply chain and therefore this guide may also be useful to the wider audience. It covers recommendations for single-storey steel buildings, such as industrial halls and multi-storey steel and composite structures.

Single-storey steel buildings are particularly suitable for reclaiming and reusing structural steelwork because:

- They have a repetitive structural system that conforms to well defined structural forms,
- They are readily assembled and disassembled,
- The structural members are usually visually exposed and accessible at a relatively safe working height,
- They are usually low occupancy structures,
- Normally these structures do not have fire protection,
- They have a good potential for standardisation in their geometry and use of the primary components,
- Each component is easy to document.

Reclaiming and reusing structural steelwork from *multi-storey steel buildings*, most of which have steel-concrete composite structures, presents a difficult challenge, as the steel beams are connected to the concrete composite floor through shear connectors with very limited accessibility. Chen et al. [4] presents and discusses five potential reclaiming methods to overcome the difficult challenge of accessing and cutting weld-through-deck shear connectors to reclaim the structural steel section from composite beam for reuse, i.e. band-saw cutting, wire-saw cutting, wall-saw cutting, laser drilling and diamond core drilling. However, these reclaim methods were implemented in laboratory trial tests only to reclaim steel sections from small-scale composite specimens. In practice, existing multi-storey steel buildings are generally reused in situ (e.g., through major refurbishment) due to their complex construction, composite systems, and context-specific engineering constraints. Relocating them to new sites typically requires extensive redesign, which limits practical feasibility. On the contrary, dry floor systems, when integrated into multi-storey buildings, offer enhanced disassembly potential for individual components. Importantly, these concepts have already been implemented in real-world projects - for example, dismountable and reusable precast concrete frames have been successfully applied in several building cases in Finland [5],[6]. Several innovations for the design of new, reusable, multi-storey systems are presented in the second volume of this guide.

1.2 Design, execution and product standards

These Recommendations are prepared to assist in structural engineering work, and refer to requirements and principles given in the following standards:

- EN 1090-1:2009+A1:2012 [7] - Execution of steel structures and aluminium structures, Part 1: Requirements for conformity assessment of structural components (incorporating CEN amendment A1:2012),
- EN 1090-2:2018+A1:2024 [8] - Execution of steel structures and aluminium structures, Part 2: Technical requirements for steel structures (incorporating CEN amendment A1:2024),
- CEN/TS 1090-201:2024 [2] - Execution of steel structures and aluminium structures - Reuse of structural steel,
- EN 1990:2023 [9] - Basis of structural and geotechnical design,
- EN 1991-1-1:2002 [10] - Actions on structures, Part 1-1: General actions – Densities, self-weight, imposed loads for buildings (incorporating CEN corrigenda Dec. 2004 and Mar. 2009),

- EN 1991-1-3:2002+A1:2015 [11] - Actions on structures, Part 1-3: General actions – snow loads (incorporating CEN corrigenda Dec. 2004 and Jun. 2009, and CEN amendment A1:2015),
- EN 1991-1-4:2002+A1:2010 [12] - Actions on structures, Part 1-4: General actions – wind actions (incorporating CEN amendment A1:2010),
- EN 1991-1-5:2003 [13] - Actions on structures, Part 1-5: General actions - Thermal actions,
- EN 1991-1-6:2005 [14] - Actions on structures, Part 1-6: General actions - Actions during execution,
- EN 1993-1-1:2022 [15] - Design of steel structures, Part 1-1: General rules and rules for buildings,
- EN 1993-1-3:2024 [16] - Design of steel structures, Part 1-3: General rules – supplementary rules for cold formed members and sheeting,
- EN 1993-1-8:2024 [17] - Design of steel structures, Part 1-8: Design of joints,
- EN 1993-1-10:2005 [18] Design of steel structures, Part 1-10: Material toughness and through-thickness properties (incorporating CEN corrigenda Dec. 2005, Sep. 2006 and Mar. 2009),
- EN 1994-1-1:2004 [19] Design of composite steel and concrete structures - General rules and rules for buildings.

The following product standards, which specify geometrical and mechanical requirements, were used in the preparation of this document and should be used in conjunction with this document:

- EN 10025-1:2004 [20] - Hot rolled products of structural steels, Part 1: General technical delivery conditions,
- EN 10025-2:2019 [21] - Hot rolled products of structural steels, Part 2: Technical delivery conditions for non-alloy structural steels,
- EN 10025-4:2019 [22] - Hot rolled products of structural steels, Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels (incorporating CEN amendment A1:2022),
- EN 10025-5:2004 [23] - Hot rolled products of structural steels, Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance,
- EN 10029:2010 [24] - Hot rolled steel plates 3 mm thick or above, Tolerances on dimensions and shape,
- EN 10034:1993 [25] - Structural steel I and H sections – Tolerances on shape and dimensions,
- EN 10051:2024 [26] - Continuously hot rolled strip and plate/sheet cut from wide strip of non-alloy and alloy steels – Tolerances on shape and dimensions,
- EN 10055:1998 [27] - Hot rolled steel equal flange tees with radiused root and toes – Dimensions and tolerances on shape and dimensions,
- EN 10056-1:2017 [28] - Structural steel equal and unequal leg angles, Part 1: Dimensions,
- EN 10056-2:1993 [29] - Structural steel equal and unequal leg angles, Part 2: Tolerances on shape and dimensions,
- EN 10204:2004 [30] - Metallic products – Types of inspection documents,

- EN 10210-1:2006 [31] - Hot finished structural hollow sections of non-alloy and fine grain steels, Part 1: Technical delivery requirements,
- EN 10210-2:2019 [32] - Hot finished structural hollow sections of non-alloy and fine grain steels, Part 2: Tolerances, dimensions and sectional properties,
- EN 10219-1:2006 [33] - Cold formed welded structural hollow sections of non-alloy and fine grain steels, Part 1: Technical delivery requirements,
- EN 10219-2:2019 [34] - Cold formed welded structural hollow sections of non-alloy and fine grain steels, Part 2: Tolerances, dimensions and sectional properties,
- EN 10279:2000 [35] - Hot rolled steel channels – Tolerances on shape, dimension and mass,
- EN 10346:2015 [36] - Continuously hot-dip coated steel flat products for cold forming – Technical delivery conditions,
- EN 10169:2022 [37] - Continuously organic coated (coil coated) steel flat products - Technical delivery conditions,
- EN 10365:2017 [38] - Hot rolled steel channels, I and H sections – Dimension and masses,
- EN 14399 series [39] - High-strength structural bolting assemblies for preloading (all parts),
- EN 14509:2013 [40] - Self-supporting double skin metal faced insulating panels, Factory made products, Specifications.

1.3 Terms and definitions

For the purposes of this guide, the following terms and definitions have been used.

Cladding	Façade and roof elements that form the building envelope and provide the required thermal and acoustic insulation, water- and air- tightness, fire protection, aesthetic appearance and have load-bearing capacity.
Component	Part of a steel structure, e.g. truss, sandwich panel. May be an assembly of several smaller components, e.g. joints etc.
Consequences classes	Eurocode based classification of buildings, based on the consequences of failure or malfunction on people, economy, or the environment; different reliability indices are associated with each consequence class.
Constituent product	Materials or products used in manufacturing building structures, with properties used in calculations of 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 used in construction (primary and secondary) and steel-based cladding.

Deconstruction (or disassembly, or demounting)	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 in which a building is taken apart with little or no attempt to recover any of its constituent components for reuse; materials resulting from demolition may, however, be recycled.
Design working life	Assumed period for which the component is to be used for its intended purpose with anticipated maintenance, but without major repair.
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.
Building envelope (or Envelope)	Components or parts of the building that separate the enclosed space from the external environment and provide a range of structural and building physics functions.
Execution class	Eurocode based 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.
Importer	Any natural or legal person established within the EU who places a construction product from a third country in the EU market.
In-situ reuse	Reuse of the component or structure, without relocation, on the same site. For example, a building structure can be retained and reused during building renovation.
Manufacturer	Any natural or legal person who manufactures a construction product or has such a product designed or manufactured and markets it under their name or trademark.
Notified body	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-demolition audit	Qualitative and quantitative assessment of construction and demolition waste streams prior to deconstruction, demolition, or renovation of buildings and infrastructures.
Primary structure or Primary steelwork	Primary steel frame, comprising all main load bearing elements, e.g. columns, beams and bracing.
Provenance	Basic information on the previous use of a reclaimed structural component
Purchaser	Company that purchases the steel products; generally, a steelwork contractor that manufactures the structural steelwork.
Reconditioning	Process of returning a product to good working condition by replacing components that are faulty and updating the appearance of a product, such as by cleaning, painting, or refinishing.
Recycling	Process of converting discarded materials into new materials and products; recycling steel involves remelting of scrap to form new semi-finished products.
Refurbishing	Renovation process of an existing building to suit a new use, which can involve a range of processes from replacing fitments and fittings to major structural alterations.
Relocated reuse	Reuse, when the process requires transport of the structure or component to reuse it on another site. Opposite to in-situ reuse.
Remanufacturing	Returning a product or component to the performance specification of the original manufacturer.
Repairing	Fixing a fault, but without a guarantee of the product as a whole. In the context of steel structures, this can mean strengthening a component.
Repurposing	Any operation that changes the function or purpose of a component.
Reuse	Use of old components with little or no modifications, largely in their original form; they may be reused for the original function or repurposed to a new function.
Secondary structure	Secondary steelwork, consisting of side rails and purlins, used to support the cladding/envelope. In some cases, it may provide restraint to the primary structure.
Structural component	Component within a structure designed to provide mechanical resistance and stability and/or fire resistance, including aspects of durability and serviceability.

Structural kit	Set of standardised structural components that are assembled and installed on site.
Supplier	Company stocking and supplying steel products to the market.
Test unit	Group of individual reclaimed structural components with identical geometrical properties and same function for which relevant properties can be derived from testing a single or a few representative member(s).
Type 1 structural steel	Structural steel material produced in or after 1970 and with a set of relevant properties, including their variations equivalent to one of the standard structural steel materials listed in EN 1993-1-1
Type 2 structural steel	Structural steel material produced before 1970 and with relevant properties that might significantly deviate from those of standard structural steel materials listed in EN 1993-1-1
Waste	Unwanted or undesired material to be discarded.

2 COMPONENTS OF SINGLE- AND MULTI-SOTREY BUILDINGS

2.1 Single-storey steel buildings (SSB)

Single-storey steel buildings (SSBs) are very versatile and commonly used in various applications such as warehouses, retail spaces, industrial and agricultural facilities. Their inherent properties, such as resistance and flexibility, make them ideal for a wide range of design requirements.

SSBs are classified based on their structural systems, which are tailored to meet specific functional requirements and exposure conditions. Primary structural systems employed in SSBs include portal frames and trusses. Each system has different features that influence the design, construction and overall performance of the building. SSBs generally feature long spans and may provide office spaces in a connected structure or on a mezzanine floor.

The widespread use of SSBs demonstrates the adaptability of steel as a construction material, offering a variety of configurations tailored to meet specific structural, aesthetic, and operational demands while ensuring efficiency and safety. SSBs can be easily adapted for different functionalities while providing a quick construction timeline and cost efficiency. This modern construction method, characterised by its lightweight, fire resistant, and sustainable materials, continues to evolve, offering improved designs and technologies for better performance and durability in building applications.

The schematic arrangement of a typical single-storey building using a portal frame system is shown in Fig. 2.1 [43].

There are three layers to the structure:

- Primary steelwork consisting of the frames and bracing system in the roof and walls,
- Secondary steelwork, consisting of side rails for the walls and purlins for the roof, which are usually from cold rolled elements,
- Wall and roof cladding, typically in the form of sandwich panels (also called composite panels) and double-skin built-up roof systems.

The loading comprises of:

- Self-weight of the structure and its components, including equipment, which is supported by the structure,
- Variable load acting on the structure by the occupancy and use of the building,
- Loads from environmental effects e.g. snow, wind or thermal loads and seismic actions.

Framing options for single-storey buildings are as follows:

- Simple beam/columns often referred to as braced-box structures,
- Portal frames and their variants for a range of medium span applications,
- Lattice structures (trusses) for longer spans or heavy roof loads.

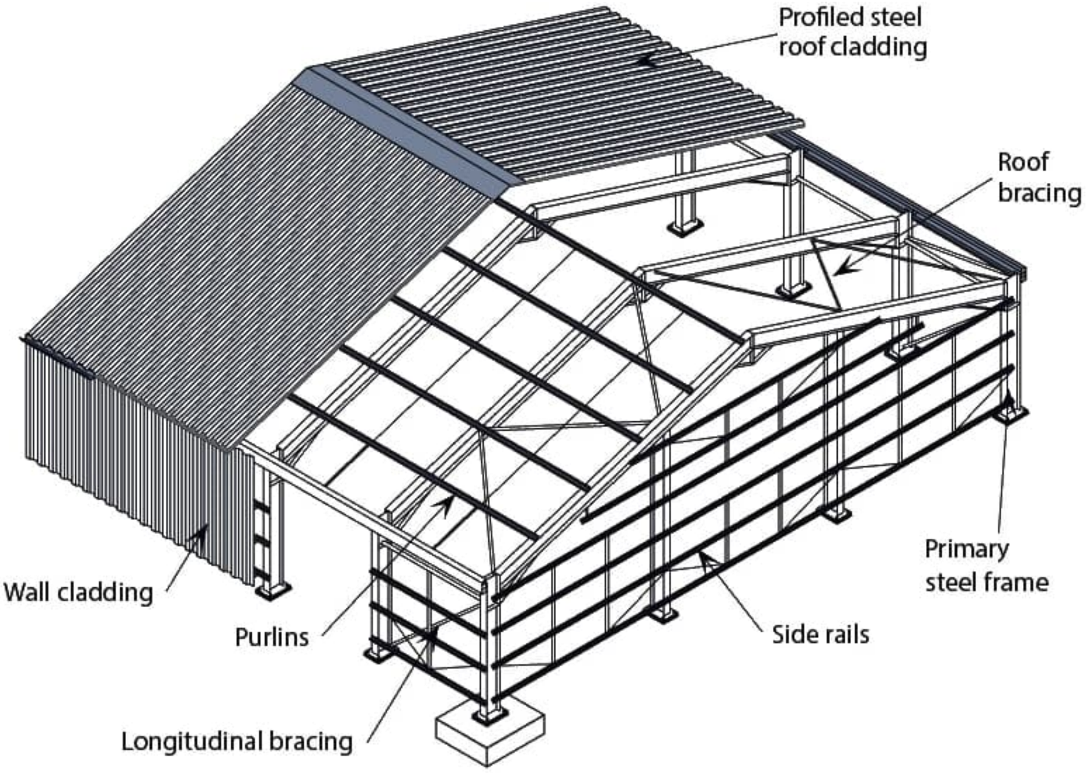


Fig. 2.1 Arrangement of a typical one-bay portal steel frame single-storey building [43]

2.1.1 Primary steelwork: structural frames and bracing system

Portal frame structures are one of the most common systems and can be pitched or flat. These systems consist of a series of rigidly connected frames that typically comprise vertical columns and beams in the transverse direction, a bracing system on the roof and longitudinal walls and purlins, side rails, and roofing systems. The columns serve as vertical supports, transferring loads from the roof to the foundation. The beams span the distance between the columns, providing the necessary support for the roof. The members are often made from rolled steel sections, welded sections, or hollow steel tubes. The rigidity of the connections allows the frames to withstand lateral loads, making them particularly suitable for areas exposed to wind or seismic forces. The design of the portal frame allows for wide-open spaces and flexible interior layouts many times without the need for internal columns, which is advantageous for warehouses or retail spaces.

Structural frames can be designed according to the principles of simple construction, continuous construction, or semi-continuous construction. In simple construction, the joints between the members are pinned, i.e. they have low rotational stiffness and do not transmit significant bending moments. Horizontal loads are resisted by a bracing system. In continuous construction, the joints are designed to provide sufficient stiffness to be considered rigid and therefore transmit bending moments between the members. Semi-continuous construction is designed to resist some moment, but not as much as in a fully rigid (continuous) frame. This allows for more flexibility in design and can improve structural performance and economy.

Truss systems are another option in single-storey steel buildings, to create a lightweight yet strong framework. Trusses efficiently support the roof while minimising material usage. Depending on the design, the truss systems can be pitched or flat, allowing for various architectural aesthetics and functional needs. Trusses distribute loads through the network of connections, making them effective for spanning large distances without internal support, which can be beneficial for sports facilities or outdoor pavilions.

Secondary elements provide support to the roofs and walls. The term *secondary* does not refer to the importance of the element, but rather to its order in the construction process. These elements consist of side rails and purlins or sometimes deep decking and cassettes that transfer loads back to the main frame. The cladding essentially provides a controlled internal environment to the building and includes components such as roof-lights and ventilation outlets. Both the cladding and the secondary steelwork can provide buckling restraint against buckling of the primary frame elements.

2.1.2 Roof structures

Roof systems are designed to transmit loads and form the enclosure to the building to maintain the required internal environment and function. From a structural point of view, roof systems are designed to support self-weight, permanent loads from secondary elements and cladding, imposed loads, snow loads, and wind loads, including uplift. The roof also provides acoustic and thermal insulation, so that the building envelope is airtight and waterproof.

The common roof systems in single-storey structures are shown in Fig. 2.2, which are:

- Pitched roof portal frames (using inclined beams/rafters connected with rigid joints to the columns) for spans up to ~50 m. They may have single- or multi-bay configurations and the pitch or slope is generally about 6^o to the horizontal;
- Trusses with a sloping top chord for spans up to ~100 m, or for heavy loads acting on the roof.

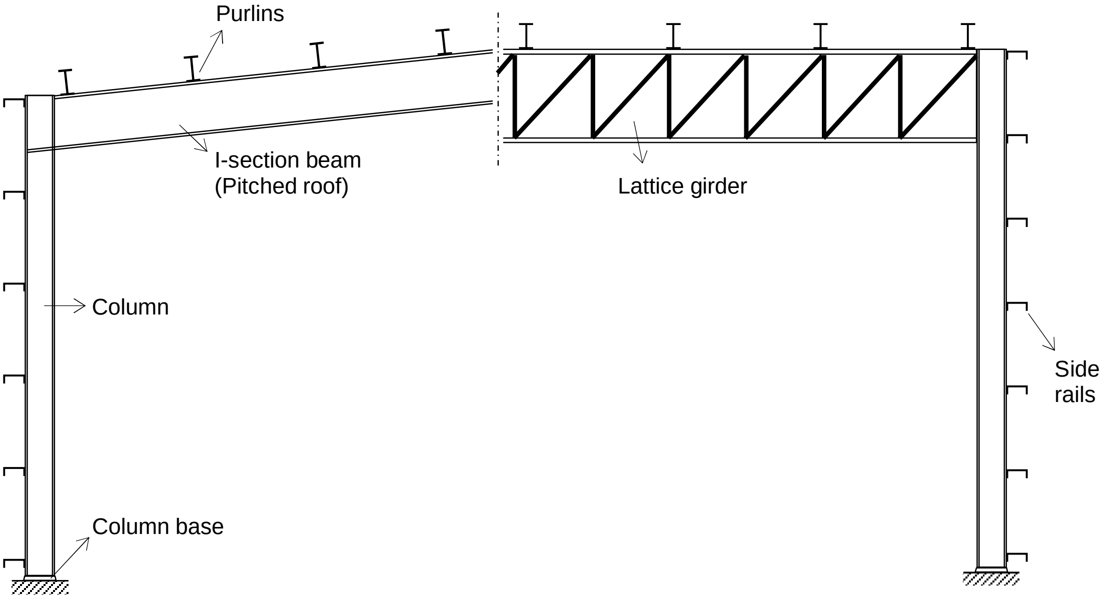


Fig. 2.2 Structural elements of typical single-storey steel buildings

2.1.3 Bracing systems

Bracing systems used in single-storey buildings may be of three categories:

- Permanent bracing,
- Temporary bracing,
- Restraint bracing to compression flanges and column splices.

Permanent bracing is designed to provide overall stability to the structure. They often include trusses (i.e., triangulated straight interconnected elements), or diaphragms provided by the roof cladding. When cross-bracings are used, they may act as tension-only members, allowing the compressed member of the cross-brace to buckle. Roof sheeting may act as a diaphragm and considerably stiffen the building.

Temporary bracing may be required depending on the construction sequence.

The most common option is to rely on secondary steelwork and fly bracings to limit the overall buckling of the primary members, as well as the buckling of compression flanges of the primary members. However, for buildings of considerable size and/or for some types of construction, an additional bracing system may be required.

2.1.4 Secondary steelwork

Secondary steelwork on the roof typically comprises a system of steel purlins spanning between the primary beams. Traditionally, hot rolled profiles were used as purlins, but more recently, cold formed purlins became very popular. Depending on the type of cladding, the purlin spacings are usually between 1.2 and 2.5 m (1.8 m being a typical value). Secondary steelwork may not be required in the case of long-span cladding that spans directly between the primary frames. For typical portal frame applications, a continuous system with overlapped or sleeved solutions is often used.

Roof purlins are usually I-beam hot rolled steel members, C- or Z-shape cold formed members, with sigma or omega sections as alternative options. They are designed to:

- Transfer loads from the roof cladding to the primary steel frame, including imposed loads due to snow and maintenance access,
- Transfer horizontal loads to the bracing system,
- Provide restraint to the beams of the primary frame.

The purlins and side rails are usually supplied as part of the cladding support system, together with fittings, fasteners and other associated components.

The wall cladding is often supported by horizontal side rails that span between the columns of the primary frame. Nowadays, cold formed C sections are a very popular option for side rails, but cold formed Z- sections may be used if the side rail is designed to be continuous over several columns. The side rails are designed to:

- Transfer load, including wind load, from the wall cladding to the columns,
- Transfer horizontal loads to the bracing system,
- Provide lateral restraint to the columns.

The cladding also stiffens the wall and transfers a significant proportion of the vertical load to the columns directly, while also restraining the side rails against buckling.

2.1.5 Cladding systems

The main function of the cladding system is to provide a controlled internal environment depending on the intended use of the building. General requirements of the cladding system are as follows:

- Provide the required level of thermal insulation,
- Resist wind pressure, and wind uplift through the fixings to the purlins,
- Prevent spread of fire,
- Provide an airtight building envelope,
- Include measures for ventilation of a building, e.g. by mechanical equipment,
- Provide acoustic insulation compatible with the intended use of the building,
- Stabilise secondary steel members, and sometimes primary steelwork, with suitable restraints.

In single-storey buildings, short span claddings (spans up to 2 - 3 m) are usually fixed to the secondary steelwork. As an alternative, long span cladding can be used, with spans up to 10 m. Long-span cladding can be in the form of deep trapezoidal sheeting or panels on roofs and horizontally installed sandwich panels on walls that span between the frames. This solution reduces the number of assembly elements and the number of building layers.

Typical cladding systems are as follows:

- Single-skin trapezoidal sheeting,
- Double-skin systems,
- Standing seam sheeting,
- Standing seam panels with liner trays,
- Composite panels often called sandwich panels.

2.1.6 Design guides for SSBs: additional documents

A set of design guides on single-storey steel buildings (SSBs) has been prepared under the direction of Arcelor Mittal, Peiner Träger and Corus. The technical content was prepared by CTICM and SCI, collaborating as the Steel Alliance. This is available at https://constructalia.arcelormittal.com/en/news_center/articles/design_guides_steel_buildings_in_europe [41]. It comprises of:

- Single-storey steel buildings - Part 1: Architect’s guide;
- Single-storey steel buildings - Part 2: Concept design;
- Single-storey steel buildings - Part 3: Actions;
- Single-storey steel buildings - Part 4: Detailed design of portal frames;
- Single-storey steel buildings - Part 5: Detailed design of trusses;
- Single-storey steel buildings - Part 6: Detailed design of built-up columns;
- Single-storey steel buildings - Part 7: Fire engineering;
- Single-storey steel buildings - Part 8: Building envelopes;
- Single-storey steel buildings- Part 9: Introduction to computer software;
- Single-storey steel buildings - Part 10: Model construction specifications;
- Single-storey steel buildings - Part 11: Moment connections.

Design guides on SSBs can also be found at https://steelconstruction.info/ [43]. These are:

- SCI P164: Design of portal frames for Europe. The Steel Construction Institute, 2001;

- SCI P313: Single storey steel framed buildings in fire boundary conditions. The Steel Construction Institute, 2002;
- SCI P252: Design of single span steel portal frames to BS 5950:2000. The Steel Construction Institute, 2004;
- SCI P346: Best practice for the specification and installation of metal cladding and secondary steelwork. The Steel Construction Institute, 2006;
- SCI P347: Single storey buildings - Best practice guidance for developers, owners, designers & constructors. The Steel Construction Institute, 2006;
- EP37: Best practice in steel construction – Industrial buildings. Guidance for architects, designers & constructors. The Steel Construction Institute, 2008;
- SCI P397: Elastic design of single-span steel portal frame buildings to Eurocode 3. The Steel Construction Institute, 2012;
- SCI P399: Design of steel portal frame buildings to Eurocode 3, The Steel Construction Institute, 2015;
- Target Zero: Guidance on the design and construction of sustainable, low carbon warehouse buildings. Tata Steel and BCSA, 2011;
- Scheme development: Selection of the external wall envelope system for single storey buildings, SS019a-EN-EU. Access Steel;
- Scheme development: Details for portal frames using rolled sections, SS051a-EN-EU. Access Steel;
- Scheme Development: Design of portal frames using fabricated welded sections, SS052a-EN-EU. Access Steel;
- Scheme development: Selection of the external roof envelope system for single storey buildings, SS018a-EN-EU. Access Steel;
- Scheme development: Overview of structural systems for single-storey buildings, SS048a-EN-EU. Access Steel.

This list is not exhaustive, and additional documents specific to certain countries or products may also be applicable.

2.2 Multi-storey steel buildings (MSB)

Multi-storey steel buildings are complex structures that employ various structural systems designed to support the loads encountered during their lifespan. The design of these buildings is based on the use of steel for its strength, versatility, and ability to be fabricated in a wide range of shapes. Multi-storey steel buildings are often classified based on their structural framework, which provide the necessary support and stability for the entire system.

In a multi-storey steel building, the typical layers of the structure are:

- Primary steelwork consisting of frames (a grid of beams, columns, floor decking and the main bracing systems),
- Secondary steelwork, consisting of edge beams, staircases, lighter bracing system and secondary floor beams,
- Cladding system for the roof, in the form of glass roofs, watertight roofs or built-up roof systems, and for the walls, in the form of glazing systems, curtain walling, brickwork, or insulated render or tiles.

One primary classification is based on the type of structural system used, which can include moment-resisting frames, braced frames, and shear wall systems. The choice of framing system is primarily influenced by the building's height, with the following typical options:

- Moment-resisting frames – suitable for buildings up to approximately 4 storeys, where horizontal loads are a key consideration,
- Braced frames – typically used for buildings up to around 12 storeys,
- Steel or concrete cores (shear wall systems) – appropriate for buildings up to about 40 storeys.

The framing for very tall buildings is influenced by their stabilising systems and is not covered by this guide.

Moment-resisting frames are characterised by their ability to resist lateral forces through rigid connections between beams and columns. The structural system allows the frames to sway under lateral loads. Frames typically feature arrangements that promote flexibility while maintaining strength. The rigidity of the connections allows the structure to maintain its shape while evenly distributing the forces throughout the framework. Moment-resisting frames help create open floor plans.

Braced frames, on the other hand, incorporate diagonal braces between the columns and beams to create a triangulated arrangement that enhances stability. These diagonal braces effectively counteract lateral forces, making this system especially suitable for tall buildings. The design reduces the amount of steel needed compared to moment-resisting frames while maintaining structural integrity. Braced frames can be configured in various styles, including X-braces, V-braces, and knee braces, depending on the architectural intent and design requirements.

Shear wall systems incorporate vertical walls made of reinforced concrete or steel, strategically placed within the building to withstand lateral loads. Walls act as vertical cantilevers, effectively transmitting forces to the foundation, making them ideal for buildings designed to withstand high wind pressures or seismic activity, providing added stiffness and stability to the structure. The integration of shear walls can significantly enhance the rigidity of the structure while allowing for open floor plans.

The loading conditions applicable to multi-storey steel buildings are as follows:

- Self-weight of the structure and its components, including equipment,
- Variable loads from occupancy and use of the building, including movable partitions,
- Loads from environmental effects, e.g. snow, wind or thermal and seismic actions,
- Loads from accidental actions, e.g. impact from vehicles, explosions.

2.2.1 Structural frames

In multi-storey buildings, the support of the building's weight and the distribution of the loads to the foundation are ensured by a main frame consisting of columns and beams. The number of load-bearing points depends on the intended use, the loads that it will require to bear, and the type of foundation chosen (which itself is influenced by the soil conditions at the building site).

Columns in multi-storey buildings are subjected to significant compression, requiring design considerations for buckling resistance. Depending on the grid size and layout, installation costs, cost of the steel profiles, ease of connection to secondary components (ceilings, walls, façades), degree of compliance with fire or corrosion resistance requirements or architectural preferences, the main type of columns generally used in multi-storey structures are H-section profiles (steel or composite) and circular hollow section profiles (steel or composite).

The beams play a role in the transfer of vertical loads. Primarily subjected to bending moments, these horizontal elements require sufficient stiffness and strength to ensure structural integrity. There is a wide variety of beam sections available, such as hot rolled sections suitable for small and medium spans, built-up sections welded from plates (usually used for longer beams), cellular profiles built from hot rolled sections with circular or hexagonal openings cut in the web (resulting in increased strength and openings for ducts) and composite sections consisting of a steel profile jointly working with the concrete slab through steel studs or steel decking.

Composite beams are suitable for large span floors (up to ~20 m) as they take advantage of the large compressive area of the concrete slab and the high tensile strength of the steel beam, showing exceptional mechanical performance in load-bearing capacity and stiffness. Several types of composite beams are available: plain slabs that require temporary formwork, precast concrete slabs with steel decking and encased beams, where the steel profile is partially concrete encased for increased fire resistance. In multi-storey buildings, slim floor designs are generally used to maximise usable space. This solution integrates the steel beam directly into the height of the concrete slab, reducing the overall floor depth.

Fig. 2.3 shows the span ranges for different floor structural options.

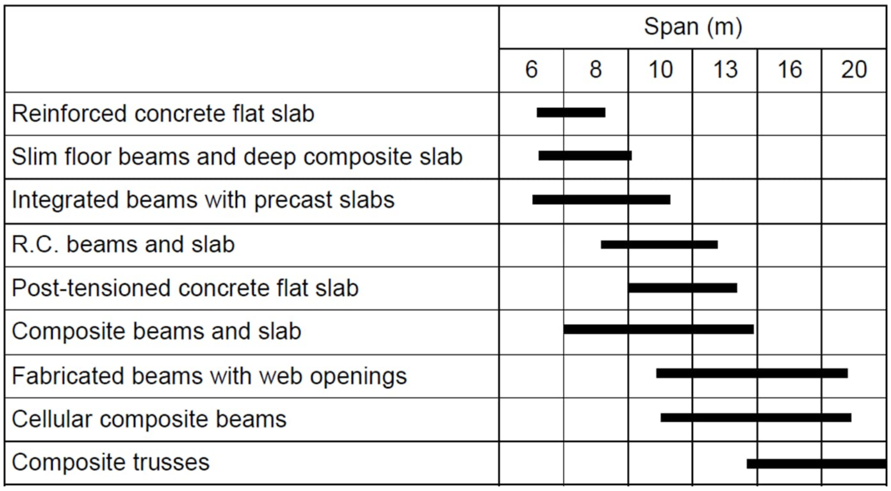


Fig. 2.3 Span ranges for floor structural systems [41],[44]

2.2.2 Bracings for multi-storey buildings

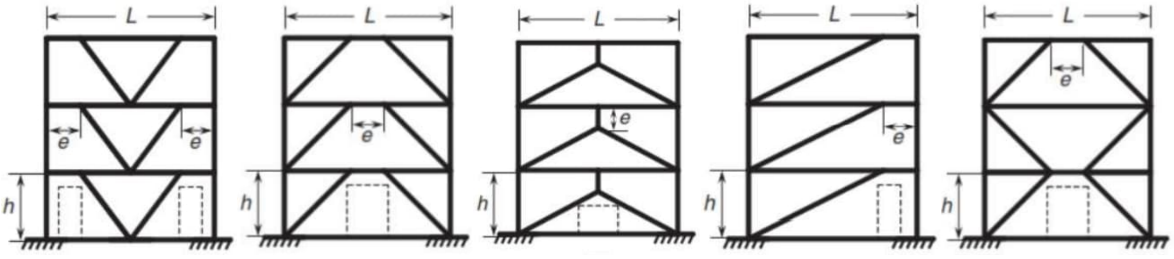
The global stability of multi-storey buildings may be ensured by bracing systems, in addition to the frame effect. Using bracing for stability jointly with rigid frames is a cost-effective solution, as using solely rigid frames to maintain stability requires heavier columns and beams, which leads to higher costs as the number of stories in the building increases. Therefore, in an optimal economic solution, frames can provide primary support in one direction, while bracing systems are located in the perpendicular direction to enhance stability.

Braced steel frames incorporate various bracing configurations within the walls, typically concealed within façade cavities or around stairwells and service areas. In seismic areas, several steel framing systems can be used to improve resistance to seismic loads. These systems include:

- Moment-Resisting Frames (MRF): These frames rely on the stiffness of the beams and columns to resist lateral seismic forces,
- Concentrically Braced Frames (CBFs): CBFs use diagonal braces within the frame to resist shear forces. In CBFs, the longitudinal axis of the braces is concentric to the midpoint of the beam,
- Eccentrically Braced Frames (EBF): Similarly to CBFs, EBFs incorporate braces, but these braces are designed to bend and dissipate energy during an earthquake. In EBFs, the longitudinal axis of the braces is eccentric to the midpoint of the beam.
- Buckling Restrained Braced Frames (BRBF): BRBFs address a limitation of traditional braces by incorporating buckling restraints that prevent brace buckling under seismic loads,
- Special Plate Shear Walls (SPSW): These walls, constructed of steel plates, act as rigid elements to resist shear forces induced by seismic loads.

The EBF system, which is a structural system that combines the MRF system and the CBF system, providing high ductility (as does the MRF system) and high elastic stiffness (as does the CBF system), includes several types of bracing configuration, as shown in Fig. 2.4.

Cross-bracing systems are used as well in multi-storey buildings to provide vertical stability, as these buildings are commonly designed with pinned members. Cross-bracing systems can be placed outside or inside the building.



a) V-braces b) K-braces c) EBF with vertical link d) Split K-braces e) Inverted Y-braces

Fig. 2.4 Types of bracing configurations in EBF system

2.2.3 Floors in multi-storey buildings

In a multi-storey building structure, floors perform the function of transferring loads to the vertical elements, and, at the same time, they provide stability in the horizontal plane due to the diaphragm effect they bring, contributing to the global stability of the structure. In multi-storey buildings, floor vibrations may be a critical factor in the design. Building services can be integrated into the floor construction itself. Alternatively, services can be suspended below the floor. In commercial buildings, raised floors allow for easy distribution of services within the space beneath the floor surface. This facilitates future maintenance and modification of these services without disrupting the building's operations.

The selection of a floor system is based on several factors, and the common options include the following:

- Concrete slab with steel decking: a simple steel decking is installed as a permanent formwork for a concrete slab that contributes to the bending resistance of the floor, as a tension component. The good connection between the concrete and the decking is ensured by the embossments of the steel sheet,
- Composite slab and composite beams with steel decking: a steel deck is installed as a permanent formwork for a composite slab, contributing to the bending resistance of the beams. The good connection between the concrete and the steel beam is ensured by steel stud connectors welded to the top flange of the steel beam,
- Prefabricated composite slab: prefabricated slab elements with up to 7 m length and 1.20 m width,
- Hollow core slabs: prefabricated hollow-core slab elements are commonly integrated with the steel beams. Hollow-core slabs can be placed on the lower flange of the steel beam or on L-shaped steel profiles welded to the web of the steel beam. To ensure the diaphragm effect of the floor, a reinforced concrete layer is laid on top,
- Precast slab with in-situ topping: plain slabs that use a combination of precast concrete slabs and cast-in-situ concrete. During the pouring of the in-situ concrete, temporary supports may be necessary to handle the combined weight of the precast slab, fresh concrete and construction workers. By incorporating appropriate connections, such as welded studs, between the slab and the supporting beams, the slab can improve the resistance to bending and stiffness of the beams,
- Dry floors: a floor made of assembled individual components such as plaster board, wood board, profiled steel sheeting, and mineral wool. Profiled steel sheeting offers a solution for incorporating building services.

2.2.4 Cladding systems

Typical cladding systems for multi-storey buildings are as follows:

- Brickwork cladding: for buildings up to three stories, brickwork cladding can be directly supported by the ground. Taller structures require additional support, typically achieved with stainless steel angles, individual brackets, or bracket angles connected to the structural frame,
- Gazing cladding systems: these glazing systems, used for tall buildings, typically involve triple glazing or double-layer facades. They are supported by aluminium posts or glass fins, offering a variety of design options. Corner fixings are common, and the panels often incorporate gaskets at their joints for weatherproofing. Also, there are discrepancies between the tolerances of steel frames and glazing panels, therefore, the connections need to allow for adjustments during installation. Thermal expansion and contraction may also occur, requiring the support system to accommodate movement effectively,
- Curtain walling systems: these cladding systems involve aluminium or other lightweight panels that are attached to the perimeter steelwork, stone cladding or precast concrete panels. The cladding system may be able to support its own weight and applied loads (the panel is hung at the top of the panel or supported at the base, from the floor), or it may require additional support (support, which typically comes in the form of mullions, vertical elements that can span multiple floors; in some cases, horizontal members called transoms can be used to connect mullions at intermediate levels, further enhancing stability). Every cladding panel system uses

a unique fixing method designed to allow movement and adjustments in three directions. As these fixing details connect the cladding to the building structure, the design of the floor slab may require adjustments to handle the localised forces exerted by the cladding. A common solution involves incorporating a dovetail channel directly into the edge of the slab during casting, providing a secure and compatible location for attaching the cladding fixings;

- Insulated tiles or render cladding systems: these cladding systems are supported on light infill walls (insulation and render, or tiles, are supported by a secondary steel frame). Tiles may be individual or preformed panels. This cladding system also allows using brickwork as external skin.

2.2.5 Design guides for MSBs: additional documents

A set of design guides on multi-storey steel buildings (MSBs) has been prepared under the direction of Arcelor Mittal, Peiner Träger and Corus. The technical content was prepared by CTICM and SCI, collaborating as the Steel Alliance. This is available at https://constructalia.arcelormittal.com/en/news_center/articles/design_guides_steel_buildings_in_europe [41]. It comprises:

- Multi-storey steel buildings - Part 1: Architect’s guide;
- Multi-storey steel buildings - Part 2: Concept design;
- Multi-storey steel buildings - Part 3: Actions;
- Multi-storey steel buildings - Part 4: Detailed design;
- Multi-storey steel buildings - Part 5: Joint design;
- Multi-storey steel buildings - Part 6: Fire engineering;
- Multi-storey steel buildings - Part 7: Model construction specifications;
- Multi-storey steel buildings - Part 8: Description of member resistance calculator;
- Multi-storey steel buildings - Part 9: Description of simple connection resistance calculator;
- Multi-storey steel buildings - Part 10: Guidance to developers of software for the design of composite beams.

Other relevant design guides on multi-storey steel buildings can be found at https://steelconstruction.info/ [44]. These are:

- SCI P332: Steel in Multi-storey residential buildings. The Steel Construction Institute, 2004;
- SCI P362: Concise Eurocode for design of steel buildings. The Steel Construction Institute, 2009;
- SCI P365: Steel building design: Medium rise braced frames. The Steel Construction Institute, 2009;
- BSCA 35/03: Steel buildings. The British Construction Steel Association, 2003;
- SCI P300: Composite slabs and beams using steel decking: Good practice for design and construction, (3rd Edition), The Steel Construction Institute and The Metal Cladding & Roofing Association, 2023;
- SCI P355: Design of composite beams with large web openings. The Steel Construction Institute, 2011;
- SCI P354: Design of floors for vibrations- A new approach. The Steel Construction Institute, 2009;

- SCI P166: Design of steel framed buildings for service integration. The Steel Construction Institute, 1992;
- SCI P416: The design of cast-in plates. The Steel Construction Institute, 2017;
- SCI P101: Interfaces - Steel supported glazing systems. The Steel Construction Institute, 1997;
- Best Practice in Steel Construction: Commercial Buildings. The Steel Construction Institute, 2008;
- Target Zero - Guidance on the design and construction of sustainable, low carbon office buildings. Tata Steel and BCSA, 2012.
- Scheme Development: Coordination of structural and architectural design for multi-storey buildings with steel frames, SS001a-EN-EU. Access Steel;
- Scheme Development: Composite slabs for multi-storey buildings for commercial and residential use, SS010a-EN-EU. Access Steel;
- Scheme Development: Integrated beams for multi-storey buildings for commercial and residential use, SS013a-EN-EU. Access Steel.

This list is not exhaustive, and additional documents specific to certain countries or products may also be applicable.

3 CLASSIFICATION OF STEEL REUSE

3.1 Life cycle stages of constructional steel components

The life-cycle of a building or a product can be divided into several stages (called Modules) according to CEN/TC 350 standards for the LCA (Life-cycle assessment) and LCC (Life-cycle cost) assessment [45]-[47] (see Fig. 3.1):

A: Product and construction stage
    A_0: Pre-construction stage
    A_1: Raw materials supply
    A_2: Transport
    A_3: Manufacturing
    A_4: Transport
    A_5: Construction-installation process
B: Use stage
    B_1: Use
    B_2: Maintenance
    B_3: Repair
    B_4: Replacement
    B_5: Refurbishment
C: End-of-life stage
    C_1: Deconstruction, demolition
    C_2: Transport
    C_3: Waste processing
    C_4: Disposal
D: Reuse, recovery and recycling potential

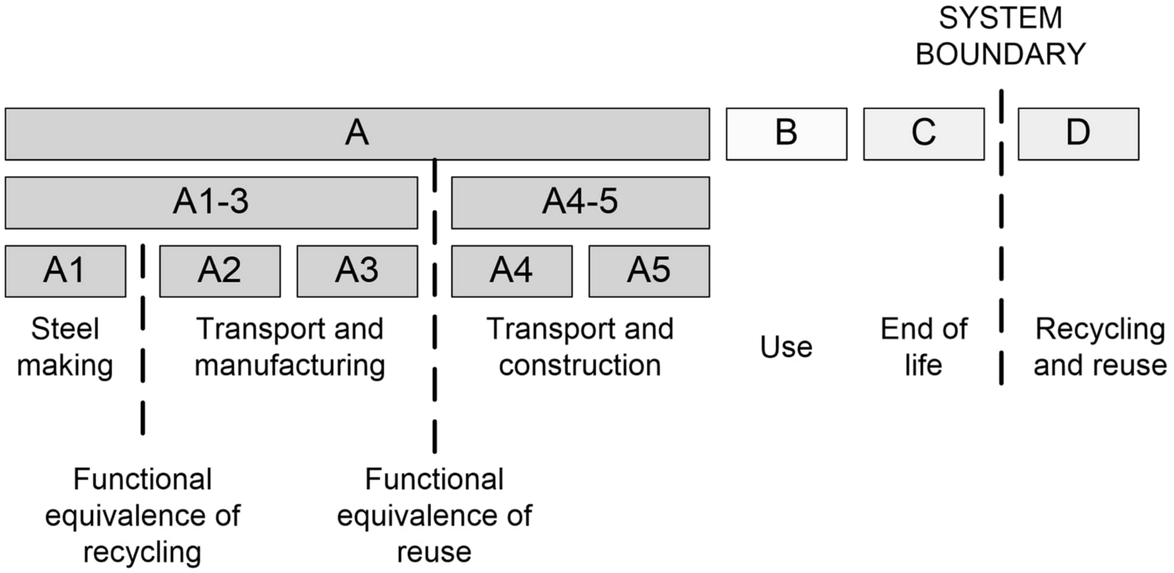


Fig. 3.1 Life-cycle stages of steel and steel-based components

3.2 Reuse scenarios

As illustrated in Fig. 3.2, several basic cases of components reuse can be recognised depending on the level of disassembly:

D_0: Reuse of the entire steelwork or its part (e.g., several bays) in-situ without disassembly,
D_I: Reuse of the disassembled steelwork (may include the envelope),
D_{II}: Reuse of the fabricated components (e.g. sandwich panels, columns),
D_{III}: Reuse of the constituent products (e.g. sections, plates).

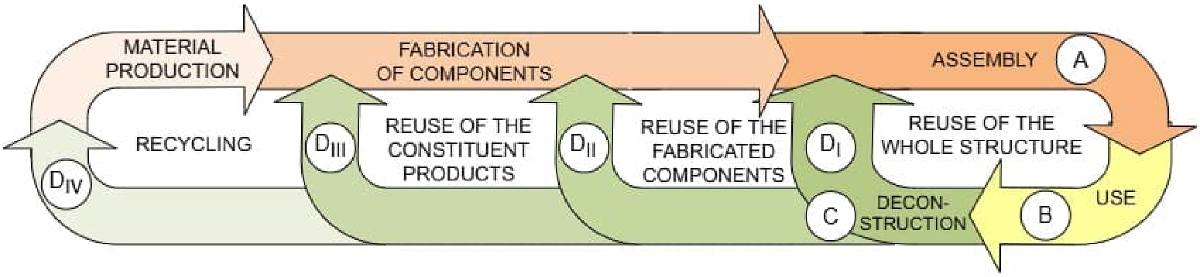


Fig. 3.2 Reuse scenarios in the value chain

From the deconstruction and transport point of view, several scenarios may exist:

D_A: In-situ reuse without disassembly (retrofitting),
D_B: Reuse on the same site in the same configuration,
D_C: Reuse on the same site in different configuration,
D_D: Reuse on a different site in the same configuration,
D_E: Reuse on a different site in different configuration.

This process is explained in Table 3.1 in terms of this classification.

Table 3.1 Classification of reuse cases

Case	**In-situ reuse**	**Relocated reuse**			
		Same site		**Different site**	
		Same configuration	Different configuration	Same configuration	Different configuration
Entire steelwork	$D_{0\text{-}A}$	-[a]	-	$D_{0\text{-}D}$[b]	-
Disassembled steelwork	-	$D_{I\text{-}B}$	$D_{I\text{-}C}$	$D_{I\text{-}D}$	$D_{I\text{-}E}$
Fabricated components	-	$D_{II\text{-}B}$	$D_{II\text{-}C}$	$D_{II\text{-}D}$	$D_{II\text{-}E}$
Constituent products	-	$D_{III\text{-}B}$	$D_{III\text{-}C}$	$D_{III\text{-}D}$	$D_{III\text{-}E}$

[a] This scenario is unlikely since if the structure was deconstructed, it is unlikely that it would be re-erected in the same configuration on the same site;
[b] Only the case of single-storey steel buildings.

In the case of *in-situ reuse* (D_A), the components are not disassembled and remain connected to the steelwork. This approach, commonly referred to as retrofitting, involves preserving the existing structure, which may be repaired, reinforced, or coated to avoid disassembly and replacement and potentially to accommodate different loading conditions. The structural resistance and serviceability must be verified in accordance with current codes. This specific case is not covered by these recommendations.

Relocated reuse (D_B to D_E) means that components are disconnected and reconditioned on the building site or in the workshop. In many cases, the building site is redeveloped, and it may be beneficial to consider integrating the steel components of the previous building in the new project (D_B and D_C).

Relocated reuse to a different site (D_D and D_E) can be organised using the material dealer (in the case of larger quantities of small components such as structural sections) or can be negotiated directly between participants in the deconstruction and new construction processes. In some small-scale structures, for example those in modular form, it is possible to relocate the building or its major components without disassembly (D_0) for a short distance by use of cranes or crawler vehicles.

Different options regarding transport need are shown in Table 3.1, where index “A” means that reuse takes place on the same building site and “B” means that components require transport (e.g. between sites, to the dealer, storage or workshop).

The cladding is a more complex component that can be reused. If it is a double skin trapezoidal system, attention should be paid to all the layers. Sandwich panels can be reused if the screw holes are hidden or reused in the second use. To preserve the protection by coatings is more of a challenge, especially if combined with longer-term deterioration, pollution and UV attack. For different layouts, the sandwich panels may be reused, but in combination with a new external layer.

4 HISTORICAL REVIEW OF CODES OF PRACTICE AND PRODUCT STANDARDS

Knowledge of the history of structural steel is important if the reuse of steel members and other steel components is to be widely adopted. During the 1970s, which is taken as the starting point for potential steel reuse within the scope of this document, detailed descriptions of the chemical composition, physical, and mechanical characteristics of steel members were required to meet country-specific standards. This is demonstrated in Fig. 4.1 for hot rolled structural steel members.

In 1961, the *Comité Européen de Normalisation* (European Committee for Standardisation, CEN) was founded by the national standards organisations in Europe to produce and implement common European standards. The adopted standards are implemented as national standards by each CEN member country and any conflicting national standards were withdrawn.

In this publication, the starting point is that the steel structure from which the steel members are to be salvaged was originally designed and specified based on the standards given in Fig. 4.1 or EN 1993. The design was based on the Limit State principles so that the probability of each limit state being reached is somewhat constant for all members in a structure and is also at an acceptable low level.

4.1 Hot rolled structural steels

4.1.1 Product standards

The designation of steel products in EN 1993 is in accordance with EN 10025-2:2019 [21]. Fig. 4.1 presents a list of corresponding former national designations and the former designations in EN 10025:1990 and EN 10025:1990+A1:1993, which were superseded by the 2004 edition and later by the 2019 edition. The material properties for structural steel are defined in Clause 5.2.5 of EN 1993-1-1, and these properties do not degrade over time, i.e., the elastic modulus of all steel grades is $E = 210000$ N/mm^2, Poisson's ratio $\nu = 0.3$ and the coefficient of linear thermal expansion $\alpha = 12\times10^{-6}$ per °C, at ambient temperatures.

Structural steel is specified by its yield strength (in N/mm^2 or MPa) and there should be a sufficient margin between the ultimate strength and the yield strength of the steel to allow for plasticity and redistribution of internal forces within a structure. Common steel grades are S235 (the default minimum value for design and development of design formulae in Eurocodes), S275 and S355 steel.

Modern structural steels contain small quantities of carbon, typically 0.17% for S235 and 0.24% for S355 (in sub-grades JR). Their higher strength is achieved through alloys, e.g. manganese, nickel, and niobium, which can affect other mechanical properties, e.g. ductility, toughness, and weldability. Ductility can be improved by reducing the sulphur levels, and toughness can be improved by the addition of nickel.

The chemical and mechanical properties are recorded in test certificates as part of the normal quality control procedures of the steel manufacturer and as presented in the specifications for manufacture of steel products. It should be recognised that the product specifications are a set of requirements to be met and are not a label for a particular type of steel.

			Austria	Belgium	Finland	France	Germany	Italy	The Netherlands	Norway	Portugal	Romania	Spain	Sweden	UK
EN 10025-2:2004	**Product Standards: Equivalent former designations corresponding to EN 10025-2**														
	EN 10025:1990 +A1:1993	EN 10025:1990	M 3116	NBN A 21-101	SFS 200	NF A 35-501	DIN 17100	UNI 7070	Euronorm 25-72	*DIN* 17100	NP 1729	STAS 500/2-76	UNE 36-080	SS *followed by number steel grade*	BS 4360
	S235JR	Fe 360 B		AE 235-B		E 24-2	St 37-2	Fe 360 B	Fe 310 0	St 37-2	Fe 360-B	OL37-1/1a/1b		13 11-00	
	S235JRG1	Fe 360 BFU	USt 360 B				USt 37-2		Fe 360 A	USt 37-2			AE 235 B-FU		
S235JR	S235JRG2	Fe 360 BFN	RSt 360 B		Fe 37 B		RSt 37-2		Fe 360 BFU	RSt 37-2		OL37-2	AE 235 B-FN	13 12-00	40 B
S235J0	S235J0	Fe 360 C	St 360 C	AE 235-C		E 24-3	St 37-3 U	Fe 360 C	Fe 360 BFN	St 37-3 U	Fe 360-C		AE 235 C		40 C
			St 360 CE						Fe 360 CFN			OL37-3k/3kf			
	S235J2G3	Fe 360 D1	St 360 D	AE 235-D	Fe 37 D	E24-4	St 37-3 N	Fe 360 D	Fe 360 DFN	St 37-3 N	Fe 360-D		AE 235 D		40 D
S235J2	S235J2G4	Fe 360 D2					—			—		OL37-4kf			
S275JR	S275JR	Fe 430 B	St 430 B	AE 255-B	Fe 44 B	E 28-2	St 44-2	Fe 430 B	Fe 430 A	St 44-2	Fe 430-B	OL44-2k	AE 275 B		43 B
S275J0	S275J0	Fe 430 C	St 430 C	AE 255-C		E 28-3	St 44-3 U	Fe 430 C	Fe 430 BFN	St 44-3 U	Fe 430-C		AE 275 C		43 C
			St 430 CE						Fe 430 CFN			OL44-3k/3kf			
	S275J2G3	Fe 430 D1	St 430 D	AE 255-D	Fe 44 D	E 28-4	St 44-3 N	Fe 430 D	Fe 430 DFN	St 44-3 N	Fe 430-D		AE 275 D		43 D
S275J2	S275J2G4	Fe 430 D2					—			—		OL44-4kf			
S355JR	S355JR	Fe 510 B		AE 355-B		E 36-2	—	Fe 510 B	Fe 510 BFN	—	Fe 510-B	OL52-2k	AE 355 B		50 B
S355J0	S355J0	Fe 510 C	St 510 C	AE 355-C	Fe 52 C	E 36-3	St 52-3 U	Fe 510 C	Fe 510 CFN	St 52-3 U	Fe 510-C		AE 355 C		50 C
	S355J2G3	Fe 510 D1	St 510 D	AE 355-D	Fe 52 D		St 52-3 N	Fe 510 D	Fe 510 DFN	St 52-3 N	Fe 510-D	OL52-3k/3kf	AE 355 D		50 D
S355J2	S355J2G4	Fe 510 D2					—		Fe 510 DDFN	—					
	S355K2G3	Fe 510 DD1		AE 355-DD		E 36-4	—			—	Fe 510-DD	OL52-4kf			50 DD
S355K2	S355K2G4	Fe 510 DD2					—			—					
EN 1993:2005	**Codes of Practice for the Design of Steel Structures: Equivalent former designations corresponding to EN 1993**														
	ENV 1993-1-1:1992		ÖNORM B 4300	NBN 212:1970 and NBN E 27-071:1987	Rakennus-määräys-kokoelma B7	Règles CM66	DIN 1880		NEN 6770, part 2 (1997-2012) NEN 6770, part 1 (1990-1997) NEN 3851 (1972-1990) N 1055 (1955-1972)	NS 3472: 1984 NS 3472: 2001	REAE, Decreto n.º 46160	STAS10108/0-78	NBE MV 10X and 11X series (before 1996) NBE EA-95 (after 1996)	BSK 99 Handbooks StBK-N*X*, *X* = 1, 2, 3, 4 ,5	BS 5950 (after 1985) BS 449 (before 1985)

Fig. 4.1 National products and structural design standards before 2004

The widely used steel grades, S235, S275 and S355, conform to common standards since the 1970s and possess comparable properties to the structural steels commonly used today. If a hot rolled product is labelled as conforming to another specification, the difference may be only in the type and amount of testing required by this other specification. Therefore, a closer examination will show whether the structural components meet the user's requirements.

4.1.2 Codes of practice and standards for design

The first European standard for the design of steel structures was issued in 1992, as a prestandard ENV 1993-1-1 [48]. It was intended to be a framework for the preparation of harmonised technical specifications for construction products in the various European countries. These design standards were used in conjunction with a National Application Document (NAD) valid in the country where the building was located. Later this ENV was converted into a European Norm (EN), EN 1993 or Eurocode 3, and the NADs became National Annexes (NA). From 2005-2010, Eurocodes have been widely applied in all European countries and have generally replaced all National Structural Design Standards (see Fig. 4.1). Eurocodes also provide a means of ensuring public safety throughout the EU.

Another important part of Eurocode 3 is the way in which it is integrated with product standards to allow CE marking to support the Construction Products Regulation (CPR) [49].

4.2 Cold formed structural steels

In Europe, the ECCS Committee TC7 originally produced the European Recommendations for the design of light gauge steel members in 1987 [51], followed by ENV 1993-1-3:1996 [52]. This recommendation was further developed and published in 2006 as the European Standard Eurocode 3: *Design of steel structures. Part 1-3: General Rules. Supplementary rules for cold formed thin gauge members and sheeting* [16]. EN 1993-1-3:2006 represents the unified European Code for cold formed steel design and contains specific provisions for structural applications using cold formed steel products made from coated or uncoated thin gauge hot or cold rolled sheet and strip. It is intended to be used for the design of buildings or civil engineering works in conjunction with EN 1993-1-1 and EN 1993-1-5. EN 1993-1-3 permits only the design using the limit states method (LSD).

EN1993-1-3 includes in Chapter 10 design criteria for the following applications:

- Beams restrained by sheeting;
- Linear trays restrained by sheeting;
- Stressed skin design;
- Perforated sheeting.

Cold forming technology enables the production of tailored sectional configurations. However, analysis and structural design of unusual members may be complex. Structural systems formed by different cold formed sections connected to each other (e.g., purlins and sheeting) can also lead to complex design situations, not entirely covered by design codes. Numerical FEM analysis is always an alternative for design, but for many practical situations, modelling can be complicated. For such complex design problems, modern design codes allow the use of testing procedures to evaluate structural performance. Testing can be used to replace the calculation-based design or to be used in combination with calculations. Only accredited laboratories can perform such tests and produce the relevant certification.

5 EVALUATION OF STRUCTURAL REUSABILITY

The reusability of most of the structural components, assemblies or constituent products can be estimated already during the pre-deconstruction audit. This will greatly affect planning of the deconstruction including tools and techniques to be used on site. General rules for the auditing are presented in the new EU construction & demolition waste management protocol including guidelines for pre-demolition and pre-renovation audits of construction works [50] and more specific guidance for the constructional steelwork is provided by the PROGRESS project in Pre-deconstruction audit protocol (Chapter 3 of [42]).

5.1 Parameters influencing reusability

An important aspect when evaluating structural steel for reuse is that it should be free of deterioration when salvaged from its previous use. Therefore, the structural members should not have significant imperfections or permanent deformations, nor have local damages, inclusive plastic deformations, and reduced cross sections (e.g., through holes, openings, cracks, or excessive corrosion) and should not have been subjected to extreme events such as impact, or fatigue, or be damaged by fire.

Deterioration is the reduction in material characteristics and/or size due to its exposure conditions. For example, a steel member may suffer from corrosion under adverse exposure conditions, which reduces its geometric properties. A defect is a reduction in structural capacity in cases where loads have exceeded the structural capacity, or because of local impact, drilling, or welding on the structural properties. Damage is the result of extreme loads which could not reasonably be foreseen or designed for, e.g. extreme seismic loading, impact (e.g. from a vehicle), blast or explosion.

Steel does not undergo major changes due to ageing, except for surface rusting and the possible effect of inelastic deformations. Corrosion can be prevented by an appropriate form of protection that includes the preparation and application of surface paint systems or metallic coatings by means of thermal spraying or galvanising.

The ageing of the material is the gradual deterioration (because of time or use) mainly of mechanical and physical properties. There are two basic types of ageing: thermal ageing embrittlement, and strain ageing. Thermal ageing embrittlement represents a process of change in material properties due to the disintegration of oversaturated solid ferrite solution over a long period of time without any external mechanical load. This can occur especially in low carbon steel, namely up to 0.2% Carbon, and gradually leads to decreased ductility, notch toughness and fracture toughness of the material, an increased transition temperature, and an increase in the lower and upper limit of notch toughness. Strain ageing refers to a process consisting of changes in material properties after and/or during plastic deformation. There are two types of deformation ageing: static strain ageing, e.g. foundation settlement, where material properties change after elements suffer plastic deformations, and dynamic strain ageing, e.g. after large-scale seismic events, when material properties change rapidly during high deformation. Strain ageing affects the mechanical characteristics in the sense that the yield strength measured after ageing is often higher, but the ductility in the fracture decreases. The two phenomena are frequently considered in combination and so the term *ageing* is often used interchangeably.

Fatigue is defined as a process of cycle-by-cycle accumulation of damage in a material that undergoes fluctuating stresses and strains. A significant feature of fatigue is that the load is not high enough to cause immediate failure. Instead, failure occurs after a certain number of load fluctuations, i.e. after the accumulated damage has reached a critical level. For example, crane runaway girders are fatigue-prone structures.

In some circumstances, steel may creep due to slow plastic deformations at high temperature, typically during a fire. The creep strength in the range of temperatures where the creep applies is always lower than the material yield strength.

5.2 General approach

The overall process from reclamation to reuse of steel components is summarised in the flowchart in Fig. 5.1.

The scope is limited to buildings constructed with elements produced after 1970 so that the materials are generally consistent with modern product specifications and with limit state design methods considered in the current standards.

If a building becomes available for salvage of the primary steel structure, and possibly its secondary components and cladding, a pre-deconstruction audit should be carried out before the building is demounted. This will enable identification of building components that can be reused. Pre-deconstruction audits are described in Part 3 of this manual.

From this initial inspection of the building, a recommendation is made on whether the steel components can be reused or if demolition is the more suitable option. If steel products can be salvaged, it is important to define the expected reuse scenario. In the case of relocated reuse, a decision may be made on the potential reuse of the entire structure, or its individual elements. Guidance on the evaluation of the reusability of reclaimed elements is given in Section 5.1. The materials should then be sampled and, if necessary, tested according to the protocol in Appendix A. The structural reusability of existing elements is then evaluated according to the test results.

If reuse is a viable option based on the dimensions, quality and quantity of the reclaimed members, the building structure may then be demounted and all elements labelled and batched. Components often have to be cleaned to remove coatings and accumulated dirt or subjected to other reconditioning processes. Finally, the structural design and verification of the reclaimed steel members and other components is carried out for the chosen reuse scenario (see Section 6).

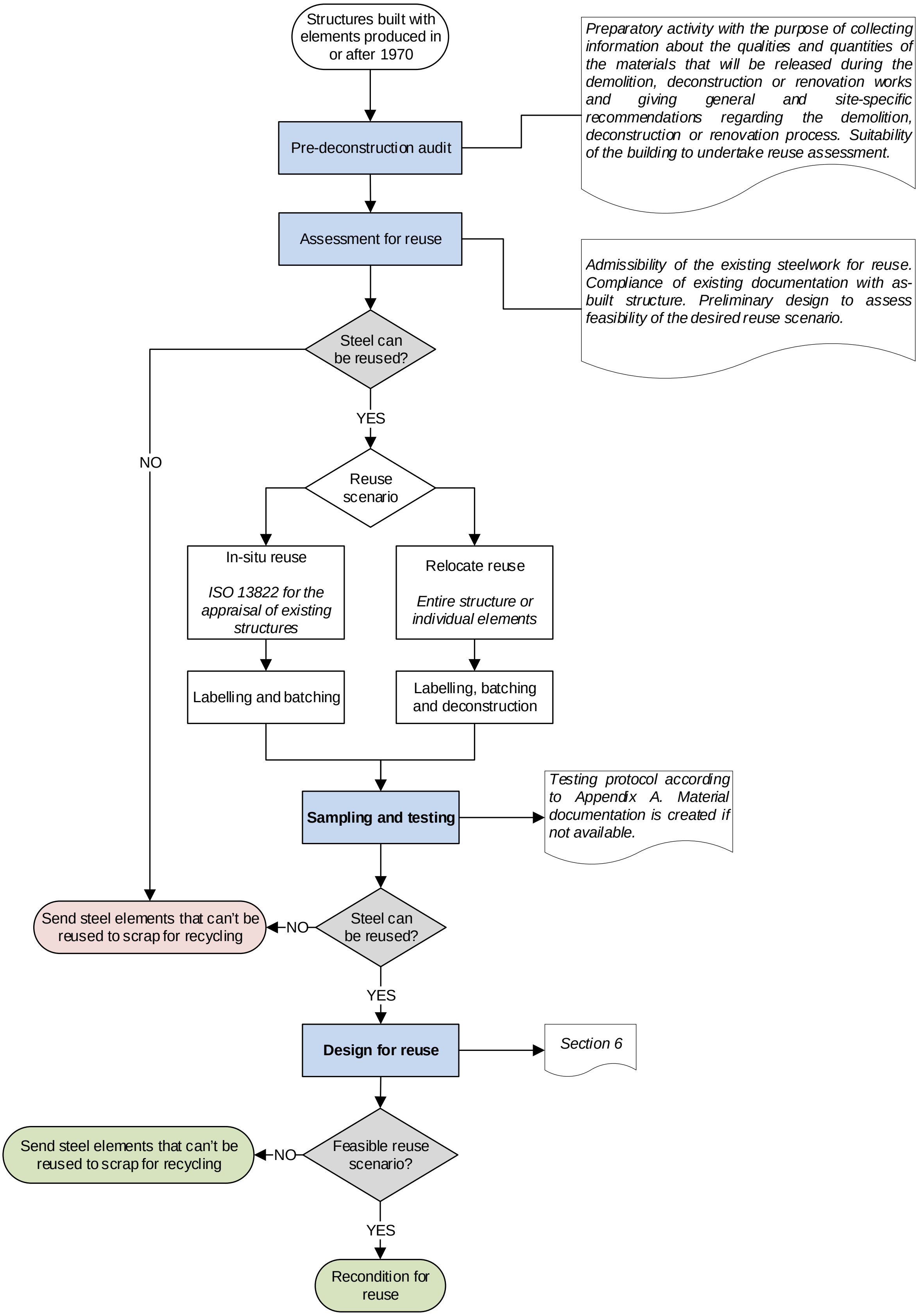


Fig. 5.1 Overall process: from reclamation to reuse and design of steel products

5.3 Design procedure

The design of structures made from reclaimed steel members follows the same principles of limit state design and verification using the partial factor method, as for “new structures”. There are, however, some additional rules and provisions, which are given in this part of the document. Specific provisions to check structural integrity are derived from the principles of structural reliability [9].

Fig. 5.2 gives an overview of the Eurocode-based design philosophy.

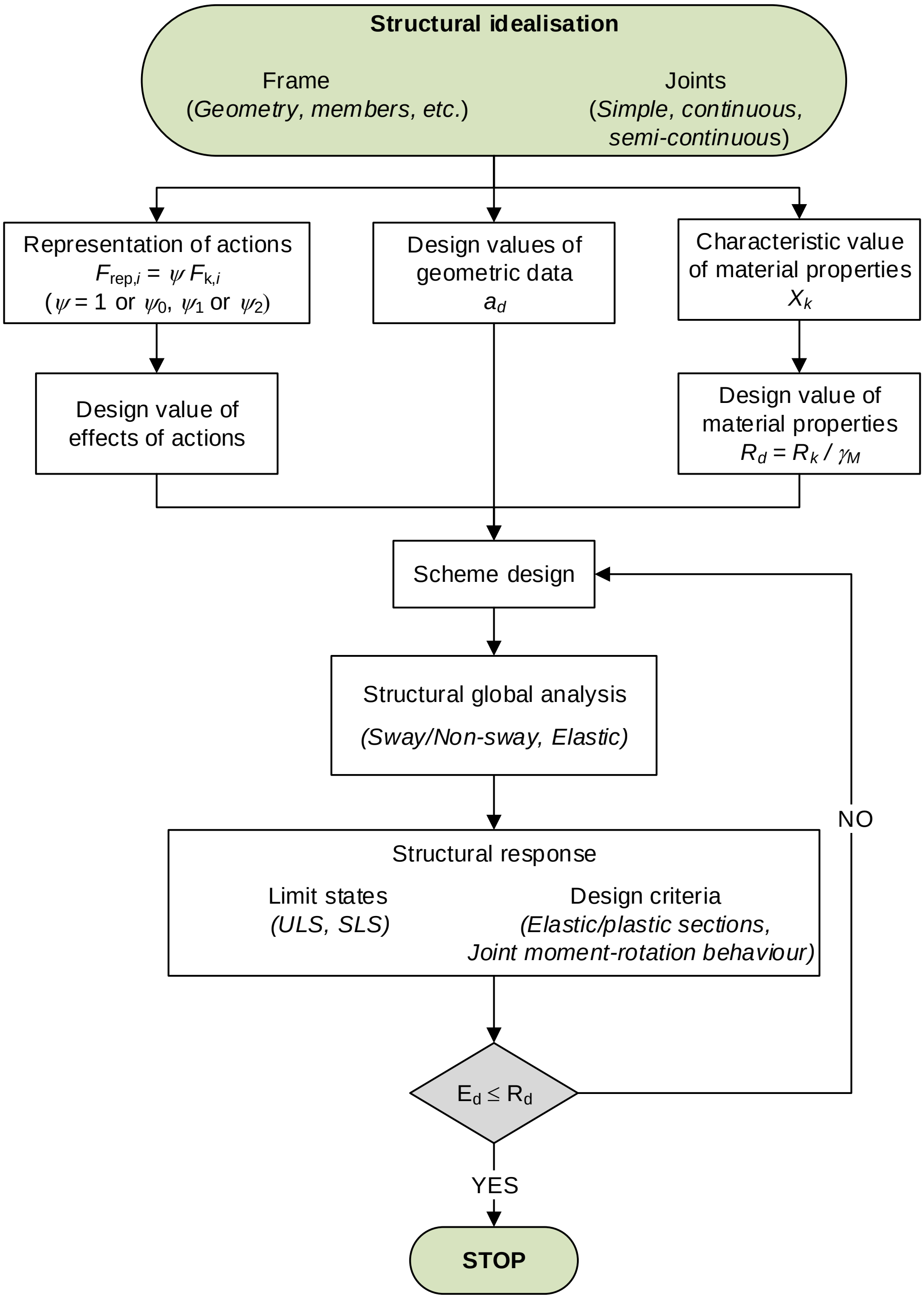


Fig. 5.2 Procedural overview of frame design

The approach to design comprises the following steps depending on the type of structure and the contractual role of the designer (on behalf of the client or the main contractor):

- The designer chooses a viable structural framing scheme based on the spatial requirements of the building;
- The design values of the effects of actions, based on the relevant standards, and the geometry of the building are obtained;
- The design values of the material strengths are determined for the structural steel members, see Section 5.4.1;
- The conceptual design of the structure is completed based on this input information and various options are presented to the client;
- At this stage, it will become apparent whether the possible use of reclaimed steel components is both practical and viable;
- The final design of the structure is then performed taking into account client feedback to the proposed conceptual design;
- The designer decides the type of structural analysis to be adopted. The recommendation for design using reclaimed steelwork is to adopt an elastic global analysis;
- Limit state verifications are carried out to determine the structural response. These consist of checking the Serviceability Limit States (SLS), i.e., deflections of the frame and of the members under service loading conditions, and the Ultimate Limit States (ULS), i.e., resistance of members, as well as member and frame stability;

When the structure does not meet the new design requirements, the structural system will have to be modified or strengthened accordingly.

5.4 Structural steel for reuse

Reclaimed structural steel may be used in structural design according to the provisions of EN 1993. For consistency with this design standard, the material should comply with specific performance and quality requirements, which are described below.

The definition of structural steel in this context is the steel from members made of hot rolled sections and their end connections, including trusses made from rolled members. Fabricated (welded) sections, such as plate girders, may be included in this definition, but they are often designed for specific loading conditions in terms of the weld size, web stiffeners, etc. and so they require verifications for the specific reuse scenario. Secondary members made from cold formed steel sections can also be reused, without modifications or at cut-down lengths, although they are not normally considered under the definition of structural steel.

CEN/TS 1090-201:2024 [2] provides complementary provisions to EN 1090-2 for the assessment and testing of reclaimed structural components for the execution of steel structures up to EXC3 (see EN 1090-2). The CEN/TS 1090-201:2024 provisions apply to products used in structures to be designed (see EN 1993-1-1) for quasi-static loading and not subject to fatigue loading. The document provides requirements for the evaluation of the reusability of reclaimed structural components and constituent products. It also includes a general reusability assessment and a quality assessment for plates, hot rolled profiles and hot finished or cold formed hollow sections in carbon steel used as constituent products.

The quality assessment includes the determination and declaration of mechanical and geometrical properties, as well as weldability. These properties are those listed as required to be specified for non-standard products according to Clause 5.1 of EN 1090-2. The dimensions and tolerances of all reclaimed products have to be measured and documented. The component can be declared with reference to a standard profile, if all geometrical requirements are fulfilled. The mechanical properties can be investigated by non-destructive hardness testing and finally the relevant properties are obtained by one or a few destructive tests and the results are assigned to all members of the test unit. In the end, steel can be declared according to the grades defined in product standards.

The document does not apply to cold formed structural steel sections and sheeting as described in EN 1090-4, or to mechanical fasteners.

Document [53] outlines the most important issues in considering the design of new structures with reclaimed structural steel according to EN 1993, including scope, proposal, and action plans. The scope comprises the design of reclaimed structural steel components for reuse (structural elements only). It also proposes 8 steps of the process downstream, and detailed descriptions of the respective Process Chain Steps (PC Steps) are given.

5.4.1 Classification of reclaimed steelwork

In the frame of the RFCS PROGRESS project [54], reclaimed steel was classified according to verification of its material performance requirements (adequacy assessment) and quality assurance requirements (reliability assessment) in the following classes:

- **Class A**: steel materials that meet performance requirements and with approved quality assurance from original certificates,
- **Class B**: steel materials that meet all performance requirements through comprehensive material testing (see Appendix A) and with approved quality assurance, i.e. certificates of compliance to the relevant European Product Standards, by testing,
- **Class C**: steel materials classified as the most conservative grade according to the age and location of the structure (unidentified steel).

The adequacy assessment is intended to justify the necessary/required material characteristics according to the material/product standard or according to Clause 5.1 of EN 1090-2, while the reliability assessment is intended to justify that the reliability requirement for the design procedures according to the Eurocodes are met.

CEN/TS 1090-201:2024 [2] proposes four testing protocols, A to D. The required destructive testing is adjusted based on the information gathered during prospecting. Recommendations for the choice of a specific protocol are given below and are illustrated in Fig. 5.3. The provenance of the steel should be considered as known when at least the geographical location, building year and former function of the components are known. Two types of structural steel are included, i.e., type 1 and type 2.

Type 1 structural steel may be expected to have mechanical properties and weldability similar to steel grades according to the European standards listed in Clause 5.3 of EN 1090-2:2018+A1:2024. The variability of their mechanical properties may be assumed to be in accordance with Annex E of EN 1993-1-1. Products reclaimed from structures built with elements produced in, or after, 1970 are seen as made of “modern” steel, so called Type 1 steel.

For Type 2 structural steel, the variability of the properties cannot be reliably assumed, and more tests should be undertaken as well as a statistical analysis of the results, according to protocol C in 5.3.4.5 of CEN/TS 1090-201:2024.

Structural components of unknown provenance shall not be grouped in test units and comprehensive testing is required, according to protocol D in 5.3.4.6 of CEN/TS 1090-201:2024.

CEN/TS 1090-201:2024 introduces a more detailed technical framework for the verification of reused structural steel components. Unlike the A/B/C classification system presented in the RFCS PROGRESS project [54], the CEN/TS 1090-201:2024 defines four testing protocols (A–D), selected based on the provenance of the steel (known vs. unknown) and its classification as either Type 1 (post-1970) or Type 2 (pre-1970). These protocols specify requirements for testing, inspection, and declaration of mechanical properties in accordance with EN 1090-2 and Eurocode 3. In this guide, the testing protocols defined in CEN/TS 1090-201:2024 will be adopted.

5.4.2 Material performance requirements

EN 1090-2 for the execution of steel structures (i.e. fabrication and erection) allows steel and sections not covered by European standards for constituent products to be specified. The following mechanical properties have to be determined according to Clause 5.1 of EN 1090-2 [8]:

- Strength, i.e. yield strength, f_y, and tensile strength, f_u,
- Elongation after fracture, ε_f, that gives information on how much the material deforms,
- Heat treatment delivery condition.

The steel grade should be in the range of S235 to S700. The ductility requirements for the design to EN 1993-1-1 are presented in Table 5.1 (recommended values that may be modified by the NAs).

Table 5.1 Ductility requirements (CEN recommended values)

	Yield ratio f_u/f_y	**Elongation at failure**
For plastic global analysis	≥ 1.10	≥ 15%
For elastic global analysis	≥ 1.05	≥ 12%

EN 1090-2 also states that the characterisation of the following properties may be required, although not mandatory:

- Stress reduction of area,
- Impact strength or toughness,
- Through-thickness requirements (Z-quality),
- Limits on internal discontinuities or cracks in zones to be welded.

Where welding of the structure made from reclaimed steel is anticipated, the chemical composition must be determined for use in preparing the welding procedure specification. There are very simple non-destructive testing techniques to determine the steel composition, such as the positive metal identification technique (see Appendix A.) The full characterisation of the chemical composition is also required, and the reclaimed material needs to be characterised, due to absence of original certificates. The steel weldability shall be declared as follows [8]:

- Classification according to the material grouping system defined in CEN ISO/TR 15608 [55], or
- A maximum limit for the carbon equivalent, or,
- A declaration of its chemical composition in sufficient detail for its carbon equivalent to be calculated.

5.4.3 Quality assurance requirements

Reclaimed steel has to meet certain quality and safety requirements in order to be characterised to ensure its capacity to be used in structural design based on EN 1993. The main question to be answered is "*To which specific product standard was the material manufactured to*?", to check the conformity, quality and traceability of the product.

Material traceability is the ability to trace back the source of a specific steel material to its original identity as delivered from the mill, through a proper identification and quality assurance system. Suppliers and fabricators who intend to reclaim structural steel materials have to establish an in-house quality assurance system to ensure the traceability of these materials. Each steel member shall be marked with a unique identification number for which quality control checks are introduced and recorded. Such a unique identification will facilitate future reference to the factory production control certificate, manufacturer test certificate, inspection record and/or test report without confusion.

If mill certificates are available, it is possible to trace back the reclaimed steel components to verify that they meet the relevant material specifications and requirements.

New steel materials are sourced with a valid declaration of performance (DoP) and a manufacturer test certificate based on the delivery specification. In contrast, the reuse of the material is permitted with satisfactory verification against its reusability.

5.4.4 Steel properties to be declared for hot rolled steel reclaimed elements

This section summarises the steel properties that need to be assessed for reclaimed hot rolled steel elements according to clause 5.1 of EN1090-2 (including hollow sections – Table 5.2). Further commentary on these properties is also provided.

Strength

The yield and ultimate strengths should be determined and evaluated according to Appendix A. The declared yield and ultimate strengths for the reclaimed structural steel to be used for the design shall be defined according to a steel grade specified by the reference product standard (say S275) that ensure the reliability requirements (see Appendix A).

Elongation

The use of reclaimed steelwork is limited to applications where significant ductility is not required (i.e. elastic global analysis, no use in primary seismic system; DCL design). However, elongation must be assessed according to Clause 5.1 of EN 1090-2, which needs to be determined by destructive tensile testing. Based on historical data, there is no concern that structural steel reclaimed from buildings erected after 1970 will not meet the design requirements according to Eurocode 3 (see Table 5.1) - [56] to [58]. The minimum elongation requirement for reclaimed steel is taken from Table 5.1 and not from the reference product standard.

Table 5.2 Material properties to be declared according to Clause 5.1of EN 1090-2

Property	**To be declared**	**Procedure**
Strength (yield and tensile)	Yes	Determined by destructive and non-destructive tests.
Elongation	Yes	Determined by destructive tests.
Stress reduction of area requirements (STRA)	If required	Generally, not required to be declared.
Tolerances on dimensions and shape	Yes	Based on dimensional survey.
Impact strength or toughness	If required	If required, determined by destructive tests. Conservative assumption as the default.
Heat treatment delivery condition	Yes	Conservative assumption as the default.
Through thickness requirements (Z-quality)	If required	Generally, not required to be declared.
Limits on internal discontinuities or cracks in zones to be welded	If required	Generally, not required to be declared.
In addition, if the steel is to be welded, its weldability shall be declared as follows:		
Property	**To be declared**	**Procedure**
Classification in accordance with the materials grouping system defined in CEN ISO/TR 15608, or	-	Not applicable for reclaimed steelwork.
A maximum limit for the carbon equivalent of the steel, or;	Yes	Maximum to be declared from manufacturer’s test certificates.
A declaration of its chemical composition in sufficient detail for its carbon equivalent to be calculated	Yes	Determined by non-destructive and destructive tests.

Tolerances on dimensions and shape

Reclaimed elements can be checked against geometric tolerances according to the relevant product standard (see Table 5.4). Elements within allowable tolerance are acceptable and satisfy the assumptions made in the design standard. However, there is no limitation to use reclaimed steelwork with bespoke dimensions, i.e., members for which tolerances from Table 5.4 are not met, as long as the design considers measured section properties rather than tabulated standard section sizes. Member bow and cross section imperfections (as out-of-squareness or out-of-flatness) imperfections still need to comply with the requirements from EN 1090-2.

Through thickness requirements

Through thickness properties are generally not required for reclaimed sections, such as beams or columns. Some joint details/components may require the steel plate to have specific through thickness properties [62]. If through thickness properties are required, the reclaimed plate must be tested as specified in EN 1993-1-10 [18].

Impact strength or toughness

Impact strength or toughness (commonly known as the Charpy value) might be required for a specific project, such as for thick, highly stressed steelwork, especially when exposed to low temperatures. For internal steelwork which is not subjected to fatigue, a conservative assumption about the material toughness can be adopted, meaning that a minimum Charpy V-notch impact value of 27J at 20°C can be assumed if no tests are performed (JR subgrade) - [56] to [58]. If material toughness must be determined, destructive tests are required in accordance with the relevant standard (see Appendix A).

Heat treatment delivery condition

Heat treatment delivery conditions have an impact on, for example, the grain size, residual stresses, etc. For the scope of the current document, this condition will have implications for reclaimed hollow sections. Hollow sections for structural applications are cold formed according to EN 10219 or hot finished according to EN 10210. The heat treatment delivery condition will influence the level of residual stresses in the hollow section, which in turn will have implications on the buckling design of the member. As measuring such property is not economically feasible, it is recommended that all reclaimed hollow sections are assumed to be cold formed according to EN 10219.

Declaration of chemical composition

The chemical composition is necessary to establish the durability and weldability of reclaimed structural steel. A declaration of chemical composition based on tests is necessary (see Appendix A). The chemical composition must measure certain chemical elements according to the relevant reference produce standard (EN 10025-2/3/4, clause 7.2 or EN 10219-1, clause 6.6), from which the carbon equivalent value (CEV) can be calculated.

5.4.5 Assessment of material properties

Generally, reclaimed steel is assessed for adequacy and reliability, which are closely interrelated. Steel usually has to be assessed, i.e., actual material properties have to be evaluated against the material performance requirements. In the absence of such certificates, material testing must be performed using appropriate sampling and testing methods, to demonstrate the adequacy of the material. The reliability assessment is to ensure that the steel products are manufactured under a stringent quality assurance system and that they meet the quality assurance requirements.

Materials can be classified upon completion of these assessments, which are obviously interrelated, according to the system proposed above and in accordance with the flow chart shown in Fig. 5.3. The classification is necessary to establish whether the reclaimed steel material can be allowed for structural use according to EN 1993 with or without any restrictions.

The four protocols [2], A to D, are defined for the assessment of material properties:

- **Protocol A**: Documentation available (Type 1 structural steel with original inspection documents);
- **Protocol B:** Single sample DT (Type 1 structural steel with known provenance);
- **Protocol C:** Statistically representative DT (Type 2 structural steel with known provenance);
- **Protocol D:** Comprehensive DT (Structural steel with unknown provenance).

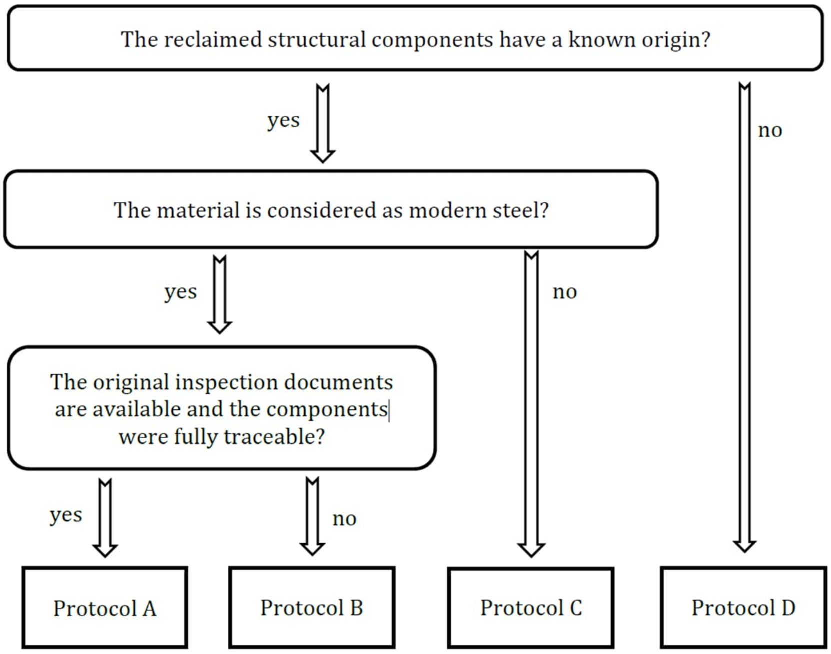


Fig. 5.3 Flowchart for the choice of testing protocol [2]

Protocol A: Documentation available (Type 1 structural steel with original inspection documents) covers the case of products that are traceable and for which the original documentation is available. According to Protocol A, steel material can be designed according to EN 1993, as the appropriate assessment of adequacy and reliability is justified by existing documentation. Examples of steelwork classified as Class A include steelwork reclaimed from a cancelled project (never erected) or steelwork reclaimed from different sources, for which documentation is available. An optional minimal testing procedure for Class A steel can be used to confirm the grade of the reclaimed steel.

Protocol B: Single sample DT (Type 1 structural steel with known provenance). An important distinction is made regarding the age of the material that is assessed.

Components that were produced more recently are indeed likely to have properties, including their variability, matching those assumed in design with current steel grades. These can be assessed based on a single test (Protocol B). When protocol B is applied, the properties assigned to all members of a valid test unit can be declared as a reference to a steel grade. The requirements on yield and ultimate strength above represent the 95% fractiles based on the means and variances given in Annex E of EN 1993-1-1.

Alternatively, the yield and ultimate tensile strength may be declared by referring to an equivalent steel grade for which the measured yield and ultimate tensile strength satisfy the criteria in Table 5.3.

Table 5.3 Minimum yield and ultimate tensile strength in single sample testing for equivalent steel grades

Equivalent steel grade	$R_{eh} \geq$ (MPa)	$R_m \geq$ (MPa)
S235	267	397
S275	313	452
S355	391	505
S420	463	559
S460	490	560

The minimum yield and ultimate tensile strength in Table 5.3 are the 5% fractiles, assuming material properties according to EN 1993-1-1:2022, Annex E.

Protocol C: Statistically representative DT (Type 2 structural steel with known provenance). When protocol C is applied, the characteristic properties assigned to all members of a valid test unit may be based on a statistical analysis in accordance with EN 1990.

For Protocols B and C, the material is shown to comply with the performance requirements through comprehensive material testing (see Appendix A). The testing procedure comprises a combination of non-destructive and destructive tests, e.g., to EN ISO 6892-1, together with inspection of geometric tolerances.

Protocol D: Comprehensive DT (structural steel with unknown provenance). If the origin of the reclaimed products is unknown, it is not possible to gather several components in a test unit and assign them common properties. In this case, protocol D requires that the components are tested individually with destructive methods. The test results can then be used directly, as characteristic values, or compared with the nominal values in a relevant product standard. When the provenance of the reclaimed products is known, components can be sorted in test units and non-destructive hardness testing is performed on all members of a test unit to safely select the members for destructive testing.

As an alternative, if the reclaimed steel material remains unidentified steel, free of defects, it may be permitted to be used for non-safety critical structures, e.g. agricultural buildings. In this situation, it should be assumed that the steel is of the weakest grade of structural steel used at the time of its first use. Relevant material product standards and design codes based on the structure erection date can be used.

After the adequacy assessment of the reclaimed steelwork is performed, a reliability assessment is required to ensure that the reclaimed product can be used in structural design according to EN 1993. The basic requirement of this assessment is essentially related to the fact that EN 1993 relies on known statistical distribution of the yield and tensile strengths to specify the partial factors for cross sections and member resistances which meet the reliability requirements according to EN 1990. To undertake such an assessment, the test results must meet certain minimum values for the yield and tensile strengths.

Appendix A provides an assessment and testing protocol which includes the definition of groups of elements for testing (test units), frequency of testing, types of testing procedures to be used to undertake the adequacy assessment and a procedure to achieve adequate reliability requirements (reliability assessment).

5.4.6 Assessment of reclaimed steelwork execution and certification

There will be no difference in the fabrication processes, procedures, standards, or tolerances for either new steel or reclaimed steel. It is therefore appropriate that refabricated, reclaimed structural steelwork can be CE Marked in accordance with EN 1090-1.

In addition to careful control of the fabrication process, material properties must be declared according to Clause 5.1 of EN 1090-2 if no material certificates/documentation are available. When using reclaimed steel, declaring such properties according to Clause 5.1 of EN 1090-2 may be a stockholder's responsibility. Stockholders who wish to trade reclaimed elements back to the construction industry are responsible for providing material documentation as expected from the manufacturers of “new” steel.

The previous statement is related to plain reclaimed elements without any welding procedures. If reclaimed steel elements have welded parts, the welding procedures must be inspected and tested to ensure that they meet the fabrication requirements of EN 1090-2.

5.5 Constituent products

5.5.1 Relevant properties

A constituent product in the reuse context represents an individual element extracted from an existing structure selected for disassembly and then reused as a new product for the fabrication and construction of another structure. This may include hot rolled and cold formed steel profiles.

Steel sections, plates and bars used as members must have dimensions and tolerances that comply with the standards of Table 5.4 and structural hollow sections with those listed in Table 5.5.

Table 5.4 Rolled steel sections, plates or bar: material and dimension standards

Form	**Dimensions**	**Tolerances**	**Material quality**	
			Non-alloy steels	**Weathering steels**
I and H sections	EN 10365	EN 10034	EN 10025-2[(a)] EN 10025-3 EN 10025-4	EN 10025-5[(b)]
Hot rolled taper flange I sections	EN 10365	EN 10024		
Channels	EN 10365	EN 10279		
Rolled asymmetric beams	*See manufacturers' information.*			

Angles	EN 10056-1	EN 10056-2		
Rolled Tees	EN 10055	EN 10055		
Fabricated sections and member bow imperfections	-	EN 1090-2		
Plates (reversing mill) [(c)]	-	EN 10029		
Plates (cut from coil) [(c)]	-	EN 10051		
[(a)] Steel grades S235, S275, S355 and S450. The steel grades S235 and S275 may be supplied in qualities JR, J0 and J2. The steel grade S355 may be supplied in qualities JR, J0, J2 and K2. The steel grade S450 is supplied in quality J0. [(b)] Steel grades S235 and S355. The steel grade S235 may be supplied in qualities J0W and J2W. The steel grade S355 may be supplied in qualities J0W, J0WP, J2W, J2WP and K2W. [(c)] The scope of EN 10029 covers plates of 3 mm up to 250 mm rolled in a reversing mill process, whereas EN 10051 covers plates up to 25 mm de-coiled continuously hot rolled uncoated flat products.				

Table 5.5 Structural hollow sections: material and dimension standards

Form[(a)]	**Dimensions and tolerances**	**Material quality**
Hollow sections (hot finished)	EN 10210-2	EN 10210-1
Hollow sections (cold formed)	EN 10219-2	EN 10219-1
[(a)] Hollow sections for use in constructional steelwork (both hot finished and cold formed) are supplied in steel grade S235 in quality JRH, steel grade S275 in qualities J0H and J2H, and S355 in qualities J0H, J2H, and K2H. Note: Selection of either EN 10210 or EN 10219 specifies whether structural hollow sections are to be hot finished or cold formed. Hot finished structural hollow sections to EN 10210 cannot be directly replaced with cold formed structural hollow sections to EN 10219 as the properties do not correspond directly.		

Besides geometric properties, the material characteristics are of high importance for the performance of the members. The steel grade can be obtained by using the testing protocols given in 5.4.5. These protocols may also be used to define the fracture toughness of the steel necessary to fulfill the requirements of EN 1993-1-10 [17].

EN 1993-1-10 gives requirements for the selection of steel qualities to avoid brittle fracture by specifying toughness properties and to avoid lamellar tearing in welded elements by specifying through-thickness properties.

Alternatively, Table 5.6 provides limited plate thicknesses for the UK practice assuming that steelwork is welded with “moderate” and “very severe” details according to references [59] and [60] for a stress level equal or more than 0.5 × fy(t).

Table 5.6 Maximum thickness (mm) for each steel grade and designation (UK)

Welding detail	**Steelwork**	**S235**			**S275**			**S355**		
		JR	**J0**	**J2**	**JR**	**J0**	**J2**	**JR**	**J0**	**J2**
Moderate	Internal	45	82.5	115	40	70	102.5	22.5	45	67.5
	External	27.5	67.5	97.5	22.5	60	85	12.5	37.5	55
Very severe	Internal	27.5	45	67.5	22.5	40	60	12.5	22.5	37.5
	External	12.5	37.5	55	10	32.5	50	5	17.5	30

Since the scope of the current guide is limited to the reuse of reclaimed steel, for structures where fatigue is not a design consideration, the limiting thickness values proposed by SCI P419 [61] can be used, see Table 5.7. The background document to EN 1993-1-10 [18] confirms that the limiting thicknesses may be extremely safe-sided if used for non-fatigue structures.

SCI P419 adopts the same procedures as Eurocode, based on the fracture mechanics approach, but reduces the calculated crack growth for applications where fatigue is not a design consideration. Table 5.7 follows the same format as Table 2.1 of EN 1993-1-10. The values in Table 5.7 can be used in countries other than the UK, when fatigue is not a design consideration, subject to any requirements of the specific National Annex of the country of construction.

Table 5.7 Limiting thickness values when fatigue is not a design consideration [61]

Steel grade	Sub Grade	Charpy energy CVN		Reference temperature, T_{Ed} (°C)																				
				10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
		at T (°C)	J_{min}	$\sigma_{Ed} = 0.75\, f_y(t)$							$\sigma_{Ed} = 0.5\, f_y(t)$							$\sigma_{Ed} = 0.25\, f_y(t)$						
S235	JR	20	27	200	200	200	195	125	87	63	200	200	200	200	200	200	161	200	200	200	200	200	200	200
	J0	0	27	200	200	200	200	200	195	125	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	J2	-20	27	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200
S275	JR	20	27	200	200	200	133	91	64	47	200	200	200	200	200	170	121	200	200	200	200	200	200	200
	J0	0	27	200	200	200	200	200	133	91	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	J2	-20	27	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	M,N	-20	40	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	ML, NL	-50	27	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200
S355	JR	20	27	200	177	114	77	54	40	30	200	200	200	200	147	104	76	200	200	200	200	200	200	200
	J0	0	27	200	200	200	177	114	77	54	200	200	200	200	200	200	147	200	200	200	200	200	200	200
	J2	-20	27	200	200	200	200	200	177	114	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	K2,M, N	-20	40	200	200	200	200	200	200	177	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	ML, NL	-50	27	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200
S460	Q	-20	30	200	200	200	200	147	96	65	200	200	200	200	200	200	187	200	200	200	200	200	200	200
	M, N	-20	40	200	200	200	200	200	147	96	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	QL	-40	30	200	200	200	200	200	200	147	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	ML, NL	-50	27	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200
	QL1	-60	30	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200

The inspection documents (or test certificates) give enough evidence that the mill product satisfies a required grade and subgrade. At the steel manufacturing plant, the quality control system places markings in the form of stamped numbers or letters on each length or batch of products so that it can be traced back to its particular cast and manufacturing route up to the point of assembly of the members [63]. The inspection document for each batch of steel is the most important document for the steel manufacturer, the fabricator, the erector, and to the subsequent purchaser of the finished component or structure. In addition to the chemical composition and mechanical properties, the inspection document should also record the steelmaking route and any heat treatments applied to the material by the steel manufacturer.

Steel that is not readily identifiable as to grade has to be tested to determine conformity to standards. A sampling protocol has to be established to provide the adequate knowledge of the materials needed for a reliable evaluation (see Appendix A). Unidentified reclaimed steel may be used in non-safety critical structures, for example in agricultural buildings.

The following procedure is proposed for the verification of the structural reusability of steel members as constituent products:

- Documentation showing the location and building structure from where the members were recovered, including the date of construction of the original building, should be provided for all members,

- All products to be reused should come from a structure constructed with elements produced in or after 1970 that was not exposed to extensive dynamic loading and other severe conditions, e.g. fire,
- All surfaces should be visually inspected, to ensure that the steel surfaces are free of rust, and that there is no excessive corrosion. (Elements need to be visually exposed, and therefore any fire protection should be removed). In the case of cold formed sections, the weld seam has to be inspected for any defects,
- Coatings containing toxic substances, e.g., lead, cadmium, asbestos, and surface scaling need to be removed by preparing the surfaces according to EN ISO 8501-1 [64],
- Members from reclaimed steel should not include welded splices (unless the welds are tested) and should not have holes in locations where new holes are to be drilled in the member (the minimum of 3 times the hole diameter or 100mm is a reasonable detailing rule for the distance between new holes and splices),
- Sectional dimensions (if not known) should be measured and the sections classified, as in Table 5.4 and Table 5.5. At least three locations along the members should be selected for comparison against nominal values,
- For open cross-sections (wide flange H and I- section beams), EN 10034 specifies tolerances on shape dimensions of these members. The following tolerances have to be adopted: height of cross-section, flange width, web thickness, flange thickness, out-of-squareness, and web off-centre,
- For closed cross-sections that are Circular Hollow Sections (CHS) and Square (SHS) and Rectangular Hollow Sections (RHS), EN 10219-2 specifies tolerances on shape dimensions of cold formed structural hollow sections, while EN 10210-2 specifies tolerances on shape dimensions of hot finished structural hollow sections. The following tolerances should be adopted: outside dimensions (CHS and RHS), thickness (CHS and RHS), out-of-roundness (for CHS), concavity/convexity (for RHS) and squareness of sides (for RHS),
- Tolerances on the member straightness should comply with EN 1090-2 and for CHS and RHS should comply with EN 10219-2 and EN 10210-2. Tolerances on older sections may be different and therefore some straightening may be required, e.g. see Table 5.8 for historical data from the UK and Romania,
- The members should have a smooth surface. However, bumps, cavities, or shallow longitudinal grooves resulting from the manufacturing process are permissible, provided the remaining thickness is within tolerance. Surface defects may be removed by grinding, provided that the thickness(es) of the cross-section after the repair is not less than the minimum permissible thickness. If the actual dimension after blasting/grinding does not meet the nominal dimensions minus the nominal tolerance, the section should be relegated to the next lighter section,
- Diffuse necking (reduction in cross-section) is not permitted for example in connections and elements in tension,
- Reclaimed sections that are outside economic repair/reconditioning should be scrapped,
- Reclaimed structural steel should be classified for design purposes according to CEN/TS 1090-201:2024 and presented in Section 5.4.1.

Table 5.8 Review of geometrical tolerances for individual members

Products	Tolerances				
	BS4 UK (1962) [65]	Dorman Long UK (1964) [66]	NSSS UK (1994) [67]	EN 1090-2 EU (2018) [8]	STAS 767 RO (1988) [68]
Beam	L/960	L/960	L/1000 or 3 mm	L/1000	L/1000, but max. of 15 mm
Column up to (but not including) 9.14 m	L/714	L/960	L/1000 or 3 mm	L/1000	
Column up to 13.72 m	L/960	L/960	L/1000 or 3 mm	L/1000	
Columns over and equal to 13.72 m	L/960 – 4.75 mm	L/960 + 9.5 mm	L/1000 or 3 mm	L/1000	

5.5.2 Reliability

The reliability of the Eurocode 3 design methods is guaranteed through the use of partial factors. The recommended values of these factors are defined in the relevant part of Eurocode 3 and in the respective national annexes. For reclaimed sections there are currently no specific information provided.

If the testing protocols and requirements defined in CEN/TS 1090-201:2024 [2] it seems reasonable to apply the same partial factors as long as there are no other provisions on a national level.

It is also important to note that EN 1993-1-1:2022, Annex E, [15] provides the statistical properties of geometric and material characteristics that were assumed during the determination of the current values of the partial factors.

The assumed scatter bands (mean values, coefficients of variation) for the dimensional properties are given in Table E.2 of EN 1993-1-1:2022. For dimensional properties not specifically mentioned in Table E.2, it is assumed that the mean values are equal to the nominal values and the standard deviations are equal to half of the interval between the nominal value and the lower bound of the applicable tolerance interval in EN 1090-2 or other relevant product standards. The values in Table 5.9 (Table E.2 of [15]) represent the products currently available on the European market satisfying the relevant European product standards.

Table 5.9 Assumed variability of dimensional properties [15]

Dimension type	**Parameter**	**Mean value**	**Coefficient of variation**	**Upper reference value**	**Lower reference value**
		X_{m}		$X_{5\%}$	$X_{0.12\%}$
Outer dimensions of cross-section	Depth h	$1.0\ h_{\mathrm{nom}}{}^{\mathrm{a}}$	0.9 %	$0.98\ h_{\mathrm{nom}}{}^{\mathrm{a}}$	$0.97\ h_{\mathrm{nom}}{}^{\mathrm{a}}$
	Width b	$1.0\ b_{\mathrm{nom}}{}^{\mathrm{a}}$	0.9 %	$0.98\ b_{\mathrm{nom}}{}^{\mathrm{a}}$	$0.97\ b_{\mathrm{nom}}{}^{\mathrm{a}}$
	Outer diameter d of circular hollow section	$1.0\ d_{\mathrm{nom}}{}^{\mathrm{a}}$	0.5 %	$0.99\ d_{\mathrm{nom}}{}^{\mathrm{a}}$	$0.98\ d_{\mathrm{nom}}{}^{\mathrm{a}}$
Thickness	Rolled and welded I- and H-sections: flange thickness t_{f}	$0.98\ t_{\mathrm{f.nom}}{}^{\mathrm{a}}$	2.5 %	$0.95\ t_{\mathrm{f.nom}}{}^{\mathrm{a}}$	$0.91\ t_{\mathrm{f.nom}}{}^{\mathrm{a}}$

Dimension type	Parameter	Mean value	Coefficient of variation	Upper reference value	Lower reference value
		X_m		$X_{5\%}$	$X_{0.12\%}$
	Rolled and welded I- and H-sections: web thickness t_w	1.0 $t_{w.nom}$[a]	2.5 %	0.96 $t_{w.nom}$[a]	0.93 $t_{w.nom}$[a]
	Hot rolled (seamless) or welded structural hollow sections (acc. to EN 10210 (all parts)): wall thickness t	0.99 t_{nom}[a]	2.5 %	0.95 t_{nom}[a]	0.92 t_{nom}[a]
	Cold formed sections made from coils or plates (acc. to EN 10219 (all parts)): wall thickness t	0.99 t_{nom}[a]	2.5 %	0.95 t_{nom}[a]	0.92 t_{nom}[a]
	All other welded sections made from heavy plates: thickness$t$	0.99 t_{nom}[a]	2.5 %	0.95 t_{nom}[a]	0.92 t_{nom}[a]

[a] Nominal dimensions according to the applicable product standard or specification.

Table 5.10 gives the statistical values assumed for the material properties, i.e. yield strength and ultimate tensile strength.

Table 5.10 Assumed variability of material properties [15]

Parameter	Steel grade	Mean value	Coefficient of variation	Upper reference value	Lower reference value
		X_m		$X_{5\%}$	$X_{0.12\%}$
Yield strength, f_y	S235, S275	1,25 $R_{eH,min}$ [a]	5,5%	1,14 $R_{eH,min}$ [a]	1,06 $R_{eH,min}$
	S355, S420	1,20 $R_{eH,min}$ [a]	5,0%	1,1 $R_{eH,min}$ [a]	1,03 $R_{eH,min}$
	S460	1,15 $R_{eH,min}$ [a]	4,5%	1,07 $R_{eH,min}$ [a]	1,00 $R_{eH,min}$
	Above S460	1,10 $R_{eH,min}$ [a]	3,5%	1,04 $R_{eH,min}$ [a]	1,00 $R_{eH,min}$
Ultimate tensile strength, f_u	S235, S275	1,20 $R_{eH,min}$ [a]	5,0%	1,11 $R_{m,min}$ [a]	1,03 $R_{m,min}$
	S355, S420	1,15 $R_{eH,min}$ [a]	4,0%	1,08 $R_{m,min}$ [a]	1,03 $R_{m,min}$
	S460 and above	1,10 $R_{eH,min}$ [a]	3,5%	1,04 $R_{m,min}$ [a]	1,00 $R_{m,min}$
Modulus of elasticity, E	All steel grades	210 000 MPa	3,0%	200 000 MPa	192 000 MPa

[a] $R_{eH,min}$ and $R_{m,min}$ are the minimum yield strength R_{eH} and the lower bound of the ultimate tensile strength R_m, according to the applicable product standard, e.g. EN 10025 (all parts).

If the assessment of the properties of the elements gives similar statistical properties for reclaimed members, it seems again reasonable to apply the values of partial factors generally used for new elements provided that also geometrical tolerances, such as out-of-straightness or out-of-flatness, comply with the product standards.

5.6 Structural components or entire primary structure

Steel structures and steel construction products are, in general, highly demountable. Provided that attention is paid to deconstruction in the design stage, there is no technical reason why almost all the steel building stock should not be regarded as components for future use in new applications. In some sectors, e.g., industrial and agriculture, reuse of single-storey steel structures and cladding components is already relatively common.

According to EN 1090-2, a component represents part of a steel structure, which may itself be an assembly of several smaller components. A steel structure represents an organised combination of connected components designed to carry loads and provide adequate rigidity.

The following selection and acceptance criteria for reuse of a steel component, i.e., part of a structure, or entire primary structure, in the scope of the current project are proposed:

- The structural components, or the entire primary structure, should belong to a steel building constructed after 1970, using elements manufactured in or after that year, and must not have been subjected to significant dynamic loading or other extreme conditions,
- All reclaimed steel shall be measured and certified for its geometric section properties, and then classified according to the system proposed in Section 5.4.1 of this document,
- First, the individual structural members are evaluated according to Section 5.5. The term “evaluation”, in this context, is as defined in EN 1090-1, i.e., to demonstrate compliance with the requirements, such as material properties, geometry and structural characteristics,
- In addition to Section 5.5, all welds should be 100% visually inspected throughout their entire length for surface imperfections according to EN ISO 17637 [69]. The visual inspection should be carried out before any other NDT inspection. If surface imperfections are detected, additional surface testing by liquid penetrant testing or magnetic particle inspection should be carried out on the inspected weld. Generally, ultrasonic testing or radiographic testing applies to butt welds and liquid penetrant testing, or magnetic particle inspection applies to fillet welds,
- Existing bolts from previous applications should not be reused,
- In case of remanufacturing, the reused steel component / detail / structural component or module / primary structure can be CE marked to EN 1090-1 [7].

Fig. 5.4 presents the framework for the reuse process of a steel structure or structural component.

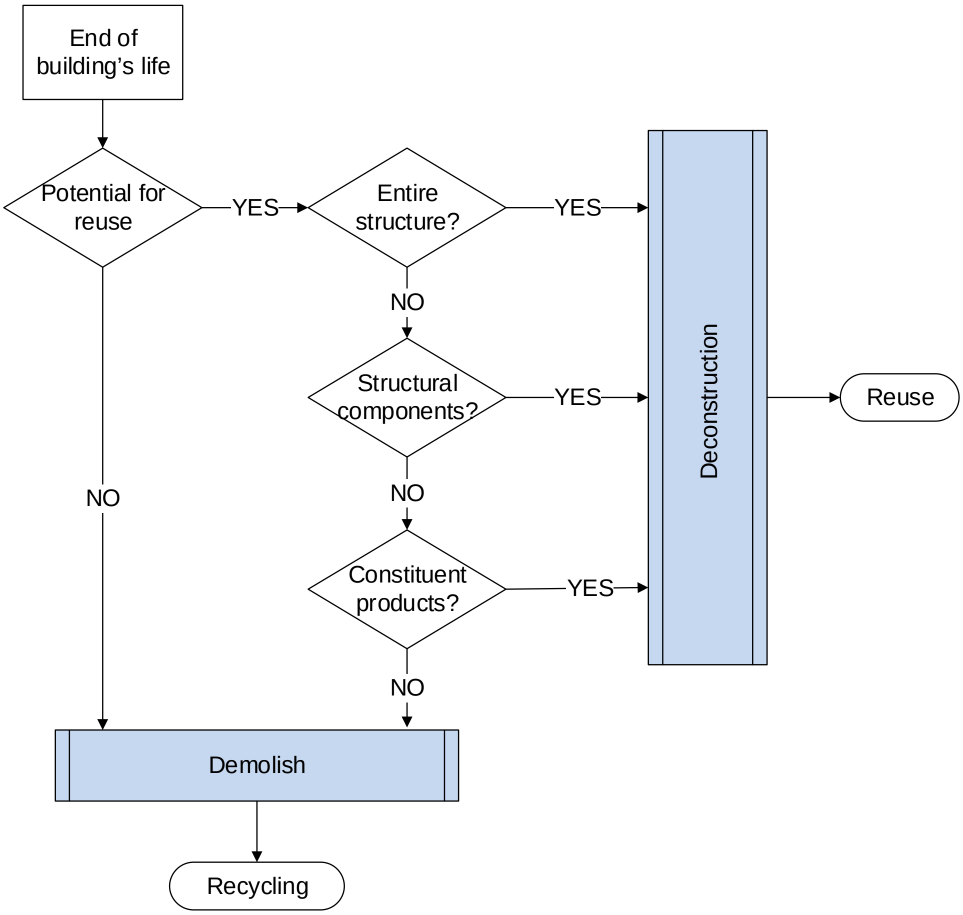


Fig. 5.4 Overall framework for reuse process of steel structure/components

However, some degree of uncertainty is inevitably associated with the use of reclaimed steelwork. The overall framework for verification of the reusability of components, or entire primary structures is presented in Fig. 5.5.

The flowchart in Fig. 5.5 identifies three possible classes, after checking the eligibility and compliance with the tolerances in EN 1090-2:

- Class RSC1: the structural component has not been CE marked in the first life and has to be certified as a new structural component/structure. Detailed investigations are necessarily for this Class – steel materials meet performance requirements through extensive testing,
- Class RSC2: the structural component has been CE marked in the first life according to EN 1090-1 and the original documentation is available. Each component should be reassessed to confirm compliance with EN1090-2 and the harmonised standards. Steel materials meet performance requirements through limited testing and with approved quality assurance from original certificates. The welding has to pass the visual and other NDT inspections. The reclaimed structural component can be reused in designs according to EN 1993-1-1 with some restrictions: (i) plastic global analysis is not allowed when the reclaimed steel is reused; (ii) a conservative value of the γ_{M1} safety factor is recommended to address possible uncertainties as the assessment processes are likely to be less reliable than those undertaken for the new structural steel components,

- Class RSC3: the structural component was not CE marked, or the CE marking documentation is unavailable. The structural components have to conform with EN 1090-2 standard. Steel materials and welding can be evaluated through limited material testing and the steelwork recertified. The reclaimed structural component can be reused in designs according to EN 1993-1-1 with the same restrictions as for Class RSC2.

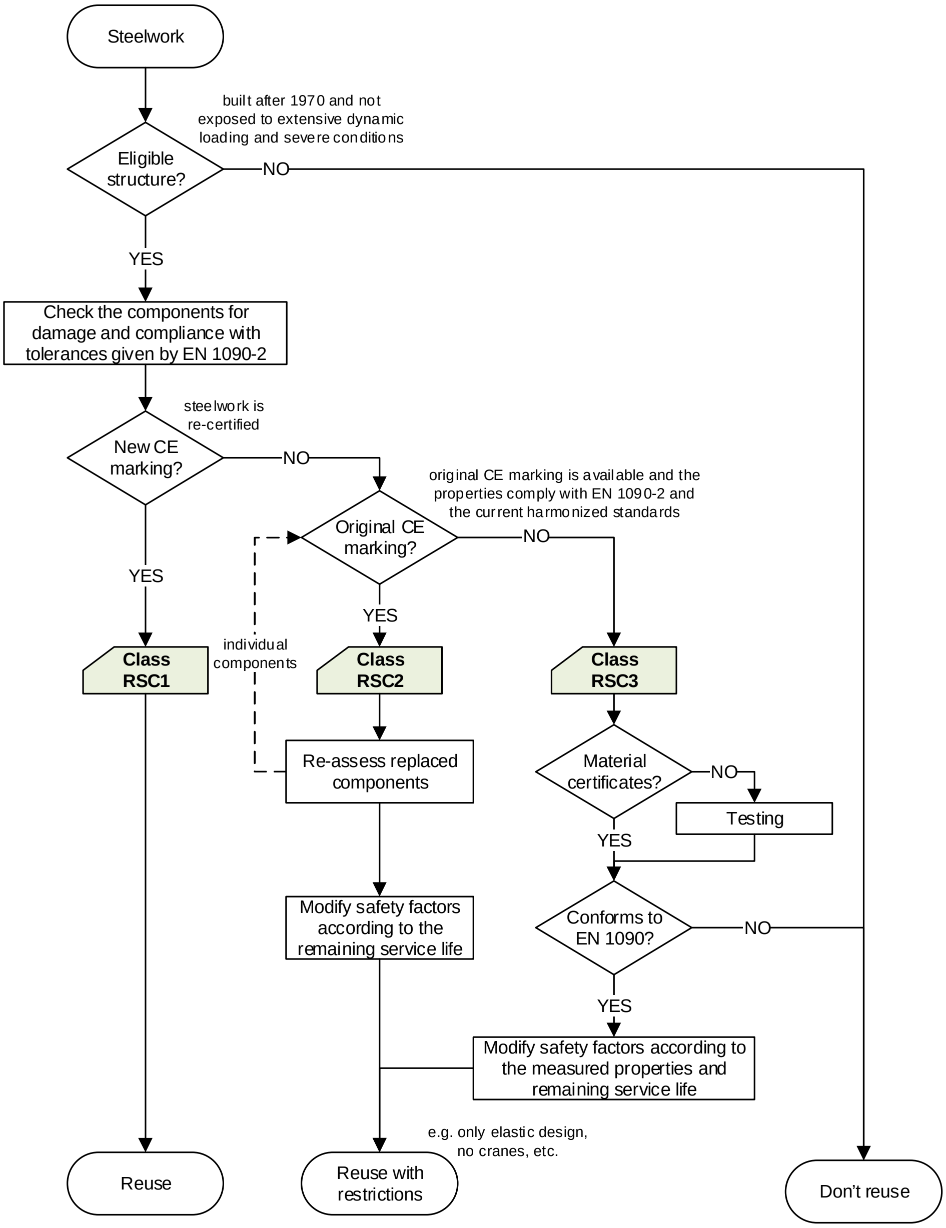


Fig. 5.5 Overall framework for verification of the reusability of components or entire primary structures

5.6.1 Execution classes

The execution class required for the steelwork is a reliability differentiator for providing choice of quality, testing and qualification requirements. The basis for CE marking is that the manufacturer declares that its products meet the specified performance characteristics that are defined as essential to the application of the products in construction.

For any project, the required quality of fabrication or execution class (EXC) must be specified. EN 1090-2 [8] requires the execution class to be specified for the works as a whole, an individual component, or a detail of a component. In some cases, the execution class for the structure, the components and the details will be the same, while in other cases the execution class for the component and the details may be different from that for the whole structure.

EN 1090-2 specifies requirements, which are mostly independent of the type and shape of the steel structure (e.g. buildings, bridges, plated or latticed components), including structures subjected to fatigue or seismic actions. Certain requirements are differentiated in terms of execution classes.

According to EN 1090-2 there are four classes, ranging from EXC1, for which the requirements are the least strict, to EXC4, for which the requirements are the most strict. However, it should be noted that the requirements of EXC4 should be defined individually but considering at least those applicable to EXC3. It is up to the designer to select the EXC required for the structure, an individual component, or a particular detail of a component.

The selection of the execution class should be based on the following three factors:

- the required reliability;
- the type of loading for which the structure, component or detail is designed;
- the type of structure, component, or detail.

In terms of reliability management, the selection of the execution class should be based on the required Consequences Class (CC). The consequences class are defined in EN 1990:2023 [9]. The provisions of Eurocodes cover design rules for structures classified as CC1 to CC3.

In terms of the type of loading applied to a steel structure or component or detail, the selection of execution class should be based on whether the structure or component or detail is designed for static actions, quasi-static actions, fatigue actions, or seismic actions.

The selection of execution class (EXC) based on the type of loading is as given in Table 5.11 from EN 1993-1-1:2022.

Table 5.11 Selection of execution class (EXC) based on the type of loading

Consequence Class (CC)	**Type of loading**				
	Static, Quasi-static	**Seismic**			**Fatigue**[b]
		DC1[a]	**DC2**[a]	**DC3**[a]	
CC3	EXC3[c]	EXC3[c]	EXC3[c]	EXC3[c]	EXC3[c]
CC2	EXC2	EXC2	EXC2	EXC3[d]	EXC3
CC1	EXC1	EXC2[e]	EXC2	EXC2	EXC2

[a] Seismic ductility classes (DC's) are defined in prEN 1998-1-1.

[b] See EN 1993-1-9.

[c] EXC4 may be considered for special cases, including those typically covered by CC4 of EN 1990.

[d] Only the primary seismic resisting system falls in EXC3; the gravity load resisting system may fall in EXC2.

[e] If the seismic action index is not greater than 2,5m/s^2 (low seismic action class, see prEN 1998-1-1), the execution class of structures in DC1 may be EXC1.

If the required execution class for particular components and/or details is different from that applicable to the structure in general, then these components and/or details should be clearly identified.

The National Annexes may specify the choice of execution class in terms of types of components or details. The following is recommended [15]:

> *"If EXC1 is selected for a structure, then EXC2 should apply to the following types of component:*
> *a) welded components manufactured from steel products of grade S355 and above;*
> *b) welded components essential for structural integrity that are assembled by welding on the construction site;*
> *c) welded components of CHS lattice girders requiring end profile cuts;*
> *d) components with hot forming during manufacturing or receiving thermic treatment during manufacturing."*

The present document focusses on the reuse of single- and multi-storey steel buildings and their components. The reused components can be framed on *Consequences Class* CC1 or CC2 according to Annex A of EN 1993-1-1, for static, quasi-static or seismic DC1 type of loading. Consequently, execution class 2 (EXC2) is the most appropriate for the majority of single- and multi-storey buildings, i.e. buildings or parts of buildings not covered by CC1 or CC3. The list of requirements related to execution classes is given in Annex A.3 of EN 1090-2.

Reclaimed structural steel components must clearly be treated differently, as it might have been manufactured to a withdrawn standard and is most unlikely to have any documented test results from time of manufacture. EN 1090-2 sanctions the use of other materials by stating that: "*If constituent products that are not covered by the standards listed are to be used, their properties shall be specified*". There will be no difference in the fabrication processes, procedures, standards or tolerances for either new steel or reclaimed steel. It is therefore appropriate that reclaimed structural steel components can be CE Marked in accordance with EN 1090. In addition to careful control of the structural components, material properties must be declared according to EN 1090-2.

Specification of EXC may not always be sufficient alone for the differentiation of the acceptance criteria and the extent of inspection for welds/details of different importance or criticality. This may result in the following:

a) the acceptance criteria may become too onerous for welds that are not important,
b) the extent of specified inspection may become too large for welds that are not important,
c) the specified inspection may miss the critical locations.

The use of weld inspection classes (WICs) (see Annex L of EN 1090-2) may be useful in directing the scope and percentage extent of supplementary testing according to the criticality of the weld. This may be beneficial both from a safety point of view and from an economic point of view, as unnecessary inspection and repair may be avoided. The initial choice of weld inspection classes (WICs) should take into account the likelihood that defects would arise for particular weld configurations (e.g., welds to be executed in difficult conditions such as overhead welds, site welds, welds for temporary attachments). Subsequently, weld inspection classes (WICs) may be reduced based on experience in production.

Weld inspection classes are to be used based on the following criteria for selection:

a) utilisation for fatigue,
b) consequence of failure of weld for the structure,
c) direction, type and level of stresses.

Annex A.3 of EN 1090-2:2018 lists requirements specific to each of the execution classes referenced in this European Standard. “Nr” in the table means “No specific requirement in the text”.

In the case of reclaimed components or reuse of the primary structure, the contractor or the stockholder is responsible for specifying the Execution Class (EXC) for the structure as a whole, as well as for individual components or details, where a different EXC is appropriate. If a different EXC is assigned to a component or detail, it shall not be lower than the ECX specified for the works as a whole. The EXC for any component or detail shall be clearly identified in the new specification if it differs from the EXC class for the structure.

The organisation holding the reclaimed structural components or the entire primary structure has important responsibilities involving the examination and testing of the steelwork, keeping of comprehensive records and formal declarations of material properties when the reclaimed structural components or the entire primary structure is distributed into the supply chain.

When reclaimed structural components or entire primary structure are distributed into the supply chain, it must be accompanied by a formal declaration, following the requirements of EN 1090-2. The declaration must make clear which properties have been assumed, and which have been determined by test.

The following properties shall be declared:

- geometrical data (tolerances in dimensions and shape),
- weldability – If required, if not “No performance determined (NPD)" may be declared,
- fracture toughness of structural steel products,
- reaction to fire – To be declared that the materials are classified as Class A1; or if a coating with organic content larger than 1%, the relevant class of the organic content,
- release of cadmium and its compounds –"NPD" to be declared,
- emission of radioactivity – "NPD" to be declared,
- durability – To be declared according to component specification,
- execution class (EXC),
- reference to component specification.

5.7 Cold formed structural steelwork elements

Purlins and side rails are usually proprietary cold rolled, thin-walled galvanised sections. Currently, Z-, C- and Σ- sections are used as purlins or side rails, the manufacturers offering design data in terms of load/span tables or software. Purlins and side rails acting as secondary beams supported by primary beams (e.g. rafters) or columns are often restrained by the building envelope (e.g. trapezoidal sheeting, cassettes, sandwich panels, etc.).

The shape and sign of the bending moment diagram depend not only on the type of loading, gravity, or uplift, but also on the support conditions of the purlin, which can be simply supported or continuous over two or more spans. When it is continuous, the purlin may have uniform cross-section in the span and over the support, or stepped cross-sections, by overlapping the profiles over the supports. In the latter case, the Z-sections can be selected to adapt the purlin capacity to the moment variation along the member, and also to the transverse load demand at the supports.

Manufacturers produce their own specific shapes, depths ranging from 100 to 350 mm and thicknesses from 0.8 mm to 3.2 mm. These purlins are usually suitable for frame spacings between 4 to 9 m and purlin spacing between 1.2 to 2.5 m.

All steels used for cold formed steel members and profiled sheeting shall be suitable for cold forming and, if relevant, for welding. Steels used for the galvanised members and sheets that are to be galvanised should also be suitable for galvanising.

EN 1993-1-3 [16] specifies that the materials in Table 5.1a are conform with harmonised product standards, while the materials in Table 5.1b are conform to EN or ISO product standards. For other steels, the suitability for cold forming shall be demonstrated by a bend test according to EN ISO 7438 [70] or by an equivalent test.

For an effective design, when spans range from 6.0 m to 7.0 m, continuous purlins are usually made with sections lapped and bolted on intermediate supports. Alternatively, double sections can be used to strengthen the purlin at intermediate supports.

Sheeting itself can be used as a continuous restraint system to prevent lateral and torsional deformations of purlins. To be effective, such restraint systems must possess sufficient translational and rotational stiffness. When the restraint by sheeting is not fully effective, non-continuous or discrete lateral bracing devices, spaced along the purlins, can be used.

The reclaiming potential of secondary cold formed steel elements is generally lower than that of primary hot rolled steelwork. This is primarily due to cladding being typically fixed with a high number of mechanical fasteners, which complicate the disassembly process and increase the risk of damaging the elements during deconstruction.

Previous sections provided an overview of the process to reuse hot rolled structural steel elements according to EN 1090-2. For cold formed structural elements, similar principles shall apply, taking into account the recommendation of EN 1090-4 [71]. Alternative specifications of source material similar to Section 5.2.5 are also possible. The following sections clarify key aspects to allow for structural steel reuse of cold formed elements.

It is unlikely that within the scope of this guide welding of cold formed steel elements is found or required for future application. Therefore, the assessment of execution processes may only be related to geometric tolerances of the cold formed steelwork according to EN 1090-4 and EN1993-1-3 recommendations.

5.7.1 Classification of cold formed reclaimed steelwork for reuse

Cold formed steel structural members and sheeting, used for load-bearing purposes in structural engineering, shall be subjected to classification with regard to strength requirements and dimensions. In this sense, the assessment of the corrosion protection system also constitutes a part of the overall classification.

According to Section 5.4.1, reclaimed steel should be classified according to the verification against (i) material performance requirements (adequacy assessment) and (ii) quality assurance requirements (reliability assessment), framing the steel in one of the following classes, i.e., class A, class B or, class C [54]. For Class C reclaimed steelwork, as a wide range of steel grades are likely to be available, it is not recommended to assume a yield and tensile strengths of more than 120 MPa and 260 MPa respectively.

Cold formed steel members are usually protected against corrosion by means of metallic coatings as specified in EN 10346 (designation of the coating mass Z, ZM, ZA or AZ) and, if necessary, by means of an additional organic coating as specified in EN 10169 [37]. The provisions of EN 10346 [36] shall apply to the determination of the coating mass. The type and scope of the tests to be performed are given in Table E.8 of EN 1090-4.

5.7.2 Selection and acceptance criteria

The following selection and acceptance criteria are proposed for assessing the reusability of secondary steelwork, i.e., purlins and side rails, in the scope of this guide:

1. The structural components (member composing the secondary structural system) or the secondary structural system, should be a part of a building and should not have been exposed to any extreme conditions,
2. Each element should be dismantled, while the stability is ensured. Elements providing lateral stability shall not be dismantled prior to the removal of the main elements or prior to the installation of the temporary bracing,
3. Structural steel components should be packed, handled, and transported in a safe manner, so that permanent deformation does not occur, and surface damage is minimised. Products that have been handled or stored in a way or for a length of time that could have led to significant deterioration shall be checked before use, to ensure that they still comply with the relevant product standard,
4. First, the individual structural members are evaluated according to EN 1090-1 [7],
5. All reclaimed steel should be certified to the section properties and classified according to the system proposed in Section 5.4.1 of this document,
6. If the initial drawings are missing, all dimensions of the components/structure shall be measured to check that they meet the tolerances, at the level of cross-section, member, or structural system, EN 1090-4. All measurements, to verify the cross-sectional shape and dimensions, shall be carried out at a distance of at least 250 mm from the end of the sections to exclude any influence of end-flare on measured results. The thickness of the section shall be measured on the flat sides of the section. The straightness and twisting of a section shall be checked over the entire length of a section resting on a flat base. The length shall be measured along the centreline of the largest surface:
    a. Essential and functional manufacturing tolerances for press braked or folded members are given in Annex D of EN 1090-4,
    b. For roll-formed members, EN 10162 [72] applies. The minus tolerance on the height of the lip of the edge stiffeners shall conform to the following: (1) the minus tolerance on the height of the lip of each individual edge stiffener shall not be larger than 10% of the nominal lip height, with a maximum of minus 2

mm; (2) the average tolerance on the height of the lip of all the edge stiffeners in each cross-section along the member length shall not be larger than half of the permitted minus tolerance for outside dimensions limited by one radius and a free edge. Positive tolerance is a functional tolerance,
    c. The thickness may be measured at any point located more than 40 mm from the edges. The tolerances on thickness shall be as given in Tables 1 to 4 of EN 10143 [73] and apply throughout the whole length,
7. The product surface shall be visually inspected to verify compliance with the requirements in Sections 7.4 to 7.6 of EN 10346 [36]. The coating surface can vary and change to a dark appearance by oxidation. The available coating masses should be in accordance with Table 11 of EN 10346. NDT tests have to be performed to check the coating thickness. If necessary, the methods described in Annex A (Z, ZF, ZA and AZ) or Annex B (AS) of EN 10346 shall be used,
8. The reused steel component or structural component can be CE marked according to EN 1090-1.

5.7.3 Material performance requirements

The evaluation of constituent products not covered by referenced standards is allowed by clause 5.1 of EN1090-4. It is stated that *"If constituent products that are not covered by the standards listed in Clause 5.3 are to be used, their properties shall be specified".*

The following properties were identified as required for an appropriate evaluation:
- Yield strength or 0.2% - proof strength ($R_{eH}/R_{p0.2}$) in MPa;
- Tensile strength (R_m) in MPa;
- Elongation after fracture A_{80} mm in %;
- Bend radius to thickness ratio, if relevant;
- Adhesion of metallic coating;
- Tolerances on dimensions and shape, including minimal thickness;

If the steel is to be welded, its weldability shall be declared as follows:
- A maximum limit for the carbon equivalent of the steel, or;
- A declaration of its chemical composition in sufficient detail for its carbon equivalent to be calculated.

In addition to the above properties, the coating mass and the coating thickness should be evaluated.

Provided that the reclaimed steel meets all relevant material requirements and the fabrication processes comply with EN 1090 standards, there will be no difference in the fabrication procedures, standards, or tolerances between the new and the reclaimed steel. Therefore, reclaimed steel structural steelwork can be CE marked in accordance with EN 1090-1.

The test regime for cold formed steelwork is intended to allow the necessary material properties according to clause 5.3 of EN 1090-4 to be declared, based on dimensional survey, by non-destructive tests, by destructive tests, or by making conservative assumptions.

For the scope of the current design guide, the nominal yield strength for cold formed elements shall be in the range of 220 N/mm^2 and 550 N/mm^2. The minimum nominal tensile strength should be in the range of 300 N/mm^2 to 560 N/mm^2. The ductility requirements for design to EN 1993-1-1 are presented in Table 5.1 (recommended values that may be modified by the NAs).

5.7.4 Adequacy and reliability assessment

Regarding hot rolled elements, evaluations of adequacy and reliability are required for cold formed steelwork to ensure that the reclaimed product can be used in structural design according to EN 1993-1-3. A testing procedure to undertake the adequacy and reliability assessment of cold formed elements is proposed in Appendix A.

5.7.5 Product properties to be declared for cold-formed reclaimed elements

This section summarises the steel properties that need to be assessed for reclaimed cold formed steel elements according to Clause 5.3 of EN1090-4 (see Table 5.12). Further commentary on these properties is also provided.

Table 5.12 Material properties to be declared according to Clause 5.3 of EN 1090-4

Property	**To be declared**	**Procedure**
Yield strength or 0.2%-proof strength (R_{eH}/$R_{p0,2}$)	Yes	Determined by non-destructive and destructive tests.
Tensile strength (R_m)	Yes	Determined by non-destructive and destructive tests.
Elongation after fracture A_{80} mm in %	Yes	Determined by destructive tests.
Tolerances on dimensions and shape, including minimal thickness	Yes	Based on dimensional survey.
Bend radius to thickness ratio, if relevant	If required	If required, determined by destructive tests.
Metallic coating composition, designation and layer mass and thickness	Yes	If required, determined by non-destructive or destructive tests and visual inspection
Adhesion of metallic coating	Yes	Based on visual inspection
In addition, if the steel is to be welded, its weldability shall be declared as follows:		
Property	**To be declared**	**Procedure**
A maximum limit for the carbon equivalent of the steel, or	If required (usually not required as welding procedures are often not used)	Maximum to be declared from manufacturer’s test certificates.
A declaration of its chemical composition in sufficient detail for its carbon equivalent to be calculated		Determined by non-destructive and destructive tests.

Yield strength, tensile strength and elongation
According to EN 10346 [36], the tensile tests shall be performed without coating, in the test direction given in Tables 7 to 11 and Section 7.2.5.2 of the same standard.

Geometric tolerances and limitations
The geometric tolerances on dimensional shape shall comply with EN 10143 [73]. EN 1993-1-3 specifies minimum thicknesses for cold formed steel elements. Recommendation from EN 1090-4 shall also be followed.

Bend radius to thickness ratio and adhesion of metallic coating
As the reclaimed steelwork is already bent, a visual inspection to assess possible cracks and the adhesion of the metallic coating near the bend region shall be undertaken for each reclaimed element. The adhesion assessment has the objective of detecting any adhesion less than “perfect” (see Appendix A for more details).

Metallic coating composition, designation and layer mass
The composition of the metallic coating needs to be specified according to EN 10346. Section 3 of EN 10346 specifies the key chemical components for each coating type. For the coating layer weight assessment, Section 7.3 from EN 10346 must be considered (see also Appendix A).

Chemical composition
For cold formed products, EN 10346 may be used, where in Table 2 of the standard the chemical composition of steels for construction is presented. The intent of this declaration is to allow the calculation of the carbon equivalent value (CEV), which is a key measure of weldability. If the reclaimed cold formed steelwork is not to be welded, the chemical composition does not need to be assessed (see Appendix A for more details).

5.7.6 Durability

The disassembling process may cause damage to the steelwork, and especially to the coating. The steelwork loses coating mass over time (at a rate that depends on the building/steelwork environment). This means that the coating mass for subsequent life cycles is reduced, thus reducing the durability of the steelwork system. Therefore, it is necessary that the test protocol assesses the remaining/available coating mass for the reclaimed cold formed steelwork.

5.8 Composite floor decking

Composite steel floor decking is widely used as a structural solution that combines the complementary properties of steel and concrete. Its main advantage lies in its ability to create a lightweight, strong, and durable system that speeds up construction while offering high performance in terms of load-bearing and fire resistance. Due to the permanent bonding of steel and concrete in current solutions, 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.

5.9 Claddings

Composite panels are covered by EN 14509 – *Self-supporting double skin metal faced insulating panels. Factory made products* [74]. Specifications. For panels manufactured after 2004, key information is available on a tape on the panel joist, which includes the manufacturer, date of manufacture, and the panel data including the core type. Since 2000, pentane has been used as the blowing agent for the core and does not contain CFC (chlorofluorocarbon) or H-CFC (hydrochlorofluorocarbon).

Roofing and cladding sheets are covered by EN 14782 – *Self-supporting metal sheet for roofing, external cladding and internal lining. Product specification and requirements* [75].

Recommendations are proposed for evaluating the potential for reusing sandwich panels. For the evaluation of safety aspects for reuse, the rules in EN 1990 (safety factors) and the rules in the harmonised product standard EN 14509 are used for the type of testing of essential properties. A basic requirement for a limited amount of testing is that the name of the manufacturer is known and that a copy of the original declared values (values given by the manufacturer) is also known. This might limit the use of a reduced testing program for panels older than 25 years, because of the lack of commonly known rules, unless they have been produced under national type approvals with an existing type testing and third-party control. For other cases, a full testing program following the rules in EN 14509 is recommended.

The evaluation of potential to reuse sandwich panels is as follows:

- Architectural or aesthetical based,
- Performance based; evaluation of essential properties as in EN 14509.

For this purpose, colour change of the surface or damages in surface are visually observed.

5.9.1 Selection and acceptance criteria

The mechanical panel properties to be declared and to be determined based on Type Testing are according to EN 14509:

- wrinkling strength,
- shear strength and shear modulus,
- creep coefficient (for permanent loads only),
- compression strength and compression modulus,
- tensile strength and tensile modulus,
- durability properties,
- tolerances.

The reference level of the mechanical properties are the values declared by the manufacturer at the time of delivery of the panels. This reference level is further called the zero level.

The evaluation of the possible degradation of the panel mechanical properties is first evaluated by comparing the level of cross panel tensile strength to the zero level. If considerable degradation (over 10% lower characteristic value compared to the declared value) is noticed, the panel shear strength and compression strength is tested. The characteristic value of the panel shear strength, determined on panels sampled from the panels dismounted is the value used for design when reusing the panels.

The mean value of the shear modulus is measured from the panels to be reused. For the wrinkling strength, compression strength and modulus, the originally declared values are reduced with the ratio of the characteristic shear strength to the originally declared shear strength. This procedure is conservative, as the experience shows that the ageing affects mostly the cross panel tensile strength and panel shear strength. Results from testing dismounted panels at the end of the 1990s indicate that the ageing rate of wrinkling strength is approximatively half of the ageing in shear strength. The material safety factors are suggested to be the same as based on original type tests.

It is suggested to test samples taken from the demounted panels for the cross panel tensile strength as specified in section A1 of EN 14509. The number of samples should be at least

3, and up to 10, which will lead to greater accuracy of the results. The density of the samples is measured from samples taken close to the samples for tensile strength.

5.9.2 Tensile strength and density

The characteristic value of the tensile strength is compared to the original declared value. If there is degradation at a level of less than 10%, the panels can be reused using the properties originally declared properties for all mechanical strength properties. If the degradation is more than 10%, a set of samples for testing shear strength and modulus and compression strength and modulus should be taken. At least 3 samples each should be taken, preferably 5 for shear and 10 for compression tests.

5.9.3 Shear strength

The shear strength and shear modulus are tested for the samples taken from the dismounted panels. If the degradation in tensile strength is not more than 10%, one shear test is performed. The test result shall be at least the same as the declared value. The full scale of tests is performed if the cross panel tensile strength has degradation more than 10% compared to original declared tensile strength. The characteristic value is calculated for the shear strength. This value is used for the design of the panels to be reused.

5.9.4 Compression strength

The compression strength is tested for the samples taken from the dismounted panels. The tests are performed if the cross panel tensile strength has degradation more than 10% compared to original declared tensile strength. The characteristic value is calculated for the compression strength. This value is used for the design of the panels to be reused.

5.9.5 Bending moment/wrinkling strength

For the bending moment or wrinkling strength, the originally declared value can be used if the tensile strength has degradation less than 10%. If the degradation is higher, then either the wrinkling strength is reduced with the same ratio as the shear strength in comparison to the originally declared shear strength, or the wrinkling strength is tested for panels sampled from the dismounted panels. The characteristic value of test results is then used in design when reusing the panels.

5.9.6 Material safety factors

The material safety values determined by the original type testing is used. Alternatively, the safety values determined form the tests on dismounted panels can be used calculated as given in section A.16 of EN 14509.

5.9.7 Durability properties

It is necessary to repeat testing for durability only if there is a degradation of tensile strength of more than 10%. In that case, only the short-term durability testing (14 days, see EN 14509 Annex B, clause B.2.4, is required for all other core types than mineral wool, and 7 days for mineral wool core, see EN 14509 Annex B, clause B.3.4). For panels with core of mineral wool, the degradation of properties shall be less than 15%, and for all other types less than 17%.

5.9.8 Tolerances

The tolerances should be visually inspected, and if a deviation is noticed, the panels are checked that they are fit for reuse.

5.9.9 Thermal behaviour

For sandwich panels with a polyurethane (PU) core, if there is reduction in closed cells ratio (see ISO 4590) is decreased by more than 10%, the thermal conductivity shall be retested and a new design value determined (EN 14509, clause A.10).

5.9.10 Fire safety

For panels with core materials using fire retardants shall be retested for the small flame behaviour in order to check that the effect of fire retardants is still active. Otherwise, a reclassification might be needed.

5.9.11 Certification for reuse

A summary of the evaluation and certification procedures for reuse of sandwich panels is presented in Table 5.13.

Table 5.13 Summary on evaluation procedure for reuse of sandwich panels

Evaluation criteria	Property
Mechanical strength	
Testing cross panel tensile strength 3 samples, a minimum (EN 14509, A1): Calculate characteristic result for tensile strength. Testing one sample for shear strength (EN 14509, A.3 or A.4)	
1. Tensile strength Actual value ≥ 0.9 × Declared value, and 2. Shear strength Actual value ≥ 0.9 × Declared value	If YES, no further testing is required. All declared values for mechanical strength can be used.
	If NO, new declared values to be determined with a test programme according to EN 14509 for (i) tensile strength, (ii) compression strength, and (iii) shear strength. The wrinkling strength is reduced with the same amount that shear strength is reduced.
Durability	
Tensile strength Actual value ≥ 0.9 × Declared value	If YES, no further testing is required. Panels are fit for use.
	If NO: For MiWo panels: The 7 days testing (see EN 14509 clause B.3.4) is to be done. The reduction in tensile strength after ageing shall not exceed 15% of the mean value of the tensile strength in ambient temperature For all other panel types: The procedure in EN 14509 Annex B.2 is followed so that the panels are tested 14 days in the temperature as described in B.2.4. The reduction in tensile strength after ageing shall not exceed 17% of the mean value of the tensile strength in ambient temperature
Tolerances	
Damage is evaluated by visual inspections	If no serious damages or faults are found, then the panel can be reused.

	If serious damages are found causing weakness in strength, insulation behaviour or tightness of joints, then those panels are rejected.
Moisture content	
Wetness of core material	If no notable wetness of core material found, the panels can be reused
Thermal behaviour	
For PU panels: 1. Closed cell ratio Actual value ≥ 0.9 × Value obtained by type testing and 2. Change in density < 10%	If YES, no further testing is required; original thermal conductivity value can be used.
	If NO, then new test for determining thermal conductivity is to be done following the rules in Section A.10 of EN 14509.
Fire safety	
Small flame tests, see clause C.1.2 of EN 14509	Tests to be done with core material including fire retardants. The classification is checked and if needed reclassified. The panels are fit for use, fulfilling the requirements in the project for reuse.

6 STRUCTURAL ANALYSIS AND DESIGN FOR EXISTING STEELWORK

This section discusses several considerations that may affect the design of structures using reclaimed steel members. The principles of Limit State design should be followed and the rules for resistances and serviceability given in Parts 1.1 and 1.8 of EN 1993 may be applied, using resistance partial factors γ_M and the same methods of analysis and design.

Structures made from (or including) reused steel components have to satisfy the same basic principles as given in EN 1990. For the “new” design life, the structure shall be designed and constructed according to the following conditions:

- Resist all actions likely to occur based on the member resistance,
- Remain fit for use in terms of serviceability and durability,
- Satisfy modern regulations in terms of structural integrity.

In European practice, buildings other than agricultural, temporary and monumental buildings are designed for an intended working life of 50 years, and this is reflected in the characteristic values of actions found in EN 1991, and the partial factors applied to those actions. The design service life affects the design values of the effects of actions but not the resistance and serviceability verifications presented below.

Ductility and toughness must be adequate for the structure to perform as intended. Typical design assumptions and Eurocode guidance assume a minimum level of ductility to allow compact flexural members to reach the plastic capacity of the section and to allow localised tensile yielding without rupture at stress concentrations, see Table 5.1. In fact, the designer relies on ductility for a number of aspects of the design, including stress redistribution in the ultimate limit state (ULS), in the design of bolt groups, and in the fabrication process for welding, bending, and straightening. In structural design using reclaimed steel, potential reductions in ductility and toughness can generally be neglected. This is because, under normal building service conditions, the strain demands are typically less than 1.5%, and these low strain levels do not significantly affect structural performance.

In most cases, reclaimed steel members can be expected to perform as intended for new steel, without accounting for any material property changes. However, geometric imperfections may affect the member buckling resistance and therefore it may be necessary to increase the relevant partial factor.

6.1 Achieving reliability

The application of the partial factor method requires the definition of the design values of the actions, material and product properties, geometrical data, and model uncertainties. The design values for the actions, Q_d, are obtained from the characteristic values, Q_k, based on a 50-year reference design service life and the corresponding target reliability. The target value of the reliability index β is related to the probability of failure, P_f, corresponding to a specified reference period, as follows:

$$\boldsymbol{P}_{\mathrm{f}} = \boldsymbol{\Phi}(-\boldsymbol{\beta}) \tag{6.1}$$

where $\Phi(\,)$ is the standard normal cumulative probability distribution function.

Actions on structures are defined in EN 1991-1-1 [10]. Self-weights and imposed loads are not sensitive to the reference period, and therefore, the normal 50-year reference period can still be used. For snow loads and wind actions, EN 1991 gives adjusted values for reference periods other than the 50-year period in Annex D of EN 1991-1-3 [11] for snow loads and Note 4 in Clause 4.2 in EN 1991-1-4 [12] for wind loads.

EN 1990 [9] defines five Consequence Classes (CC), depending on the consequences of failure or malfunction of the structure, and provides design rules for three of them, as follows:

- CC0: *very low* consequence for loss of human life, or economic, social or environmental consequences *insignificant,*
- CC1: *low* consequence for loss of human life, and economic, social or environmental consequences *small or negligible*, $\beta_{50\text{-year}} = 3.3$,
- CC2: *medium* consequence for loss of human life, economic, social or environmental consequences *considerable*, $\beta_{50\text{-year}} = 3.8$,
- CC3: *high* consequence for loss of human life, or economic, social or environmental consequences *very great*, $\beta_{50\text{-year}} = 4.3$,
- CC4: *extreme* consequence for loss of human life, or economic, social or environmental consequences *huge.*

It is also noted that designs with the partial factors given in the Eurocodes generally lead to a structure with a β value greater than 3.8 for a reference period of 50-year.

The design service life of the structure is not explicitly linked to the consequence class in EN 1990 and can be understood as a period for which a structure is to be used for its intended purpose, with anticipated maintenance, but without major repairs. Annex A.1.4 of EN 1990:2023 gives the following categories together with design service life for permanent structures:

- Agricultural and similar structures / Replaceable structural parts with a design service life of 25 years (category 2),
- Building structures not covered by another category, with a design service life of 50 years (category 3),
- Monumental building structures with a design service life of 100 years (category 4).

Structures or parts of structures that can be dismantled in order to be re-used should not be classified as temporary structures.

Structures designed according to the Eurocodes are expected to perform and remain fit for the appropriate design service life. For typical buildings, the working life is usually 50 years, i.e., category 3, corresponding to a reliability index $\beta_{50\text{-year}} = 3.8$, or $\beta_{1\text{-year}} = 4.7$, corresponding to a failure probability of ~10^{-4} / year. If a structure is intended for a shorter design service life, a lower target reliability index may be justified. This would correspond to $\beta_{50\text{-year}} < 3.8$ for a 50-year period. However, according to ISO 13822 [76], a minimum value of $\beta_{50} \geq 2.3$ should still be maintained to ensure human safety. On the contrary, if the design life is extended, for example, to 100 years, a $\beta_{50\text{-year}} > 3.8$ would be appropriate, reflecting a greater reliability requirement over a 50-year reference period. It should be noted that these β-values are target values used in the design to ensure a consistent level of reliability. They represent theoretical benchmarks and do not necessarily reflect actual failure rates.

Gulvanessian et al. [77] clearly explain that the β indices are used as operational values for code calibration purposes and for comparison of reliability levels of structures that naturally depend on the design working life and are used in the whole system *actions – resistances – partial factors*.

Different measures to reduce the risk of failure may be interchanged to a limited extent, provided that the required reliability level is maintained. When designing with reclaimed steel, it may be necessary to compensate for a slightly lower partial factor by a high level of quality management, control and inspection to the structure. This is an example of a reliability differentiation by the requirements of the quality levels.

Reliability differentiation may also be applied through (i) the partial factors for actions γ_F, or (ii) the partial factors for resistance, γ_M, which is further elaborated next. The first option is usually preferred.

6.1.1 Partial factors for actions

The partial factors for actions allow for the variability of loading to be considered in the design. In most cases, the probabilities of failure increase by more than expected loads, but less than intended self-weight load counteracting overturning moments may also have the same effect of increasing failure probability.

The actions considered for persistent and transient (fundamental) design situations should include:

- the design value of the leading variable action;
- the design combination values of accompanying variable actions.

When applying partial factors to actions, combinations of actions ΣF_d for persistent and transient (fundamental) design situations should be calculated according to EN1990:2023, by one of the following:

- formula (8.12) of EN1990:2023 or (6.2) below; or
- the most adverse of the two expressions in Formula (8.13) of EN1990:2023 or (6.3) below; or
- the most adverse of the two expressions in Formula (8.14) of EN1990:2023 or (6.4) below.

$$\Sigma F_\mathrm{d} = \sum_i \gamma_{\mathrm{G},i} G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j} \psi_{0,j} Q_{\mathrm{k},j} \tag{6.2}$$

or

$$\Sigma F_\mathrm{d} = \begin{cases} \sum_i \gamma_{\mathrm{G},i} G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1} \psi_{0,1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j} \psi_{0,j} Q_{\mathrm{k},j} \\ \sum_i \xi_i \gamma_{\mathrm{G},i} G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j} \psi_{0,j} Q_{\mathrm{k},j} \end{cases} \tag{6.3}$$

or

$$\Sigma F_\mathrm{d} = \begin{cases} \sum_i \gamma_{\mathrm{G},i} G_{\mathrm{k},i} \\ \sum_i \xi_i \gamma_{\mathrm{G},i} G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j} \psi_{0,j} Q_{\mathrm{k},j} \end{cases} \tag{6.4}$$

where

F_{d}	represents the design value of an action;
Σ	denotes the combination of the enclosed variables;
$\gamma_{\mathrm{G},i}$	is the partial factor for permanent action *i*;
$G_{\mathrm{k},i}$	is the characteristic value of permanent action *i*;
$\gamma_{\mathrm{Q},1}$	is the partial factor for the leading variable action 1;
$\psi_{0,1}$	is the combination factor for the leading variable action 1 (if applied);
$Q_{\mathrm{k},1}$	is the characteristic value of the leading variable action 1;
$\mathrm{Q}_{\mathrm{k},j}$	is the characteristic value of an accompanying variable action *j*;
$\psi_{0,j}$	is the combination factor for the variable action *j;*
$\gamma_{\mathrm{Q},j}$	is the partial factor for the variable action *j*;
ξ	is the reduction factor applied to unfavourable permanent actions; the value is 0.85 unless the National Annexes gives a different value.

Provided its coefficient of variation is small, a permanent action, *G,* should be represented by a single characteristic value G_{k}. If a single characteristic value of G_{k} is used, then its value may be taken as the mean value of *G*. If the uncertainty in *G* is not small, or if the structure is sensitive to variations in its value or spatial distribution, then the permanent action *G* should be represented by upper and lower characteristic values $G_{\mathrm{k,sup}}$ and $G_{\mathrm{k,inf}}$ respectively. The upper (or "superior") characteristic value $G_{\mathrm{k,sup}}$ should be selected as the 95% fractile and the lower (or "inferior") characteristic value $G_{\mathrm{k,inf}}$ as the 5% fractile of the statistical distribution of *G*.

Combination of actions for ultimate limit states with partial factors on actions should be chosen depending on the design situation, according to EN1990:2023:

- Table A.1.3, when using Formula (8.12); or
- Table A.1.4, when using Formula (8.13); or
- Table A.1.5, when using Formula (8.14).

If design values of actions for persistent and transient (fundamental) design situations are chosen according to Table A.1.4 or Table A.1.5 of EN1990:2023, then the most adverse of the two expressions in the relevant formula for combination of actions shall be verified.

It is common practice to lower the required safety level when evaluating and upgrading existing structures, as long as the limits for human safety are not exceeded, see refs. [78] and [79]. This is justified by the fact that, for existing structures, a shorter design life is often assumed and accepted. Similarly, for designs with reclaimed steelwork, it is reasonable to consider the option to assume a shorter design life, say 15-30 years (category 2 above).

6.1.2 Feasible scenarios to adopt a lower design life

In the previous section, it was suggested that the combination factors for the actions could be slightly reduced when designing with reclaimed steel assuming a shorter expected service life. It is recommended that this option require a higher level of quality management control and inspection to the structure.

For a new building, standard EN 1990 reliability requirements must be met (even if individual reclaimed elements are used). Examples where the lower partial factors for a notional design working life of 15-30 years can be used are: (i) retrofitting of existing buildings, or (ii) the cases where the whole building is relocated to a different location.

When designing a new structure while promoting steel reuse, it is possible to adjust the structural layout, such as the spacing of floor beams or frames, to ensure that the load effects on reclaimed elements remain within acceptable limits. By doing so, the structure can meet the standard reliability requirements for a notional design working life of 50 years, in accordance with EN 1990. This approach allows compliance with the Eurocode design process while accommodating the variability inherent in reused steel components.

6.1.3 Partial factors for resistance

The partial factors on resistance defined in EN 1993-1-1 are summarised in Table 6.1. The characteristic values of resistance are divided by the relevant partial factors to obtain their design resistances. These values are nationally determined parameters and can be modified in the National Annex used to implement EN 1993-1-1 in each country, see Table 6.2. The values in these Tables are given for new steels, and were obtained from test data collected between 1969 and 1980, see [80] and [81], and later, in 2002 [82].

The use of reclaimed steel has been restricted to buildings fabricated and constructed after 1970. Thus, it is unlikely that the steel properties are different from those steels used in calibrating the partial factors for cross-section verifications, γ_{M0} and γ_{M2}. Both factors accommodate the variability of material strength, so that the steel strength in the actual structure may differ from the strength used in calculations. Thus, the steelwork designer can safely adopt the same values from Table 6.1 for γ_{M0} and γ_{M2} in designs using reclaimed steel.

Table 6.1 Partial factors γ_M for resistance in EN 1993

Partial factor		**Recommended (CEN) value**
γ_{M0}	Resistance of cross-sections	1.00
γ_{M1}	Resistance of members to instability	1.00
γ_{M2}	Resistance of cross-sections in tension to fracture	1.25

Table 6.2 Partial factors γ_M for resistance in the National Annexes*

Partial factors	**Austria**	**Belgium**	**Denmark**	**Finland**	**France**	**Germany** [(i)]	**Italy**	**Ireland**	**The Netherlands**	**Norway**	**Portugal**	**Romania**	**Spain**	**Sweden**	**UK**
γ_{M0}	1.00	1.00	**1.10**	1.00	1.00	1.00	**1.05**	1.00	1.00	**1.05**	1.00	1.00	**1.05**	1.00	1.00
γ_{M1}	1.00	1.00	**1.20**	1.00	1.00	**1.10**	**1.05**	1.00	1.00	**1.05**	1.00	1.00	**1.05**	1.00	1.00
γ_{M2}	1.25	1.25	**1.35**	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	**a**	**1.10**
$a = \min\left(1.10, \frac{0.9 f_u}{f_y}\right)$; [(i)] For nonlinear analysis, assume $\gamma_{M0} = 1.10$															
*Partial factors for resistance correspond to the 2006 edition of EN 1993-1-1.															

The partial factor γ_{M1} is employed when designing members (beams and columns) for stability. The problem of stability requires consideration of the material properties and also a number of important factors usually grouped under the heading of *imperfections*, which include initial lack of straightness, accidental eccentricities of loading, and residual stresses. The design is usually based on the concept of *buckling curves*, which relate buckling resistance to the member's non-dimensional slenderness. Although reclaimed members have to satisfy all geometrical tolerances, it might be reasonable, in some circumstances, to consider a higher γ_{M1} value for stability verification when using reclaimed steel. This provides an added margin of safety to account for uncertainties related to the material's history, potential imperfections, and variability not fully captured in standard design assumptions. A possible approach is given in Appendix B.

If a structure is kept at its original location (in-situ reuse), there is no reason to increase the required levels of safety. This means that values of the correction factor $K_{\gamma M1}$ for existing carbon steel elements erected after 1970 may be taken as equal to 1 for such reuse scenario. The value of $K_{\gamma M1}$ is also related to the uncertainty of multiple transportation, disassembly, erection processes as well as in testing procedures to assess geometric imperfections. As most of the uncertainties are not allowed for if the building remains at its original location, the value of $K_{\gamma M1} = 1.0$ can be used.

6.2 Structural (static) analysis

Global structural analysis for ULS shall be carried out in accordance with the principles from EN 1993-1-1, with proper allowance for global (P-Δ effects, for the structure) and local (P-δ effects, for the member) imperfections and second order effects.

Global analysis may also be first- or second-order, depending on the horizontal flexibility of the structure, which dictates if ignoring second-order effects may lead to an underestimation of the internal forces and bending moments. For structures sensible to global second order effects, it is recommended that global P-Δ effects are accounted for by a geometric non-linear analysis (usually undertaken using software) or using the amplification factor according to Section 7.2.2(12)-(13) of EN 1993-1-1:2022.

Global elastic analysis is recommended when designing with reclaimed steelwork to obtain the internal forces and displacements in a structure. A geometrically linear analysis has the advantage that the superposition of the internal forces from different load cases may be used. Depending on the cross-section class, the design of members may be carried out based on the plastic or elastic cross-section resistance, according to EN 1993-1-1.

The member resistances at the ULS limit states control the safety of the structure and must be satisfied. The verification of whether a structure or member has satisfied this limit state is a technical verification based on the provisions from the design standard EN 1993-1-1:2022 (see Section 6.3).

The serviceability limit state, SLS, outlines the functional performance of the structure, and is usually based on expectations of the building owner, who needs to specify the performance criteria to be met. SLS are not safety critical, but they can impair the use and durability of the building, for example, by causing cracking and leakage through excessive deflection of cladding (see Section 6.5).

6.3 Ultimate limit states

6.3.1 Design of members: resistance of cross-sections

The rules set out in Clause 8.2 of EN 1993-1-1:2022 may be applied without restrictions in the design checks for cross-section resistance taking into consideration the cross-section classes from Clause 7.5. The resistance models should be based on the net cross-section properties. The steelwork designer can safely adopt the values for γ_{M0} and γ_{M2} according to the appropriate national annex to EN 1993-1-1:2022 (see Section 6.2.3).

6.3.2 Design of members: stability

For stability verifications of members, account should be taken of local imperfections, in accordance with Clause 7.3.3 of EN 1993-1-1:2022. Usually, this is treated implicitly within the procedures for checking individual members in Clause 8.3. In the case of members using reclaimed steel, it is recommended to substitute γ_{M1} for $\gamma_{M1,mod}$ (see Section 6.1.3).

In general, the gross cross-section of the structural members is used for determining the buckling resistance. However, if bolt holes are located within the critical cross-section (maximum internal section forces) and reduce the cross-section by more than 15% within the critical member segment, the net cross-sectional properties should be used in the design [58]. The relative (or non-dimensional) slenderness, however, should always be determined for the gross cross-section.

6.3.3 Design of joints and connections

The design of joints and connections should be based on Part 1-8 of EN 1993 using the specified partial factors γ_M. For local buckling verifications, e.g. column web in transverse compression for moment resisting joints, there is no need to update the partial factor γ_{M1}.

If steel elements that will be reused are connected by welding, it may be assumed that the weld material has the same strength as the base steelwork material [58]. However, it is recommended that the existing welds should be carefully inspected.

The steel grade of connecting plates can be considered the same as that of the base material of the structural elements which they are connected.

Best practice for connection design according to EN1993-1-8 can be found in references [83] and [84].

6.3.4 Design of the main structure

The previous sub-sections addressed the behaviour of individual members assuming both the loading and end conditions are known. The design of members in a structure naturally depends on how they are joined together and leads to the following types (i) simple construction, (ii) continuous construction, and (iii) semi-continuous construction.

In simple construction, the joints between the members are nominally pinned, so that they have small rotational stiffness and do not transmit moments. This allows all members to be designed essentially as simply supported.

In continuous construction, the joints are rotationally stiff and transmit substantial moments between members. In this case, the members can still be designed separately, provided that the internal forces are calculated taking into account the moments that are transferred among the members. This can be performed from a global elastic analysis.

Clause 7.2.2 of EN 1993-1-1:2022 permits all forms of geometrical and material imperfections in a second-order global analysis of frames. This approach requires specialist software and is rarely used in practice. Option b) of Clause 7.2.2(2) is the most likely choice, and it allows for separate treatment of all imperfections, and considers global, i.e. frame imperfections, in the global analysis, and local imperfections, in member checks. The details of the ways in which global imperfections of frames should be included are provided in Clause 7.3.2.

Permanent bracing systems are designed to resist:

- horizontal loads applied to the frame being braced,
- any loads applied directly to the bracing system, and
- effect of imperfections in the frames that it braces.

For design purposes, and in accordance with Clause 7.3 of EN 1993-1-1:2022, these imperfections are replaced with equivalent horizontal forces.

For bracing in the vertical plane, all three effects should be combined. Equivalent horizontal forces need to be considered for all ULS load combinations as their purpose is to represent the initial imperfect geometry which leads to deflections under the applied loading. These equivalent forces should be determined separately for each load combination as they depend on the magnitude of the design vertical loads.

Bracing systems to compression flanges are designed according to Clause 7.3.5 of EN 1993:2022. Imperfections are considered using one of the following methods: either by including an initial bow imperfection in the members to be restrained and designing for the additional moments, or by using an equivalent stabilising force. Where beam or compression members are spliced, there is an additional requirement that the bracing can resist an additional local force at the splice location, see Clause 7.3.5.2(2).

There is no specific guidance to the design of temporary or erection bracing in EN 1993. These systems ensure that the structure can be safely constructed. They depend on the construction sequence and should be located to reduce the cumulative tolerance errors.

6.3.5 Design of secondary structural elements

Secondary steelwork typically takes the form of cold formed purlins that span between the roof beams (rafters) and side rails than span between the columns. These elements support the cladding and are designed for wind loads, and roofs for snow. The purlins and side rails are also often used to provide restraint to the beams and columns and to transfer horizontal loads to the bracing system.

Section 11 of EN 1993-1-3 provides guidance on the design of purlins and side rails. Because these elements are usually proprietary, manufacturers have been developed and tested suitable sections, providing design data in the form of design tables or software.

6.4 Seismic design considerations

Designers should note that the seismic design of single-storey buildings do not usually require special design consideration according to Section 6 of prEN1998-1-2 [85]. Singlestory buildings are generally treated as low-ductility class (DC1), which means that the design requirements from EN1993-1-1 are sufficient. The trade-off in this practice is that a smaller behaviour factor needs to be considered when assessing the design seismic action. However, as single storey buildings have a low mass, the seismic action does not usually govern the design. If a DC1 concept is assumed in the design, there is no concern in utilising the reclaimed steel elements for structures subjected to seismic action, but a behaviour factor q=1 is recommended in the design of the reclaimed steelwork.

The recommendations provided in this guide may be adapted for other structures, such as multi-storey buildings, for which the seismic action has other significance (presence of higher masses, building height, dissipative zones). Structures designed in accordance with the dissipative structural behaviour concept shall belong to structural ductility classes DC2 or DC3. These classes correspond to an increased ability of the structure to dissipate energy in plastic mechanisms. Depending on the ductility class, specific requirements on the class of cross-sections and rotational capacity of the connections shall be met. 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 columns 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.

The assessment and testing procedures proposed in Appendix A, are in agreement with the requirements proposed in EN1998-3 [86] for existing buildings. For the cases where an assessment of an existing structure is undertaken and a dissipative behaviour (medium or high ductility class) is required, the recommended testing procedures should follow the recommendations for a CC3 structure according to Appendix A. Further guidance can be found in references [87] to [89].

6.5 Serviceability limit states

6.5.1 Deflections and displacements

Serviceability limit state conditions (deflections, displacements, vibrations) are presented in Clauses 5.4, 8.4 and Annex A of EN1990:2023 [9]. Serviceability criteria and limits are given in Annex A of EN1990:2023 and can be used for designing structures using reclaimed steel members. Serviceability requirements should be specified individually for each project. Additional provisions related to serviceability criteria are given in the other Eurocodes. Other maximum values should be as specified by the relevant authority or, where not specified, may be agreed for a specific project by the relevant parties. Elastic analysis is used to determine the deflections of the frame in the serviceability limit state.

In the following, some other conditions are discussed that may influence the serviceability criteria.

Cladding

Differential deflection between adjacent frames needs to be limited to prevent the fixings of the cladding elements from becoming overstrained, resulting in tearing and possible leakage. For example, cladding sheets in single-storey buildings deflect significantly less than the deformations calculated for the bare frame. This is due to the sheeting acting as a stressed skin diaphragm, which provides a considerable stiffening effect on the structure.

The actual deflection depends on the building proportions and cladding type, but reductions in horizontal deflections of more than 50% (from those calculated for the bare frame) are typically measured on as-built structures.

Gables

A sheeted and/or braced gable frame is very stiff in its own plane. The calculated differential deflections between the end frame and the adjacent frame can be very high. This differential deflection will always be modified by the presence of the building floor, roof sheeting and roof bracing.

Masonry

When brick or blockwork side walls are constructed in such a way they receive support from the steel frame, they should be detailed to allow them to deflect with the frame by using a compressible damp proof course at the base of the wall. Suitable restraint should be provided at the top of the brickwork panel and at intermediate points, if necessary. If the brickwork is continued around the steel columns, forming stiff piers, it is unreasonable to expect the panels to deflect with the frame. In this case, more onerous deflection limits should be applied to the frame.

Ponding

On low slope pitched roofs and flat roofs, the possibility of ponding of water on the roof should also be considered. The minimum recommended roof slope is 3^o after taking into account the vertical deflection. The recommended standard slope is 6^o to the horizontal, for which pounding can be disregarded. Trussed rafters typically have a slope of 3^o to the horizontal, but as they are much stiffer than solution with hot rolled/fabricated profiles, ponding effects are not critical.

REFERENCES

[1] Kibert CJ (2013). Sustainable construction: green building design and delivery: green building design and Delivery. John Wiley & Sons.

[2] CEN – European Committee for Standardisation (2024). CEN/TS 1090-201: Execution of steel structures and aluminium structures - Reuse of structural steel, Brussels, Belgium.

[3] Densley Tingley D, Allwood J (2014). Reuse of structural steel: the opportunities and challenges. In: European Steel Environment & Energy Congress 2014, 15-17 September, Teeside University, UK.

[4] Chen H-M, Wang Y, Zhou K, Lam D, Guo W, Li L, Ajayebi A, Hopkinson P (2022). Reclaiming structural steels from the end of service life composite structures for reuse - An assessment of the viability of different methods. Developments in the Built Environment 10, 100077.

[5] Yrjölä J. (2022). New white paper: Dismount and reuse of precast concrete structures, Peikko Group. Retrieved from: https://www.peikko.com/blog/new-white-paper-dismount-and-reuse-of-precast-concrete-structures/.

[6] ReCreate Project. (2023). Reusing concrete building components - A practical example. [Video]. YouTube. https://www.youtube.com/watch?v=EzppFH_Fg4w.

[7] CEN – European Committee for Standardisation (2009). EN 1090-1: Execution of steel structures and aluminium structures, Part 1: Requirements for conformity assessment of structural components (incorporating CEN amendment A1:2012), Brussels, Belgium.

[8] CEN – European Committee for Standardisation (2018). EN 1090-2: Execution of steel structures and aluminium structures, Part 2: Technical requirements for steel structures (incorporating CEN amendment A1:2024), Brussels, Belgium.

[9] CEN – European Committee for Standardisation (2023). EN 1990: Eurocode: Basis of structural and geotechnical design, Brussels, Belgium.

[10] CEN – European Committee for Standardisation (2002). EN 1991-1-1: Eurocode 1: Actions on structures, Part 1-1: General actions – Densities, self-weight, imposed loads for buildings (incorporating CEN corrigenda Dec. 2004 and Mar. 2009), Brussels, Belgium.

[11] CEN – European Committee for Standardisation (2002). EN 1991-1-3: Eurocode 1: Actions on structures, Part 1-3: General actions – snow loads (incorporating CEN corrigenda Dec. 2004 and Jun. 2009, and CEN amendment A1:2015), Brussels, Belgium.

[12] CEN – European Committee for Standardisation (2002). EN 1991-1-4: Eurocode 1: Actions on structures, Part 1-4: General actions – wind actions (incorporating CEN amendment A1:2010), Brussels, Belgium.

[13] CEN – European Committee for Standardisation (2003). EN 1991-1-5: Eurocode 1. Actions on structures, Part 1-5: General actions - Thermal actions, Brussels, Belgium.

[14] CEN – European Committee for Standardisation (2005). EN 1991-1-6: Eurocode 1. Actions on structures, Part 1-6: General actions - Actions during execution, Brussels, Belgium.

[15] CEN – European Committee for Standardisation (2022). EN 1993-1-1: Eurocode 3: Design of steel structures, Part 1-1: General rules and rules for buildings, Brussels, Belgium.

[16] CEN – European Committee for Standardisation (2024). EN 1993-1-3: Eurocode 3: Design of steel structures, Part 1-3: General rules – supplementary rules for cold formed members and sheeting, Brussels, Belgium.

[17] CEN – European Committee for Standardisation (2024). EN 1993-1-8: Eurocode 3: Design of steel structures, Part 1-8: Design of joints, Brussels, Belgium.

[18] CEN – European Committee for Standardisation (2005). EN 1993-1-10: Eurocode 3: Design of steel structures, Part 1-10: Material toughness and through-thickness properties (incorporating CEN corrigenda Dec. 2005, Sep. 2006 and Mar. 2009), Brussels, Belgium.

[19] CEN – European Committee for Standardisation (2004). EN 1994-1-1: Eurocode 4: Design of composite steel and concrete structures - General rules and rules for buildings, Brussels, Belgium.

[20] CEN – European Committee for Standardisation (2004). EN 10025-1: Hot rolled products of structural steels, Part 1: General technical delivery conditions, Brussels, Belgium.

[21] CEN – European Committee for Standardisation (2019). EN 10025-2: Hot rolled products of structural steels, Part 2: Technical delivery conditions for non-alloy structural steels, Brussels, Belgium.

[22] CEN – European Committee for Standardisation (2019). EN 10025-4: Hot rolled products of structural steels, Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels (incorporating CEN amendment A1:2022), Brussels, Belgium.

[23] CEN – European Committee for Standardisation (2004). EN 10025-5: Hot rolled products of structural steels, Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance, Brussels, Belgium.

[24] CEN – European Committee for Standardisation (2010). EN 10029: Hot rolled steel plates 3 mm thick or above, Tolerances on dimensions and shape, Brussels, Belgium.

[25] CEN – European Committee for Standardisation (1993). EN 10034: Structural steel I and H sections – Tolerances on shape and dimensions, Brussels, Belgium.

[26] CEN – European Committee for Standardisation (2024). EN 10051: Continuously hot rolled strip and plate/sheet cut from wide strip of non-alloy and alloy steels – Tolerances on shape and dimensions, Brussels, Belgium.

[27] CEN – European Committee for Standardisation (1998). EN 10055: Hot rolled steel equal flange tees with radiused root and toes – Dimensions and tolerances on shape and dimensions, Brussels, Belgium.

[28] CEN – European Committee for Standardisation (2017). EN 10056-1: Structural steel equal and unequal leg angles, Part 1: Dimensions, Brussels, Belgium.

[29] CEN – European Committee for Standardisation (1993). EN 10056-2: Structural steel equal and unequal leg angles, Part 2: Tolerances on shape and dimensions, Brussels, Belgium.

[30] CEN – European Committee for Standardisation (2004). EN 10204: Metallic products – Types of inspection documents, Brussels, Belgium.

[31] CEN – European Committee for Standardisation (2006). EN 10210-1: Hot finished structural hollow sections of non-alloy and fine grain steels, Part 1: Technical delivery requirements, Brussels, Belgium.

[32] CEN – European Committee for Standardisation (2019). EN 10210-2: Hot finished structural hollow sections of non-alloy and fine grain steels, Part 2: Tolerances, dimensions and sectional properties, Brussels, Belgium.

[33] CEN – European Committee for Standardisation (2006). EN 10219-1: Cold formed welded structural hollow sections of non-alloy and fine grain steels, Part 1: Technical delivery requirements, Brussels, Belgium.

[34] CEN – European Committee for Standardisation (2019). EN 10219-2: Cold formed welded structural hollow sections of non-alloy and fine grain steels, Part 2: Tolerances, dimensions and sectional properties, Brussels, Belgium.

[35] CEN – European Committee for Standardisation (2000). EN 10279: Hot rolled steel channels – Tolerances on shape, dimension and mass, Brussels, Belgium.

[36] CEN – European Committee for Standardisation (2015). EN 10346: Continuously hot-dip coated steel flat products for cold forming – Technical delivery conditions, Brussels, Belgium.

[37] CEN – European Committee for Standardisation (2022). EN 10169: Continuously organic coated (coil coated) steel flat products - Technical delivery conditions, Brussels, Belgium.

[38] CEN – European Committee for Standardisation (2017). EN 10365 Hot rolled steel channels, I and H sections – Dimension and masses, Brussels, Belgium.

[39] CEN – European Committee for Standardisation. EN 14399: High-strength structural bolting assemblies for preloading (all parts), Brussels, Belgium.

[40] CEN – European Committee for Standardisation (2013). EN 14509: Self-supporting double skin metal faced insulating panels, Factory made products, Specifications, Brussels, Belgium.

[41] Steel buildings in Europe: Design guides for single-storey and multi-storey steel buildings. https://constructalia.arcelormittal.com/en/news_center/articles/design_guides_steel_buildings_in_europe.

[42] Kamrath, P., Sansom, M., Ungureanu, V., & Hradil, P. (2020). Deliverable D2.1a: Safe and efficient deconstruction; Deliverable D2.1a: Deconstruction protocol for single-storey steel framed buildings. European Convention for Constructional Steelwork. https://www.steelconstruct.com/wp-content/uploads/PROGRESS-D2.1-Auditing-and-deconstruction-process.pdf.

[43] Single storey industrial buildings. https://steelconstruction.info/Single_storey_industrial_buildings.

[44] Multi-storey office buildings. https://steelconstruction.info/Multi-storey_office_buildings.

[45] CEN – European Committee for Standardisation (2019). EN 15804: Sustainability of construction works – Environmental product declarations – Core rules for the product category of construction products (incorporating CEN amendment A2:2019), Brussels, Belgium.

[46] CEN – European Committee for Standardisation (2011). EN 15978: Sustainability of construction works. Assessment of environmental performance of buildings. Calculation method, Brussels, Belgium.

[47] CEN – European Committee for Standardisation (2015). EN 16627: Sustainability of construction works. Assessment of environmental performance of buildings. Calculation method, Brussels, Belgium.

[48] CEN – European Committee for Standardisation (1992). ENV 1993-1-1: Eurocode 3: Design of steel structures, Part 1-1: General rules and rules for buildings, Brussels, Belgium.

[49] European Union (2011). Regulation (EC) No 305/2011 of the European Parliament and of the Council of 9 March 2011 laying down harmonised conditions for the marketing of construction products and repealing Council Directive 89/106/EECWFD 2008/98/EC. Available at: http://data.europa.eu/eli/reg/2011/305/oj.

[50] European Commission: Directorate-General for Internal Market, Industry, Entrepreneurship and SMEs, Oberender, A., Fruergaard Astrup, T., Frydkjær Witte, S., Camboni, M., Chiabrando, F., Hayleck, M., & Akelytė, R. (2024). EU construction & demolition waste management protocol including guidelines for pre-demolition and pre-renovation audits of construction works: updated edition 2024, Publications Office of the European Union. https://data.europa.eu/doi/10.2873/77980.

[51] ECCS P49 (1987). European recommendations for design of light gauge steel members, Publication P049, European Convention for Constructional Steelwork, Brussels, Belgium.

[52] CEN – European Committee for Standardisation (1996). ENV1993-1-3: Eurocode 3: Design of Steel Structures Part 1.3. General Rules. Supplementary rules for cold formed thin gauge members and sheeting. (including the Corrigenda to ENV1993-1-3 of 1997-02-25). European Committee for Standardization, Brussels, Belgium.

[53] Feldmann M et al. (2024). Guidance on Establishing European Rules for the Design of reclaimed Steel Components for Reuse (4th draft), CEN-TC 250-SC 3-AHG Reuse.

[54] Girao Coelho AM, Pimentel R, Ungureanu V, Hradil P, Kesti J (2020). European Recommendations for Reuse of Steel Products in Single-Storey Buildings, 1st Edition, https://www.steelconstruct.com/.

[55] CEN – European Committee for Standardisation (2017). CEN ISO/TR 15608: Welding - Guidelines for a metallic materials grouping system, Brussels, Belgium.

[56] BSI – British Standards Institution. BS 15:1948. Structural steel, UK.

[57] BSI – British Standards Institution. BS 4360-2:1969. Specification for weldable structural steels metric units, UK.

[58] SIA 269/3:2011. Existing structures – Steel structures, SIA Zurich.

[59] Brown D. G. and Iles, D. C. Selection of steel sub-grade in accordance with the Eurocodes. SCI Document ED007. SCI, 2012.

[60] PD 6695-1-10:2009. Recommendations for the design of structures to BS EN 1993-1-10. BSI, 2009.

[61] P419 – Brittle fracture: selection of steel sub-grade to BS EN 1993-1-10; ISBN 1-85942-135-0, 2017. The Steel Construction Institute.

[62] Sedlacek, G. et al. Commentary and worked examples to BS EN 1993-1-10 “Material toughness and through thickness properties” and other toughness-oriented rules in BS EN 1993. European Commission Joint Research Centre, 2008.

[63] Davison B, Owens GW (eds.) (2012). Steel designers’ manual. The Steel Construction Institute, 7th edition, Wiley-Blackwell, UK.

[64] CEN – European Committee for Standardisation (2007). EN ISO 8501-1: Preparation of steel substrates before application of paints and related products – Visual assessment of surface cleanliness, Part 1: Rust grades and preparation grades of uncoated steel substrates and of steel substrates after overall removal of previous coatings, Brussels, Belgium.

[65] BS 4 Part 1;1962. Specification for structural steel section; Part 1: Hot Rolled sections.

[66] Dorman Long & CO Ltd (1964). Handbook for constructional engineers, Fanfare Press Ltd, London, UK.

[67] The British Constructional Steelwork Association (1994). National structural steelwork specification for building construction, 3th Edition, 203, London, UK.

[68] STAS 767/0-88 (1988). Steel Structures. General technical requirements for quality of non-industrial, industrial and agricultural buildings. Romanian Institute for Standardization.

[69] CEN – European Committee for Standardisation (2016). EN ISO 17637: Non-destructive testing of welds – Visual testing of fusion-welded joints, Brussels, Belgium.

[70] CEN – European Committee for Standardization (2020). EN ISO 7438: Metallic materials – Bend test, Brussels, Belgium.

[71] CEN – European Committee for Standardisation (2018). EN 1090-4: Execution of steel structures and aluminium structures, Part 4: Technical requirements for cold formed structural steel elements and cold formed structures for roof, ceiling, floor and wall applications, Brussels, Belgium.

[72] CEN – European Committee for Standardisation (2003). EN 10162: Cold rolled steel sections –Technical delivery conditions – Dimensional and cross-sectional tolerances, Brussels, Belgium.

[73] CEN – European Committee for Standardisation (2006). EN 10143: Continuously hot-dip coated steel sheet and strip – Tolerances on dimensions and shape, Brussels, Belgium.

[74] CEN – European Committee for Standardisation (2013). EN 14509: Self-supporting double skin metal faced insulating panels. Factory made products. Specifications, Brussels, Belgium.

[75] CEN – European Committee for Standardisation (2006). EN 14782: Self-supporting metal sheet for roofing, external cladding and internal lining. Product specification and requirements, Brussels, Belgium.

[76] International Standard (2010). ISO 13822: Bases for design of structures – Assessment of existing structures, Geneva, Switzerland.

[77] Gulvanessian H, Calgar JA, Holický M (2012). Designer’s guide to Eurocode: basis of structural design EN 1990, ICE Publishing, Thomas Telford, London (2nd edition).

[78] Steenbergen RDJM, Vrouwenvelder ACWM (2010). Safety philosophy for existing structures and partial factors for traffic loads on bridges, Heron 55(2), 123-140.

[79] Matthews S (2012). Structural appraisal of existing buildings, including for a material change of use, Part 3: Structural appraisal procedures, BRE-DG 366, Building Research Establishment, UK.

[80] Sedlacek G, Spangemacher R, Hensen W, ARBED Research (1989). Background Documentation for EC3, Doc. 5.01, Background document for the justification of safety factor γ_M = 1.0 for rolled beams in bending about the strong axis. Commission of the European Communities.

[81] Sedlacek G, Ungermann D, Kuck J, Maquoi R, Janss J (1989). Background Document for EC3, Doc. 5.03, Evaluation of test results on beams with cross-sectional classes 1-3 in order to obtain strength functions and suitable model factors. Commission of the European Communities.

[82] Charbrolin B, CTICM, Labein, ProfilARBED, RWTH, SCI, TNO, SAES (2002). Partial safety factors for resistance of steel elements to EC3 and EC4. Calibration for various steels products and failure criteria. Final report. Technical steel research series, European Communities, EUR 20344 EN.

[83] SCI P358, Joints in steel construction: Simple joints to Eurocode 3, 2014. The Steel Construction Institute.

[84] SCI P398 Joints in steel construction: Moment-resisting joints to Eurocode 3. The Steel Construction Institute.

[85] CEN – European Committee for Standardisation (2023). prEN 1998-1-2: Eurocode 8 – Design of structures for earthquake resistance - Part 1-2: Buildings, Brussels, Belgium.

[86] CEN – European Committee for Standardisation (2005). EN 1998-3: Eurocode 8. Design of structures for earthquake resistance. Assessment and retrofitting of buildings, Brussels, Belgium.

[87] AISC – American Institute of Steel Construction (2022). ANSI/AISC 342:2022: Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings, Chicago, IL, USA.

[88] ASCE – American Society of Civil Engineers (2017). ASCE/SEI 41-17: Seismic Evaluation and Retrofit of Existing Buildings, Reston, Virginia, USA.

[89] FEMA P-2208:2023: NEHRP Recommended Revisions to ASCE/SEI 41-17, Seismic Evaluation and Retrofit of Existing Buildings, USA.

[90] CEN – European Committee for Standardisation (2007). EN 1993-6:2007 Eurocode 3. Design of steel structures. Crane supporting structures, Brussels, Belgium.

[91] CEN – European Committee for Standardisation (2016). EN 13018: Non-destructive testing, Visual testing, General principles, Brussels, Belgium.

[92] CEN – European Committee for Standardisation (2011). EN ISO 13385-1: Geometrical product specifications (GPS), Dimensional measuring equipment, Part 1: Callipers; Design and metrological characteristics (incorporating corrigendum Oct. 2015), Brussels, Belgium.

[93] CEN – European Committee for Standardisation (2011). EN ISO 13385-2: Geometrical product specifications (GPS), Dimensional measuring equipment, Part 2: Calliper depth gauges; Design and metrological characteristics (incorporating corrigendum Oct. 2015), Brussels, Belgium.

[94] CEN – European Committee for Standardisation (2013). EN ISO 3452-1: Non-destructive testing, Penetrant testing, Part 1: General principles (incorporating corrigendum Jun. 2014), Brussels, Belgium.

[95] CEN – European Committee for Standardisation (2010). EN ISO 15549: Non-destructive testing – Eddy current testing – General principles, Brussels, Belgium.

[96] CEN – European Committee for Standardisation (2015). EN ISO 17643: Non-destructive testing of welds – Eddy current examination of welds by complex plane analysis, Brussels, Belgium.

[97] CEN – European Committee for Standardisation (2014). EN ISO 16810: Non-destructive testing, Ultrasonic testing, General principles, Brussels, Belgium.

[98] CEN – European Committee for Standardisation (2018). EN ISO 17640: Non-destructive testing of welds, Ultrasonic testing, Techniques, testing levels, and assessment, Brussels, Belgium.

[99] CEN – European Committee for Standardisation (2016). EN ISO 17638: Non-destructive testing of welds. Magnetic particle testing, Brussels, Belgium.

[100] CEN – European Committee for Standardisation (2015). EN ISO 23277: Non-destructive testing of welds, Penetrant testing, Acceptance levels, Brussels, Belgium.

[101] CEN – European Committee for Standardisation. EN ISO 17636: Non-destructive testing of welds. Radiographic testing. X- and gamma-ray techniques (all parts), Brussels, Belgium.

[102] FEMA 352:2000. Recommended Post earthquake Evaluation and Repair Criteria for Welded. Steel Moment-Frame Buildings; Federal Emergency Management Agency USA.

[103] CEN – European Committee for Standardisation. EN ISO 6507: Metallic materials. Vickers hardness test (all parts), Brussels, Belgium.

[104] CEN – European Committee for Standardisation. EN ISO 6508: Metallic materials. Rockwell hardness test (all parts), Brussels, Belgium.

[105] CEN – European Committee for Standardisation – EN ISO 6505: Metallic materials. Brinell hardness test (all parts), Brussels, Belgium.

[106] ASTM – American Society for Testing and Materials (2017). A1038: Standard test method for portable hardness testing by the ultrasonic contact impedance method. West Conshohocken, Pennsylvania, United States.

[107] ISO 19272:2015; Low alloyed steel – Determination of C, Si, Mn, P, S, Cr, Ni, Al, Ti and Cu – Glow discharge optical emission spectrometry (routine method), 2015.

[108] ASTM – American Society for Testing and Materials (2013). E572: Standard test method for analysis of stainless and alloy steels by wavelength dispersive X-ray fluorescence spectrometry. West Conshohocken, Pennsylvania, United States.

[109] ASTM – American Society for Testing and Materials (2014). E1476: Standard guide for metals identification, grade verification, and sorting, West Conshohocken, Pennsylvania, United States.

[110] ISO 145775 (all parts). Metallic materials – Instrumented indentation test for hardness and materials parameters.

[111] CEN – European Committee for Standardisation (2007). CEN Workshop Agreement CWA 15627: Small punch test method for metallic materials, Brussels, Belgium.

[112] CEN – European Committee for Standardisation (2017). prEN 15627: Metallic materials - Small punch test method, working document, ECISS/TC 1010/WG 1 committee.

[113] CEN – European Committee for Standardisation (2020). EN ISO 6892-1: Metallic materials – Tensile testing, Part 1: Method of test at room temperature, Brussels, Belgium.

[114] CEN – European Committee for Standardisation (2002). EN ISO 14284: Steel and iron – sampling and preparation of samples for the determination of chemical composition (incorporating corrigendum Dec. 2002), Brussels, Belgium.

[115] CEN – European Committee for Standardisation (2016). EN ISO 148-1: Metallic materials – Charpy pendulum impact test, Part 1: Test method, Brussels, Belgium.

[116] ASTM – American Society for Testing and Materials (2013). E112 Standard test methods for determining average grain size. West Conshohocken, Pennsylvania, United States.

[117] SCI P427 – Structural Steel Reuse: assessment, testing and design principles, The Steel Construction Institute, 2019.

[118] Simões da Silva L, Marques L, Tankova T, Rebelo C, Kuhlmann U, Kleiner A, Spiegler J, Snider HH, Dekker RWA, Dehan V, Taras A, Haremza C, Cajot LG, Vassart O, Popa N (2017). Standardisation of safety assessment procedures across brittle to ductile failure modes (SAFEBRICTILE). Research Fund for Coal and Steel (RFCS), Final report EUR 28906, European Commission, Brussels, Belgium.

[119] Fujita M, Kuki K (2016). An evaluation of mechanical properties with the hardness of building steel structural members for reuse by NDT, Metals 6, 247, doi: 10.3390/met6100247.

[120] CEN – European Committee for Standardisation (2013). EN ISO 18265: Metallic materials – Conversion of hardness materials, Brussels, Belgium.

[121] CEN – European Committee for Standardisation (2017). EN ISO 377: Steel and steel products. Location and preparation of samples and test pieces for mechanical testing, Brussels, Belgium.

[122] CEN – European Committee for Standardisation (2017). EN 13523-1: Coil coated metals. Test methods. Film thickness, Brussels, Belgium.

[123] CEN – European Committee for Standardisation (2002). EN 13523-6: Coil coated metals. Test methods. Adhesion after indentation, Brussels, Belgium.

[124] CEN – European Committee for Standardisation (2014). EN 13523-7: Coil coated metals. Test methods. Resistance to cracking on bending (T-bend test), Brussels, Belgium.

[125] Holický M (2009). Reliability analysis for structural design. Sun Press.

Appendix A
Assessment, measurements, sampling, and testing

A.1 General

Quantification of the material properties and verification of the structure, or structural components assessment, are necessary to evaluate the reusability. Testing programmes will include a range of tests and should be carefully undertaken. A balance must be achieved between obtaining enough information to make a reasonable judgement on risk, and whether intrusive sampling damages the structure itself, as follows:

- Tests may be effective and may be interpreted in combination, e.g., a representative sample of locations revealing a particular characteristic may be examined in greater detail by a variety of more detailed tests,
- Performance evaluation procedures using Non-Destructive Testing (NDT) should prevail, if possible,
- In the case of Destructive Testing (DT), drilling or cutting must be carefully located, specified and supervised to avoid potential damages to the structure,
- Where welding is foreseen, the chemical composition should be determined, so that the welding procedure specification can be established. The suitability of the base metal for welding has to be demonstrated by the Carbon Equivalent Value (CEV).

A.2 Condition assessment and measurements

A.2.1 General

The assessment of the existing steelwork is described in Volume 3. The flowchart in Fig. 5.1 illustrates the overall framework for reclaiming existing structural steel elements. The necessity to assess the fabrication processes of existing steelwork (mainly welds) was also highlighted, to ensure that these processes are in accordance with the quality requirements according to EN1090. The following section provides further guidance for the evaluation of existing steelwork as well as for inspection of welds.

A.2.2 Inspection techniques

The inspection techniques appropriate to the current project are summarised in Table A.1. These very simple techniques will assist in the determination of the general condition of the structure and the definition of a suitable sampling and testing procedure. In practice, this is combined with detailed measurements. The following information can be gathered:

- The age of the structure and possible modifications or repairs,
- The materials of which the structure is made (or were added later on),
- The geometry and structural configuration of the building, size of members and details of the joints.

In the case when the entire primary structure is reused, the inspection of the building includes further details. The dimensions of the components at critical locations should be measured. Dimensions of joints and connectors should be recorded, including weld sizes. Inspection of all welds needs to be carried out. Additionally, because buckling resistance is affected by geometric imperfections, detailed measurement of deviations should be made according to EN 1090-2.

Table A.1 Inspection techniques

Technique	**Description**	**Comments/Value**
Visual inspection	Examination for corrosion, cracks, deformities, damage, etc.	Essential. General assessment of the physical condition of the structure. Will not reveal fine or subsurface cracks. General provisions are given in EN 13018 [91].
Field survey	Geometrical survey of positions and sizes of members and details.	Essential in absence of drawings, and to (i) check for modifications and repairs, (ii) determine the cross-section dimensions, straightness, verticality, deformation and deflection of members.
Dimensional inspection	Measurements using Vernier callipers, micrometres, three-dimensional laser scanning, ultrasonic measurements, etc.	Essential in absence of original structural drawings. Geometric data collection, size of members. For equipment and tools see e.g. EN ISO 13385-1 [92] and EN ISO 13385-2 [93].

A.2.3 Non-destructive testing of welds

NDT is generally carried out by operating equipment close to, against or fixed to the surface of the structure, and has major advantages, namely it does not damage the structure and also eliminates the need for time-consuming random sampling, and subsequent laboratory testing. Table A.2 sets out some of the techniques that can be used during this examination phase. NDT techniques can be useful to locate and/or measure the size of defects.

Table A.2 Potential NDT techniques for welds

Technique	**Description**	**Comments/Value**
Visual inspection	Covers the visual examination of fusion welds in metallic materials. The examination is normally performed on welds in the as-welded condition but exceptionally, the examination may be carried out at other stages during the welding process.	Ensures minimum quality control for every welded connection. EN ISO 17637 [69].
Penetrant testing	Dye highlights surface breaking cracks.	Indicates surface cracks in members not otherwise visible to the naked eye, approximately 25 μm. Surface defects may be accurately detected. EN ISO 3452-1 [94] gives the general principles for this technique. For welds, see EN ISO 23277 [100].

Eddy current welding inspection	Eddy current methods are used for non-destructively locating and characterising discontinuities in magnetic or nonmagnetic electrically conducting materials.	Essential to detect surface and near-surface cracks. Only applicable to simple geometries. Will not detect sub-surface embedded defects. General principles are given in EN ISO 15549 [95] and for welds see EN ISO 17643 [96].
Ultrasonic testing	Transducer converts electrical energy into ultra-high frequency sound waves which are reflected by defects and recorded.	Suitable for detecting embedded planar defects, including cracks, lack of fusion of welds, lamellar tearing, hydrogen cracking. General provisions for this technique can be found in EN ISO 16810 [97]. For welds see EN ISO 17640 [98].
Magnetic particle testing	Magnetic particle testing uses magnetisation of questionable cross sections in electrically conductive materials. For visualization of the magnetic field, a suspension usually with fluorescent steel splinters is used.	This inspection method can be used for detection of surface cracks in ferromagnetic materials only. Cracks in nonmagnetic material or in sandwiched elements cannot be detected. The method can be applied as quality control of precise setting of drilled holes to stop active fatigue cracks. EN 17638 [99] can be pointed out as a reference.
Radiographic inspection	Radiographic inspection (x-ray, γ-ray, e.g. with Iridium source) is applied to detect cracks and flaws in built-up sections to evaluate sandwiched members. The radiographic source is located on one side of the built-up element, the radiosensitive film, detector or digital storage unit on the other side of the inspected cross section.	The radiographic or γ-ray inspection is the only method with validated feasibility during laboratory tests and on-site for detection of internal failure or of cracks in the middle of sandwiched elements. EN ISO 17636 [101] can be pointed out as a reference.

A.2.4 Inspection protocol for welds

The welds between plates of fabricated members (including cellular beams) are to be inspected. The same amount of weld testing required by EN 1090-2 (see Table 24) should be applied to reclaimed steel elements. A visual inspection of 100% of the welds is mandatory. Table A.3 suggests a minimum number of welded connections to be inspected by non-destructive tests. A connection may have different weld segments. In a typical rafter-column connection, the welds between flanges and webs need to be assessed. Each of these welds may be assumed as one connection according to Table A.3.

Table A.3 Suggested minimum percentage of welds to be tested [2] [102]

Total number of connections	**Number of connections to be tested**	**Total %**
6	3 (minimum)	50%
10	4	40%
15	5	33%
20	6	30%
30	8	27%
40	10	25%

50	12	24%
75	16	21%
100	20	20%
200	30	15%
300	40	13%
500	60	12%
1000	100	10%
2000	150	8%

A.3 Definition of group of elements to be tested – test unit

Reclaimed steel members are to be considered as a group, provided that they come from the same source structure and meet the following requirements:

- Structural steel erected after 1970,
- Are of the same serial size,
- Same structural function, e.g. rafters, floor beams, columns, bracings, etc.,
- Identical detailing (length, connections, etc.),
- Local stiffeners are not considered as detrimental for grouping.

If steelwork originally manufactured to an alternative specification/product standard (other than the EN standards), is to be placed on the market, material manufactured to different product standards should not be mixed within a group – the source and manufacturing standard of all material in a group should be consistent.

A group should comprise a maximum weight of 20 tonnes. Several groups of 20 tonnes will be required if large numbers of the same member are reclaimed. Defining a group of elements to be tested in this manner allows certain material characteristics to be established for the group by testing one or more representative members from the group. For cold formed elements, a group should comprise a maximum weight of 4 tonnes.

In this protocol, the concept of a ‘group’ has a special significance, as outlined above. In product standards such as EN 10025-2 or EN 10346 section, a similar term is ‘test unit’, indicating a collection of steel products of a specified total maximum weight of the same form, grade and quality, and delivery condition. A ‘test unit’ can contain products of various thicknesses, whereas in this protocol, a ‘group’ is limited to members of the same serial size. In product standards, tests are specified to be undertaken from samples in the test unit; in this protocol, tests are specified to be undertaken from samples in the group of reclaimed elements.

A.4 Testing techniques for mechanical and chemical properties

A.4.1 General

There can be significant variability in the properties of steel in a building, even if all members and connecting elements comply with the same specifications and grades of material. It is only necessary to characterise the properties of material in a structure based on the likely statistical distributions with mean values and coefficients of variation. Knowledge of the material specification and grade to which a structural element complies, and its approximate age, will be sufficient to define these properties for nearly all evaluations.

If original construction documents are available, including drawings and specifications, it will typically not be necessary to perform material tests. When material properties are not clearly indicated on the drawings and specifications, or the drawings and specifications are not available, the material grades indicated in Table 5.7 may be assumed. Alternatively, a limited program of material sample removal and testing may be conducted to confirm the likely grades of these materials.

If sampling is performed, it should take place in regions of reduced stress to minimise the effects of the reduced area, such as the flange tips at the ends of simply supported beams, the flange edges in the mid-span region of members of moment-resisting frames, and the edges of the external plates.

A.4.2 Non-destructive and minimum invasive testing for material properties

Non-destructive hardness testing is suitable for estimating the ultimate tensile strength of the steel. Table A.4 summarises some of the alternative non-destructive techniques that can be used to assess the properties of reclaimed steel.

Table A.4 Potential NDT techniques

Technique	**Description**	**Comments/Value**
Hardness testing	Diameter of imprint measured when hardened steel ball is pressed against a smooth surface with known force.	Provides hardness number, e.g. Vickers according to ISO 6507 [103] hardness, which is a guide to yield and ultimate strength of the material. Vickers test method is stated on EN 1090-2. Other alternatives are Rockwell ISO 6508 [104] and Brinell ISO 6505 [105] test methods. See also ASTM 1038:2017 [106].
Positive metal identification	Uses X-ray Fluorescence and optical emission spectrometry to establish the metallic alloy composition, and grade identification by reading the quantities by percentage of its elements.	Essential for characterisation of weldability of steel structural members, as a function of the carbon equivalent. Provides additional information on the type and associated physical properties of steel and about its alloying materials. ISO 19272 [107]. See ASTM E572 [108] and ASTM 1476 [109].
Instrumented indentation testing	Instrumented indentation apparatus uses similar technique as hardness test with measured load and penetration in repeated loading and unloading cycles.	Output of the indentation test includes stress-strain relationship, elastic modulus, hardness and stiffness. See ISO 14577-5 [110].
Small punch testing	Small punch test uses ceramic ball pressed against the face of small circular specimen (diameter 8 mm, thickness 0.5 mm). The stress-strain relationship is then derived from the measured load versus ball displacement.	Calculation according to prEN 15627 [111], [112] can be used to predict yield and tensile strength of the steel. The equivalent stress-strain relationship of the tensile coupon may be obtained by more advanced Finite Element Modelling.

A.4.3 Destructive testing for material properties

Destructive testing (DT) techniques require the extraction of small samples from the existing structure. Potential DT techniques are identified in the table below (see Table A.5). The samples for testing are extracted by cutting or drilling. It is important to consider the likely value of the test results in relation to possible damage to the structure, e.g. embrittlement following heating when the sample is removed by flame cutting, and whether indirect methods might be more appropriate. Mechanical and metallurgical properties can usually be established by laboratory testing on the same sample. Information about extracting steel samples can be found in relevant standards, e.g., for steel see EN 10025.

Table A.5 Potential Destructive Testing (DT) techniques

Technique	Description	Comments/Value
Tensile testing	Tensile tests on meaningful samples providing yield and ultimate tensile strength, modulus of elasticity, uniform elongation, and elongation at failure.	In the absence of material certificates. For test details see EN ISO 6892-1 [113].
Chemical composition analysis	Testing for carbon, silicon, manganese, sulphur, and phosphorus.	Essential for material identification and to check the weldability of the steel as a function of the carbon equivalent, as well as the impurity levels. Tests are carried out on drilling swarf or scrapings. It provides further information on the type and associated physical properties of steel. See EN ISO 14284 [114].
Charpy impact test	Brittleness and notch ductility at a range of temperatures determined by measuring the energy required to fracture a standard U- or V-notched sample with a blow from a pendulum.	Allows characterisation of the steel sub-grade when material certificates are not available. For test details see EN ISO 148-1 [115]. Impact toughness can be also tested on sub-sized specimen and the results recalculated to match the behaviour of the full-sized tests.
Metallography	Determination of the average grain size	Determination of internal structure of the material by microscopic examination of a sample with one flat surface. See ASTM E 112 [116].

A.5 Testing Protocols

Four testing protocols, A to D, are described in CEN / TS 1090-201 [2]. Recommendations for the choice of a specific protocol are given below and illustrated in Fig. 5.3. If the donor structure was executed according to EN 1090-2 and the grades and/or qualities of the structural components can be identified, original inspection documents according to EN 10204 may be used to declare the properties of the component, see testing protocol A in Section 5.3.4.3 of CEN / TS 1090-201 [2]. Structural components which have a known provenance should be grouped in test units according to Section 5.3.4.2 of CEN / TS 1090-201 [2].

The provenance should be considered as known when at least the geographical location, building year and former function of the components are known. The measurements undertaken for one or a few representative members of a test unit may be the basis of the declared properties for all members of that test unit (see Sections 5.3.4.4 and 5.3.4.5 of CEN / TS 1090-201 [2]).

A distinction should be made between type 1 and type 2 structural steel.

Type 1 structural steel may be expected to have mechanical properties and weldability similar to steel grades according to European standards listed in Clause 5.3 of EN 1090-2:2018+A1:2024. The variability of their mechanical properties may be assumed to be according to Annex E of EN 1993-1-1.

The mechanical properties of type 1 structural steel may be determined by testing a single representative sample according to testing protocol B in Section 5.3.4.4 of CEN / TS 1090-201 [2].

For type 2 structural steel, the variability of the properties cannot be reliably assumed, and more tests should be undertaken as well as a statistical analysis of the results, according to protocol C in 5.3.4.5 of CEN / TS 1090-201 [2].

Structural components of unknown provenance shall not be grouped in test units and comprehensive testing is required, according to protocol D in 5.3.4.6 of CEN / TS 1090-201 [2].

An alternative testing approach for yield strength, ultimate strength, elongation and chemical composition to Consequence class is presented in [117].

A.6 Comprehensive testing implementation for strength and elongation

A.6.1 Introduction

Material strength and elongation can be assessed by both destructive and non-destructive tests. In the following section guidance is provided on both types of testing.

A.6.2 Reliability assessment – hot rolled and hollow section products

The results of non-destructive and destructive tests shall be compared with the minimum values presented in Table A.6 in order to determine the steel grade. Minimum values are established by reducing the mean value by 1.64 times the standard deviation for each steel grade based on the data from Table A.7.

Table A.6 Recommended minimum values for yield and tensile strength to undertake the reliability assessment of hot rolled and hollow section products

Steel grade	**Yield strength (N/mm^2)**			**Ultimate strength(N/mm^2)**			**f_u/f_y mean**	**Reference standard**
	f_y *Design*	Min.	Mean	f_u *Design*	Min.	Mean		
S235	235	267	293	360	397	432	1.47	EN 10025-2; EN 10219
S275	275	313	343	410	452	492	1.43	EN 10025-2; EN 10219
S355	355	391	426	470	505	540	1.26	EN 10025-2; EN 10219
S460	460	490	529	540	560	594	1.12	EN 10025-3/4; EN 10219

Table A.7 Steel properties data according to reference [118]

Steel grade	**Yield Strength**		**Tensile Strength**	
	Mean (X characteristic value)	CoV	**Mean** (X characteristic value)	CoV
S235	1.25	0.055	1.20	0.050
S275	1.25	0.055	1.20	0.050
S355	1.20	0.050	1.15	0.040
S460	1.15	0.045	1.10	0.035

A.6.3 Reliability assessment – cold formed products

The results of non-destructive and destructive tests shall be compared with the minimum values presented in Table A.8 in order to determine the steel grade. Minimum values are established by reducing the mean value by 1.64 times the standard deviation for each steel grade based on the data from Table A.7.

As buckling curves do not depend of the yield strength for cold formed steel elements according to EN 1993-1-3, average values for mean strength (yield and tensile) and coefficient of variation are proposed for all steel grades up to S450 in Table A.9.

Table A.8 Recommended minimum values for yield and tensile strength to undertake the reliability assessment of cold formed steel products

Steel Grade	**Yield Strength [N/mm^2]**			**Tensile Strength [N/mm^2]**			**f_u/f_y mean**	**Reference standard**
	f_y Design	Min.	Mean	f_u Design	Min.	Mean		
S220	220	226	242	300	303	330	1.364	EN 10346
S250	250	257	275	330	333	363	1.320	
S280	280	288	308	360	364	396	1.286	
S320	320	329	352	390	394	429	1.219	
S350	350	360	385	420	424	462	1.200	
S390	390	401	429	460	465	506	1.179	
S420	420	432	462	480	485	528	1.143	
S450	450	463	495	510	515	561	1.133	

Table A.9 Steel properties data according to reference

Steel grades	**Yield Strength**		**Tensile Strength**	
	Mean (X characteristic value)	CoV	**Mean** (X characteristic value)	CoV
S220 to S450	1.10	0.04	1.10	0.05

A.6.4 Non-destructive hardness tests

Introduction

Every reclaimed member is to be subjected to a non-destructive hardness test in order to establish a value for the yield strength and the ultimate strength of the steel. A relationship exists between measured hardness and steel strength that is considered sufficiently accurate to define the material grade. The relationship between measured hardness and material strength depends on the type of hardness test performed.

Hardness testing should be completed on reclaimed elements, at locations of lower stress in service. For simply supported beams, locations near the end of the element are recommended. Any surface treatment must be removed from the area that is to be tested.

The material hardness result should be taken as the mean of three measurements in the same location. Steelwork coating system must be removed to allow for the measurements.

Results from each member in a group should be assessed in accordance with EN 1990 to determine the representative value for the whole group. Once the hardness value for the group has been determined, the yield strength and tensile strength should be calculated and compared with the minimum values from Table A.6 and Table A.8 to define the steel grade.

Assessment of hardness test results

The hardness of an individual member should be taken as the average of ten measurements. If this average value for an individual member differs by more than 10% from the average value for the group of members, the inconsistent member should be removed from the group.

The characteristic value of hardness H_v of the entire group should be determined using Table D.1 from EN 1990, assuming "V_x unknown" and calculated using the following expression:

$$H_{\mathrm{v}} = m - k_{\mathrm{n}} S_{\mathrm{x}} \tag{A.1}$$

where:

H_{v} is the characteristic value of hardness for the group;
m is the group mean value (mean hardness of the members within the group);
S_{x} is the standard deviation of the results;
k_{n} is taken from Table D1 of EN 1990 for "V_x unknown", presented as Table A.10.

Table A.10 Values of k_n for the 5% characteristic value (EN 1990, Table D.1)

Number of members in the group (*n*)	**1**	**2**	**3**	**4**	**5**	**6**	**8**	**10**	**20**	**30**	**∞**
V_x unknown	–	–	3.37	2.63	2.33	2.18	2.00	1.92	1.76	1.73	1.64

An ultrasonic hardness test can be used as testing method. All members are tested for Vickers hardness according to EN ISO 6507 [103].

Correlation between hardness and material strength

EN ISO 18265:2013 [120], Annex A includes conversion tables from hardness to ultimate tensile strength. The conversion involves considerable scatter, and the obtained ultimate tensile strength is informative only.

As an alternative, reference [119] presents the relationship between Vickers hardness test and strength to be used to estimate the properties of the material.

However, CEN / TS 1090-201 [2] does not recommend this approach.

A.6.5 Destructive tensile tests: non-statistical and statistical testing

Introduction

The location of the samples for destructive tests should be selected according to the recommendations of the product standard. Appendix A of EN 10025-1 provides guidance for hot rolled members and plates. Annex C of EN 10219-1 provides guidance for cold formed welded hollow sections, while Annex C of EN 10210-1 provides guidance for hot finished hollow sections.

Destructive tensile tests are used to determine the following properties of the steel:

- Yield strength;
- Tensile strength;
- Yield to ultimate ratio;

- Elongation at failure.

The tensile destructive tests shall be performed according to EN ISO 6892-1 [113]. As a reference, test sample locations may be defined according to ISO 377 [121]. Guidance from the relevant product standard may also be followed, for example, EN 10025 or EN 10219.

The declared yield strength, tensile strength, and elongation should be based on the results of the destructive tests, not on the non-destructive tests. The declared yield strength and tensile strength should be the strengths given in the appropriate product standard for the determined steel grade, which is identified using results of the destructive tests, not on the non-destructive tests.

As a remark, it should be noticed that if a reclaimed element does not comply with a certain product standard, such as EN 10025-2, the element can still be used as long as the relevant material properties are declared, as requested by EN 1090-2, clause 5.1. As an example, if the elongation at failure measured by a destructive test does not comply with the minimum values of EN 10025-2 for a specific steel grade, but if the measured elongation is such that the minimum values of EN 1993-1-1 for the elastic global analysis are fulfilled (see Table 5.1), the reclaimed steel can still be reused.

Non-statistical testing

In addition to the 100% non-destructive testing, a single destructive test (taken from any member in the group) is required to respect the minimum values from Table A.6 or Table A.8. A single test has no statistical value and is therefore described as ‘non-statistical’.

Non-statistical destructive testing (*i.e.* one single destructive test from a group) is recommended for steel to be used in Consequence class 1 or Consequence class 2 structures. The mechanical properties of type 1 structural steel may be determined by testing a single representative sample according to testing protocol B in 5.3.4.4 of CEN / TS 1090-201 [2].

Non-statistical testing procedure is not recommended for cold formed elements.

Statistical testing – assessment of tensile test results

In addition to the 100% non-destructive testing, a minimum of three destructive tests are required, taken from members within a group. Increasing the number of tests will improve the precision of the calculated values and will generally result in higher values.

The characteristic value of yield strength and ultimate strength of the entire group should be determined using Table D1 from EN 1990, assuming “V_x known” and calculated using the following expression:

$$X_{\mathrm{d}} = m - k_{\mathrm{n}} S_{\mathrm{x}} \tag{A.2}$$

where:
X_{d} – is the characteristic value of interest (yield strength, or ultimate strength);
m – is the sample mean value;
S_{x} – is the standard deviation;
k_{n} – is taken from Table D1 of EN 1990 for “V_x known”, presented as Table A.11.

Table A.11 Values of k_n for the 5% characteristic value (EN 1990 Table D.1)

Number of DT	**1**	**2**	**3**	**4**	**5**	**6**	**8**	**10**	**20**	**30**	∞
V_x known	2.31	2.01	1.89	1.83	1.80	1.77	1.74	1.72	1.68	1.67	1.64

The use of “V_x known” is justified because the coefficient of variation for both yield strength and ultimate strength is known.

If statistical testing is completed, the calculated values from the destructive tests should be used to determine the steel grade from Table A.6 or Table A.8.

A.7 Impact toughness

Unless destructive tests are conducted, it should be assumed that the steel is subgrade JR according to EN 1993-1-10. There may be economic benefits in completing destructive tests to demonstrate that reclaimed steel is of a tougher sub-grade, particularly on thicker sections.

If required, destructive tests should be used to establish the steel sub-grade of members within a group, based on the testing of one representative member. According to EN 10025-1, six samples are required for testing purposes, taken from locations identified in Annex A of EN 10025-1.

For every 20 tonnes in a batch, one set of tests (six samples) from one single member should be used to determine the Charpy value for all members in that batch. The Charpy test should be performed according to EN ISO 148-1 [115].

A.8 Chemical composition

A.8.1 Introduction

The chemical composition of reclaimed steel should be determined so that the Carbon Equivalent Value (CEV) can be calculated using the expression in Section 7.2.3 of EN 10025-1 or Section 6.6.1 of EN 10219-1.

The chemical composition should be assessed using non-destructive and destructive techniques. The CEV for the group should be taken as the maximum CEV from any test, including both the non-destructive test results and the destructive test results.

The chemical composition of each individual member should be tested and recorded. If the measured carbon or manganese content for an individual member differs by more than 10% from the average value for the group, the inconsistent member should be removed from the group.

The anticipated chemical composition of a specific steel can be found in Section 6.6.1 of the relevant part of EN 10025 and EN 10219. For cold formed products, EN 10346 may be used, where in Table 2 of the same standard the anticipated chemical composition for steels for construction is presented.

The declaration of the chemical composition of cold formed elements needs no to assessed if the steelwork is not to be welded.

A.8.2 Non-destructive tests to determine chemical composition

Optical emission spectroscopy can be used to determine the chemical composition of a steel member. Although this technique is considered to be a non-destructive test method, a small burr is left on the surface of the steel.

The chemical composition may be assessed according to ISO 19272 [107].

A.8.3 Destructive tests to determine chemical composition

The chemical composition of the steel can be determined by analysing swarf from a drilled cavity. The member should be drilled in a low stress location. The chemical composition may be assessed according to EN ISO 14284 [114].

A.9 Geometric tolerances

A.9.1 Cross-section dimensions

The cross-sectional dimensions (depth, breadth, flange thickness, web thickness, wall thickness etc.) must be measured for all members. A declaration of the measured dimensions must be provided by the stockholder.

If the section dimensions fall outside the permitted deviations according to the product standard, the measured dimensions should be used to determine the cross-sectional properties.

A.9.2 Bow imperfections (lack of straightness)

The straightness of every member, in both axes, should be measured and compared with the permitted deviations in EN 1090-2. Members falling outside the permitted deviations should be straightened as part of the fabrication process.

A.10 Further guidance for cold formed steel products

A.10.1 Metallic coating composition, designation and layer mass

The composition of the metallic coating needs to be specified according to EN 10346 (say *Z*, *ZF*, *ZA*, *ZM*, *AZ*, *AS*). Section 3 of EN 10346 specifies the key chemical components for each coating type. All members must be tested by non-destructive test procedures.

For the coating layer weight assessment, Section 7.3 of EN 10346 must be considered. The single spot minimum coating mass value may be used to assess the actual coating designation. For coating thickness assessment, recommendations from EN 10346, Section 7, shall be applied. The film thickness of coil coated metals may be assessed according to EN 13523-1 [122].

A.10.2 Bend radius to thickness ratio and adhesion of metallic coating

As the reclaimed steelwork is already bent, a visual inspection to assess possible cracks and the adhesion of metallic coating nearby the bend region shall be undertaken for each reclaimed element. There shall be no cracks at the bended areas visible by the naked eye (EN1090-4 section 6.1). The adhesion assessment has the objective of detecting any adhesion less than “perfect”. This may be prying, hammering, bending, beating, heating, sawing, grinding, pulling, scribing, chiselling, or a combination of such methods. If the coating peels, flakes, or lifts from the substrate, the adhesion is less than perfect. EN 10346 Section 7.10 specifies that adhesion of the coating shall be testing by using “an appropriate method”, referring that the selection of the method is “left to the discretion of the manufacturer”. See also references [123] and [124].

A.11 Assessment of reclaimed steel according to Protocol D

A.11.1 Hot rolled and hollow sections products

For structural steel with unknown provenance, conservative assumptions about the material properties may be used for the analysis and design. The conservative material properties provided in Table A.12 may be assumed.

Table A.12 Recommended material properties for non-tested structural steel

Material	f_y **[N/mm²]**	f_u **[N/mm²]**	G **[N/mm²]**	E **[N/mm²]**	ε_{uk} **[%]**	v	ρ **[kg/m³]**	α_T $[10^{-6}/^{\circ}{}_{C}]$
Steel – Members	235	360	81000	210000	15+	0.30	7850	10
Steel – Welds	–	360	–	–	–	–	–	–

Based on the building’s age and location, local standards may be used to establish basis for the conservative value for yield and tensile strengths.

A.11.2 Cold formed products

For reclaimed cold formed steelwork, as a wide range of steel grades are likely to be available, it is not recommended to assume a yield and tensile strengths of more than 120 MPa and 260 MPa respectively. See EN 10346, Section 7 and EN 1993-1-3, Section 3 for more detail.

A.11.3 Welded connections

If no testing is undertaken, the reuse scenario must ideally avoid welding procedures. For the cases where welding procedures are required, a value for the CEV of 0.51 may be assumed (based on BS 4360 from 1969 [57]). Minimal non-destructive tests may be used to assess the assumed value for CEV.

Appendix B
Material partial factor for member buckling to be used for reclaimed steelwork

B.1 Background for material partial factors to EN 1990

This Appendix contains an approach for derivation of a modified partial factor $\gamma_{M1,mod}$ for resistance of members to instability of the reused steel members. This is based on the principles of EN 1990, and the references below are for this code. It defines a partial factor γ_M for a material property also accounting for model uncertainties and dimensional variations. Clause 8.3.5(2) gives:

$$\gamma_M = \gamma_{Rd}\gamma_m \tag{B.1}$$

in which γ_m is a partial safety factor for the material strength;
and γ_{Rd} is a partial factor covering uncertainty in the resistance model, plus geometric deviations if these are not modelled explicitly.

EN 1090 does not specify a value for γ_{Rd} as it depends on the construction materials and behaviour of the structural member. Typically, for steel structures, it varies between 1.05 and 1.15 [125]. The partial factor γ_m is obtained from Clause 8.3.6:

$$\gamma_m = \frac{X_k}{X_d} \tag{B.2}$$

where X_k is the characteristic value of a material or product property;
X_d is the design value of a material or product property.

For the partial factor γ_{M1}, X_k is the nominal yield stress for a specific steel grade, $f_{y,nom}$, and X_d is defined in Clause C4.4.2(3), Eq. (C.14) for a normal distribution:

$$X_d = \overline{X}\left(1 - \alpha_R \beta V_X\right) \tag{B.3}$$

where $\overline{X}$ is the mean value of a material or product property;
α_R is the importance factor of the material property, reaching values between 0 (no importance) and 1 (maximum importance); Clause C4.4.2(3) suggests a value of 0.8;
β is the target reliability index;
V_X is the coefficient of variation of the material or product property.

B.2 Derivation of $\gamma_{M1,mod}$ for design using reclaimed steel

The value of the partial factor depends on the value of the target reliability index and the uncertainties of the parameters influencing the resistance of the member such as uncertainties concerning dimensions and material properties.

If the assessment protocols of CEN TS 1090-201 [2] are applied, one may assume that the uncertainty of the properties is at least similar to new profiles. Therefore, it is reasonable to consider the same value of the partial factor than used for new profiles.

In other cases, it might be necessary to proceed with a case-by-case assessment of reliability.

In the RFCS project PROGRESS [54] a simplified approach is proposed leading to a modified partial factor. On the safe side, one might assume that the partial factor for model uncertainty and geometric deviation takes the maximum value of 1.15, if the expected geometry deviations for reused steel members are a concern for stability verifications. Then the modified partial factor $\gamma_{\mathrm{M1,mod}}$ is calculated. This coefficient is defined as follows:

$$\boldsymbol{\gamma}_{\mathrm{M1,mod}} = \boldsymbol{K}_{\boldsymbol{\gamma}_{\mathrm{M1}}} \boldsymbol{\gamma}_{\mathrm{M1}} \tag{B.4}$$

where $K_{\gamma_{\mathrm{M1}}}$ is a correction factor

The correlation factor can take the safe sided value of 1.15.

$$\boldsymbol{K}_{\boldsymbol{\gamma}_{\mathrm{M1}}} = \mathbf{1.15} \tag{B.5}$$

Consequently, a modified value of the partial factor of 1.15 is obtained.

$$\boldsymbol{\gamma}_{\mathrm{M1,mod}} = \mathbf{1.15} \tag{B.6}$$

Funded by
the European Union

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.

ECCS
CECM
EKS

EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK
CONVENTION EUROPÉENNE DE LA CONSTRUCTION MÉTALLIQUE
EUROPÄISCHE KONVENTION FÜR STAHLBAU

publications@steelconstruct.com | www.steelconstruct.com

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.