EUROPEAN RECOMMENDATIONS FOR REUSE OF STEEL PRODUCTS IN SINGLE-STOREY BUILDINGS

2017-2020

PROGRESS

PROVISIONS FOR GREATER REUSE OF STEEL STRUCTURES

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SCI Steel Knowledge
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PROGRESS

Provisions for greater reuse of steel structures

European Recommendations for Reuse of Steel Products in Single-Storey Buildings

1st Edition, 2020

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European Recommendations for Reuse of Steel Products in Single-Storey Buildings

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

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ISBN: 978-92-9147-170-6

Printed in Macasi Artes Gráficas, Coimbra, Portugal

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Bus station at Schiphol North terminal, Amsterdam – originally built in 1942, the structure has been relocated and reused twice.

Legal deposit

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 structural sections. The RFCS-funded project *PROGRESS* (*PROvisions for GREater reuse of Steel Structures*) focused on single-storey buildings, and it identified various reuse scenarios, depending on the form of construction. It also showed how these structures can be designed to facilitate reuse of the structure or its primary components.

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

- Members to be reused should not be subject to localised corrosion or damage,
- All members to be reused should come from a building structure first constructed 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 members to be reused, they must be recovered in as much of their original intact length as possible, although some additional fabrication and preparation work may be required.

Functional reusability requirements are set out in this publication but the economic value and environmental benefits of reuse are not covered in detail.

This document also addresses the key aspects that designers need to take into account in order to facilitate greater reuse of steel structures and also presents some examples of structural reuse.

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TABLE OF CONTENTS

NOTATION

Lower case

a	Geometrical data (general); γ_{M2} according to the Swedish national annex
b	Frame spacing
c_{dir}	Directional factor (for wind load calculations)
c_{prob}	Probability factor
c_{r}	Roughness factor
c_{season}	Season factor (for wind load calculations)
c_0	Orography factor
$c_{\mathrm{moduleA1-3}}$	Lifecycle cost related to the production of the material and fabrication of the steelwork
c_{moduleD1}	Lifecycle cost (net revenue) related to the export of secondary materials
$e_{\mathrm{moduleA1-3}}$	Environmental loads related to the production of the material and fabrication of the steelwork
e_{moduleD1}	Environmental loads and benefits related to the export of secondary materials
f_{u}	Tensile strength
f_{y}	Yield strength
$f_{\mathrm{y}}(t)$	Yield strength based on plate thickness
h_{c}	Columns depth
h_{b}	Beam/rafter depth
k_{n}	Value taken from Table D1 of EN 1990
m	Group mean value
n	Exponent
p	Probability for an annual exceedance of the 10-minute mean wind velocity
q	Uniformly distributed load
q_{b}	Basic velocity pressure of the wind
q_{p}	Peak velocity pressure
q_{t}	Adapted characteristic value of the variable action for the design working life
q_{t0}	Characteristic value of the variable action for a design working life of 50 years

r	Distance between eaves and apex
s	Snow load on the roof
s_{k}	Characteristic value of the snow load on the ground at the relevant site
s_{n}	Ground snow load with a return period of n years
t	Target design working life
t_0	Standard design working life of 50 years
v_{b}	Basic wind velocity
$v_{\mathrm{b,0}}$	Fundamental value of the basic wind velocity
$v_{\mathrm{b,class}}$	Basic wind velocity for European class
v_{m}	Mean wind velocity
z	Height above ground

Upper case

A	Allocation factor of burdens and credits between supplier and user of recycled materials
C_{e}	Exposure coefficient (snow load calculations)
C_{esl}	Coefficient for exceptional snow loads
C_{t}	Thermal coefficient (snow load calculations)
C_{MR}	Lifecycle costs per unit of analysis arising from material recovery (recycling and reusing) processes
C_{VM}	Lifecycle costs per unit of analysis arising from the acquisition and pre-processing of virgin material from the cradle to the point of functional equivalence where it would substitute secondary material
E	Modulus of elasticity (Young's modulus); effect of an action
E_{VM}	Specific emissions and resources consumed per unit of analysis arising from the acquisition and pre-processing of virgin material from the cradle to the point of functional equivalence where it would substitute secondary material
$E_{VM,in}$ (E_V)	Specific emissions and resources consumed per unit of analysis arising from acquisition and pre-processing of primary material in the production of the product
$E_{VM,out}$ $(E_V^*,\ E_{VMSub,out})$	Specific emissions and resources consumed per unit of analysis arising from the acquisition and pre-processing of virgin material assumed to be substituted by recyclable materials

E_{MR}	Specific emissions and resources consumed per unit of analysis arising from material recovery (recycling and reusing) processes
$E_{MR,in}$ (E_{rec})	Specific emissions and resources consumed (per unit of analysis) arising from the recycling process of the recycled (reused) material, including collection, sorting and transportation process
$E_{MR,out}$ (E_{recEoL}, $E_{MRafterEoW,out}$)	Specific emissions and resources consumed per unit of analysis arising from material recovery (recycling and reusing) processes of a subsequent system after the end-of-waste state
E_{V}	Unit impact; emissions and consumed resources arising from the acquisition and pre-processing of virgin material in today's production
E_{V}^{*}	Emissions and consumed resources arising from the acquisition and pre-processing of virgin material at the product end of life
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;*
$G_{\mathrm{k,h}}$	Favourable permanent action h
H	Building/frame height
H_{v}	Vickers hardness value
I_{v}	Turbulence intensity
K	Shape parameter depending on the coefficient of variation of the extreme-value distribution
K_{FI}	Multiplication factor
$K_{\gamma_{M1}}$	Correction factor
L	Span
L_{column}	Columns length
L_{h}	Fabricated haunch segments length
L_{a}	Fabricated apex segment length
M_{in} ($M_{MR,in}$)	Amount of input material to the product system that has been recovered (recycled or reused) from a previous system (determined at the system boundary)
M_{net}	Net amount of material calculated by adding all output flows of secondary material at the end-of-life of the product and subtracting all secondary material recovered from the previous system

$M_{\mathrm{out}}(M_{MR,out})$	Amount of material exiting the system that will be recovered (recycled or reused) in a subsequent system
P_{f}	Probability of failure
P_{n}	Annual probability of exceedance
Q	Variable concentrated load, variable action
$Q_{\mathrm{k},1}$	Leading variable action
$Q_{\mathrm{k},j}$	Accompanying variable action *i*
Q_{MR} (Q_S)	Quality of the secondary material, i.e. the quality of the recyclable material at the point of functional equivalence
$Q_{MR,in}(Q_{Sin})$	Quality of the ingoing secondary material, i.e. the quality of the recycled material at the point of substitution
$Q_{MR,out}$ $(Q_{R,out}, Q_{Sout})$	Quality of the outgoing recovered material (recycled and reused), i.e. quality of the recycled material at the point of substitution
Q_{VM} (Q_P)	Quality of the primary material, i.e. quality of the virgin material
$Q_{VM,in}$	Quality of the ingoing primary material, i.e. quality of the virgin material
$Q_{VM,out}(Q_{Sub})$	Quality of the substituted material, i.e. quality of primary material or quality of the average input material if primary material is not used
R	Material or product property
R_i	Material flow
R_{eH}	Yield strength from testing or relevant product standard
R_{m}	Ultimate strength from testing or relevant product standard
RR	Fraction of steel recovered as scrap (or components) during the lifetime of a steel product including any scrap that is generated after manufacturing the steel product under analysis
RR_{A}	Fraction of steel recovered steel scrap from the demolition waste
RR_{B}	Fraction of reusable components that are not reused
S	Rafter length between eave and apex; European snow load class I; Amount of scrap used in the steelmaking process to make a specific product
S_{A}	Amount of scrap used for new steel components production
S_{B}	Amount of scrap for the new steel material needed to repair and re-manufacture reused components
S_{x}	Standard deviation

V_x	Coefficient of variation
W_i	European wind class i
X	Material or product property; total life cycle impacts beyond the system boundary
X	Lifecycle impact (burden or credit) of product recycling or reuse beyond the system boundary
$\bar{X}$	Mean value of a material or product property
X_d	Characteristic value of interest
X_{MR}	Lifecycle impact (burden or credit) arising from the material recovery
X_{VM}	Lifecycle impact (burden or credit) arising from the acquisition and pre-processing of virgin material
Y	Recovery process yield or efficiency (for instance the ratio of steel output to scrap input of the electric arc furnace)

Greek letters and symbols

α	Coefficient of linear thermal expansion
α_R	Importance factor of a material property
β	Reliability index
γ_F	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
$\gamma_{M1,mod}$	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
δ_{max}	Maximum deflection
δ_i	Relative deflection i
ε	Strain
ε_f	Elongation after fracture

ε_n Local elongation

ε_u Uniform elongation

μ_i Snow load shape coefficient

ν Poisson's ratio

ρ Air density

σ Stress in a member considering axial load and bending effects

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*

ψ_1 Combination factor for frequent value of a variable action

ψ_2 Combination factor for quasi-permanent value of a variable action

Φ Normal distribution

Φ^{-1} Inverse standardised normal distribution

Subscripts

ad Adjusted

d Design value

inf Inferior

k Characteristic value

mod Modified

nom Nominal

sup Superior

Abbreviations

BOF Blast oxygen furnace

CC Consequence Class(es)

CEN European Committee for Standardisation

CEV Carbon equivalent value

CFC Chlorofluorocarbon

CFF Circular Footprint Formula

CFF-M	Circular Footprint Formula modular version
CHS	Circular Hollow Sections
CoV	Coefficient of variation
CPR	Construction Products Regulation
$D_{0\text{-}A}$	In situ reuse scenario
$Di_{\text{-}B}$	Reuse scenario: same configuration and same site
$Di_{\text{-}C}$	Reuse scenario: different configuration and same site
$Di_{\text{-}D}$	Reuse scenario: same configuration and different site
$Di_{\text{-}E}$	Reuse scenario: different configuration and different site
DCL	Low ductility class systems for seismic design according to EN 1998-1
DfD	Design for deconstruction
DoP	Declaration 0of Performance
DT	Destructive Testing/Test
EAF	Electric arc furnace
EN	European Norm
ETA	European Technical Assessment
EU	European Union
EXC	Execution Class(es)
FEM	Finite element method
GWP	Global Warming Potential
H-CFC	Hydrochlorofluorocarbon
hEN	European Harmonised Standard
ID	Identification; Identity.
JRC	European Commission’s Joint Research Centre
LCA	Life-cycle assessment
LCC	Lifecycle cost assessment
LSD	Limit states design method
NA	National Annex
NAD	National Application Document
NDT	Non-Destructive Testing/Test
NDP	No performance determined

O&M	Building owner's manual
P-Δ	Global second order effects
P-δ	Local second order effects
PEF	Product Environmental Footprint
PEFCR	Product Environmental Footprint Category Rules
PU	Polyurethane
RC	Reliability Class(es)
RCi	Reliability Class i
RHS	Rectangular Hollow Sections
RSCi	Steel class; eligibility and compliance with tolerances in EN 1090-2
SLS	Serviceability Limit State(s)
SSAB	Nordic and US-based steel company
STR	Design values of actions for strength
ULS	Ultimate Limit State(s)
VAT	Value Added Tax
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

1.1 General

The construction industry needs to develop more sustainable construction practices that lead to a lower carbon footprint and contribute to the circular economy. The 3Rs waste management hierarchy (*Reduce-Reuse-Recycle)* 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 3Rs concept can be understood as follows:

- *Reduce* the CO_2 emissions and energy demands associated with steel production and/or recycling, to *reduce* waste, and to *reduce* material use by developing more efficient structural systems,
- *Reuse* reclaimed steel products, where possible, to substitute the use of new steel,
- *Recycle* to minimise depletion of primary resources and minimise environmental impacts.

The reduction of carbon emissions associated with production of materials and waste reduction are important drivers in construction. As part of the circularity philosophy in construction, Kibert [1] presented some basic steps needed to obtain a closed-loop material usage and recovery, 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, at the end of its life, its materials must be reusable or recyclable.

In the context of 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 need to be addressed, particularly concerning adequacy and reliability assessment of the reclaimed steel to ensure that:

- The reclaimed steel members satisfy the performance requirements for the mechanical, physical, dimensional, and other relevant properties in order to ensure their adequacy in design to EN 1993,
- The salvaged materials meet the quality requirements from nominal specifications to ensure their reliability to be used. For structural steel, the relevant standard for structural design is to the various parts of EN 1993
- Structures made from reclaimed steel must have continued integrity and long-term 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 or a sandwich panel and the whole structure or part of it.

The purpose of this publication is to provide recommendations and practical information on the fabrication and detailing of single-storey 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,
- To address the identified barriers to reuse of steelwork [2], in particular the sourcing and procurement of reused steel, the cost implications for reuse of structural steel and re-certification of the steel members for reuse.

1.2 Scope of this publication

These recommendations provide design guidance on the improvement of existing Eurocode-based procedures for designs using reclaimed steel products, and provide information on design for future adaptability, demountability and reuse. The emphasis is on single-storey industrial buildings, but the principles can be extended to other building types. The recommendations are presented as guidelines for the reuse of single storey buildings in the context of Eurocode design. For a specific location, the relevant National Annex 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 who are interested in reusing reclaimed structural steel today and designing new steel buildings that can, more easily, be deconstructed and reused in the future. Mainstream reuse of structural steel will require action by all parts of the steel construction supply chain and therefore this guidance may also be useful to entire supply chain.

The document was produced as part of the RFCS-funded project *PROGRESS* that addressed both deconstruction and reuse of existing single-storey buildings, and showed how new single-storey buildings can be designed, constructed and documented to facilitate future reuse, adaptation and extension. Single-storey 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 are 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 readily simple to document.

The schematic arrangement of a typical single-storey building using a portal frame system is shown in Fig. 1.1. There are essentially three layers to the structure:

- Primary steelwork consisting of a frame, and bracing system (roof and longitudinal),
- Secondary steelwork, consisting of side rails for the walls, and purlins for the roof, that 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.

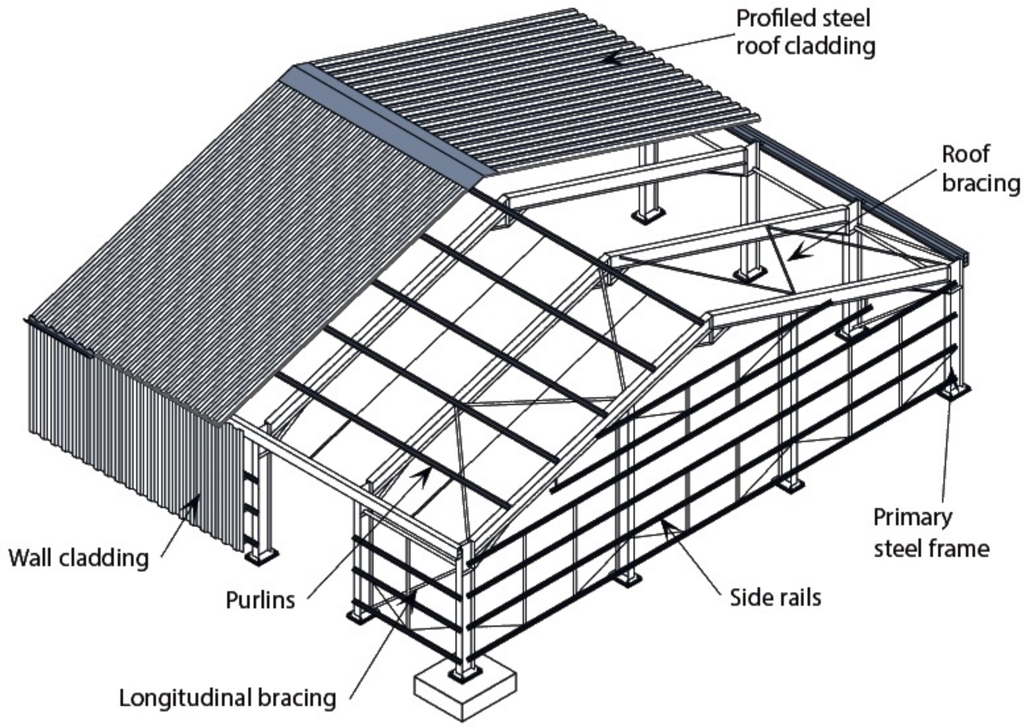


Fig. 1.1 Arrangement of a typical single-storey building, e.g. one-bay portal steel frame

Fabricated sections can be reused if calculations are made based on measured cross section dimensions. The adequacy of the measured weld sizes and fabrication procedures will need to be assessed, through existing documentation or by testing procedures.

1.3 Design and product standards

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

- EN 1090-1:2009+A1:2011 [3], which specifies requirements for conformity assessment of performance characteristics for structural steel components and kits placed on the market as construction products,
- EN 1090-2:2018 [4], which sets all the technical requirements that should be taken into account for the execution of structural steelwork,
- EN 1990:2002+A1:2005 [5], which describes the principles and requirements for safety, serviceability and durability of structures, the basis for their design and verification and gives guidelines for related aspects of structural reliability,

- EN 1991-1-1:2002 [6], which gives best-practice design guidelines and actions for the structural design of buildings and civil engineering works,
- EN 1991-1-3:2002+A1:2015 [7], which provides guidance on determining the snow load to be used for the structural design of buildings,
- EN 1991-1-4:2002+A1:2010 [8], which is the European standard for wind actions on structures,
- EN 1991-1-5:2003 [9] which is the European standard for temperature actions on structures,
- EN 1993-1-1:2005+A1:2014 [10], which gives general rules for the design of steel structures,
- EN 1993-1-3:2006 [11], which gives design requirements for cold-formed members and sheeting,
- EN 1993-1-8:2005 [12], which specialises in the design of steel joints,
- EN 1993-1-10:2005 [13] that contains design guidance for the selection of steel for fracture toughness and for through thickness properties of welded elements where there is a significant risk of lamellar tearing during fabrication.

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 [14], which sets the requirements for flat and long products of hot rolled structural steels and specifies their general delivery conditions,
- EN 10025-2 [15], which specifies the technical delivery conditions for flat and long products and semi-finished products for further processing to flat and long products of hot rolled non-alloy quality steels in grades S235, S275, S355, and S450,
- EN 10025-4 [16], which specifies requirements for flat and long products of hot rolled weldable fine grain structural steels in the thermomechanical rolled condition in grades S275, S355, S420, and S460,
- EN 10025-5 [17], which specifies requirements for flat and long products of hot rolled steels with improved atmospheric corrosion resistance in grades S235, and S355,
- EN 10029 [18], which sets out the tolerances on dimensions and shape for hot-rolled steel plates with thickness ≥ 3 mm,
- EN 10034 [19], which sets out the tolerances on dimensions and shape for structural steel I and H sections,
- EN 10051 [20], which sets out the tolerances on dimension and shape for continuously hot-rolled strip and plate/sheet cut from wide strip of non-alloy and alloy steels,
- EN 10055 [21], which gives the dimensions, and tolerances on shape and dimensions for hot rolled steel and equal flange tees with radiused root and toes,
- EN 10056-1 [22], which gives dimensions for structural steel equal and unequal le angles,
- EN 10056-2 [23], which specifies tolerances on shape and dimensions for structural steel equal and unequal le angles,

- EN 10204 [24], which is the European standard for inspection documents that authenticate materials used in metallic and non-metallic product that are a legal and regulatory requirement that must be provided to those purchasing products as proof of quality and product specification,
- EN 10210-1 [25], which specifies the technical delivery conditions for hot finished hollow sections of circular, square, rectangular or elliptical forms and applies to hollow sections formed hot, with or without subsequent heat treatment, or formed cold with subsequent heat treatment to obtain equivalent metallurgical conditions to those obtained in the hot formed product,
- EN 10210-2 [26], which specifies the tolerances, dimensions, and sectional properties of hot finished structural hollow sections of non-alloy and fine grain steels,
- EN 10219-1 [27], which specifies the technical delivery conditions for hot finished hollow sections of circular, square, rectangular or elliptical forms and applies to hollow sections formed hot, with or without subsequent heat treatment, or formed cold with subsequent heat treatment to obtain equivalent metallurgical conditions to those obtained in the hot formed product,
- EN 10219-2 [28], which gives requirements for tolerances, dimensions, and sectional properties for cold formed welded structural hollow sections of non-alloy and fine grain steels,
- EN 10279 [29], which specifies tolerances on shape, dimension and mass of hot rolled steel channels,
- EN 10346 [30], which gives guidelines and recommendations related to all continuously hot-dip coated products, including cold forming steels,
- EN 10365 [31], which specifies the nominal dimensions and masses of hot rolled channels, I and H sections,
- EN 14509 [32], which gives requirements for self-supporting double skin metal faced insulating panels.

1.4 General notes on the document

The terms and definitions used in this design guide are presented below. Section 2 presents a brief description of the anatomy of single-storey buildings. Section 3 considers the economic and environmental benefits of reusing reclaimed steel members, and defines different potential reuse scenarios. This manual is divided into three parts:

Part 1: Recommendations for reusing existing single-storey buildings,
Part 2: Recommendations for the design of single-storey buildings to facilitate future deconstruction and reuse,
Part 3: Case studies.

Part 1 discusses general technical issues related to the structural use of reclaimed steel from existing single-storey industrial buildings. It starts with an historical review of European codes of practice and product standards. Section 6 covers the selection and acceptance of materials, and their classification for “new” designs. As noted, the design procedures are in accordance with the Eurocodes. Section 7 gives information on practical aspects of fabricating and erecting structures from reclaimed steel. Section 8 discusses structural design aspects in terms of Limit States principles.

Part 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 design for disassembly and reuse of steelwork are presented in Section 9.

Section 10 defines the loads and combination of actions to be used in design calculations. Section 11 gives general improvements in construction details that facilitate future reuse.

Part 3 presents some case studies that illustrate the use of the reclaimed steel structures in various EU countries and some of the technical issues that were overcome.

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 reused steel members is presented in Appendix B.

1.5 Terms and definitions

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

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

Demolition	Process whereby a building is taken apart with little or no attempt to recover any of its constituent parts for reuse; products of demolition may, however, be recycled
Design for Deconstruction (DfD)	Designing for deconstruction is a decision-making process at the design stage as to how a building can be taken apart and potentially reused
Design working life	This is the assumed period for which the component is to be used for its intended purpose with anticipated maintenance but without major repair being necessary
Distributor	Any natural or legal person in the supply chain, other than the manufacturer or the importer, who makes a construction product available on the market
Envelope	The enclosure around the building that separates the enclosed space from the external environment and provides a range of structural and building physics functions.
Execution class	Classified set of requirements specified for the execution of the works as a whole, of an individual component or of a detail of a component
Hazardous waste	Waste that poses substantial or potential threats to public health or the environment
Importer	Any natural or legal person established within the EU and who places a construction product form a third country in the EU market
In-situ reuse	The component or structure is reused, without displacement, on the same site. For example, a building structure can be retained and reused during building renovations.
Manufacturer	Any natural or legal person who manufacturers a construction product or who has such a product designed or manufactured and markets that product under his name or trademark
Notified body	A Notified Body is an independent (non-governmental) third-party body, recognised by the EU/EEA and is authorised to carry out conformity assessments for products that meet the requirements of a harmonised standard (hEN) or European Technical Assessment (ETA)
Pre-demolition audit	Qualitative and quantitative assessment of construction and demolition waste streams prior to deconstruction, demolition, or renovation of buildings and infrastructures
Primary structure	The primary steel frame, comprising all main load bearing elements, e.g. columns, beams and bracing

Purchaser	The company purchasing the steel products; generally, this is a steelwork contractor that executes the structural steelwork
Reconditioning	Process of returning a product to good working condition by replacing major components that are faulty, and making changes to update the appearance of a product, such as by cleaning, painting or refinishing
Recycling	Process of converting waste materials into new materials and products; recycling steel involves re-melting of scrap to form new semi-finished products
Refurbishing	This means the process of renovation of an existing building to suit a new use and can involve a range of processes from replacing fitments and fittings to major structural alterations
Relocated reuse	Requires transport of the structure or component in order to reuse it on another site.
Remanufacturing	Returning a product or component to the performance specification of the original equipment manufacturer
Repairing	Fixing a fault but with no guarantee of the product as a whole. In the context of steel structures, this can mean strengthening a component, often by welding plates
Repurposing	Any operation that changes the function or purpose of a component
Reuse	Use of old components with little or no reprocessing, largely in their original form; they may be reused for the original function (a conventional reuse scenario), or repurposed
Secondary structure	The secondary steelwork, consisting of side rails and purlins for the walls and the roof, respectively, used to support the cladding/envelope and to provide restraint to the primary structure
Structural component	Component used as a load-bearing part of the structure that is designed to provide mechanical resistance and stability and/or fire resistance, including aspects of durability and serviceability. The component is often part of a load-bearing steel structure, which may itself be an assembly of several smaller components
Structural kit	Set of standardised structural components that are assembled and installed on site
Supplier	The company stocking and supplying the steel products to the market
Waste	Unwanted or undesired material to be discarded

2 COMPONENTS OF SINGLE-STOREY BUILDINGS

2.1 General

Typical single-storey steel buildings consist of a primary steelwork, i.e. frame and bracing system, secondary steelwork and cladding that have the following characteristics:

- Function: single storey buildings are often designed as large enclosures that are used for manufacturing and industrial functions and/or storage, and can cover a range of uses, including warehouses, retail outlets, science parks, and distribution centres. These buildings generally have long spans in single or multi-bay form and often office space is provided in a connected structure or on a mezzanine level.

Loading conditions:

- 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 - see the comparisons in Table 2.1:

- 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.

Proportions: in terms of the building span and height. The optimum span of a portal frame is 4 to 6 times its height to eaves level, for example.

Table 2.1 **Comparison of basic building forms for single storey buildings [33]**

Simple construction	Portal frames	Lattice structures
Advantages		
Simple design	Medium to long span range of 30 to 50 m with a range of member sizes.	Longer spans than portal frames (up to 100 m).
Simple connections	Stable in-plane due to the rigidity of the frame.	Heavy loads may be supported by trusses.
Rapid erection but usually of more members	Member sizes and haunches may be optimised by plastic design.	Modest deflection because of the high truss stiffness.
Disadvantages		
Relatively short spans (say up to 20m)	Limited to relatively light vertical loading.	More significant fabrication costs.
Bracing may be needed for in-plane stability	Heavy overhead cranes may require strengthening of the structure	Longitudinal bracing system often required
Economy due to a continuous system not achieved	Member stability requires connection to secondary steelwork.	May required complex connections

2.2 Primary steelwork

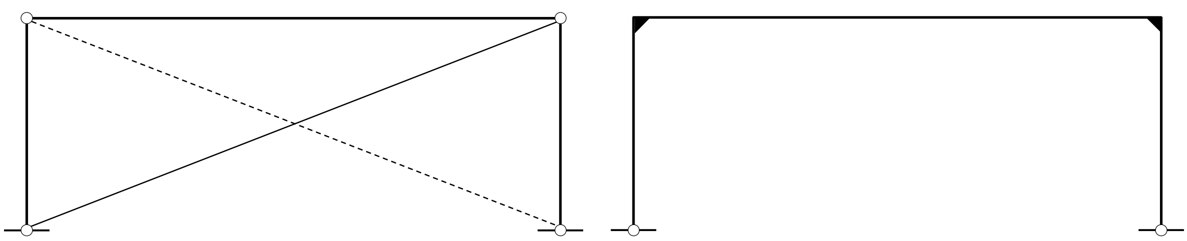
2.2.1 Structural frames

Structural frames can be designed according to the principles of simple construction, continuous construction, or semi-continuous construction. In simple framing, the joints between the members may be assumed to be nominally pinned, i.e. they have low rotational stiffness and do not transmit significant moments. Horizontal loading is resisted by a bracing system (typical case of a braced box).

In continuous construction, the joints are designed to provide sufficient stiffness so that the consideration of full rigidity between the frame members is valid, and therefore the transmission of moments as a continuous system is also valid. The frame itself has to resist all the vertical and horizontal actions and to provide lateral stiffness in order to limit lateral displacement (drift). In semi-continuous frames, the joints have a finite rotational stiffness and resistance, and will therefore transmit some moments from the beams to the columns.

A frame can be classified as non-sway or sway. Non-sway frames are relatively insensitive to second order effects, so that the internal forces in the structure can be determined on the basis of its un-deformed shape. It is generally accepted that a frame can be classified as *braced* if the bracing system reduces its horizontal displacement by at least 80%. Unbraced frames, if designed economically, are typically classified as sway frames (sensitive to second order effects) in which in-plane stability is provided by continuity of the framed structure.

Fig. 2.1 shows two simplified framing schemes consisting of either in-plane bracing or a portal frame action. The frame is the primary load bearing system that consists of beams (or girders), connected to columns, in which the column bases are supported by the foundations. The base connections can be classified as nominally pinned, semi-rigid of fixed. The beams and columns can be hot-rolled profiles, built-up/fabricated sections, hollow sections or latticed members. The primary structure is composed of the frame members and the bracing system.



a) Braced frame b) Portal frame

Fig. 2.1 Framing schemes, e.g. single bay flat roof frame

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

2.2.2 Roof structures

Roofs can be flat or pitched forms. Roof systems are designed to transmit the loads and to form the enclosure to the building in order to maintain its required internal environment and function. From a structural point of view, roof systems are designed to support the 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.

Flat roof systems use rolled I/H section beams for spans up to 15 m, cellular beams (with multiple web openings) for spans up to 20 m, and lattice trusses with parallel chords for long spans, typically longer than 20 m. In flat roofs, ponding of water can be avoided by means of an efficient drainage system.

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

- Pitched-roof portal frames (using inclined beams (rafters) connected to columns with rigid joints) for spans up to 50 m. They may be in single or multi-bay configurations and the pitch or slope is generally about 6° to the horizontal;

- Trusses with a sloping top chord (typically fabricated with hollow sections with welded joints) for spans up to 100 m or for heavy loads acting on the roof or suspended below the roof.

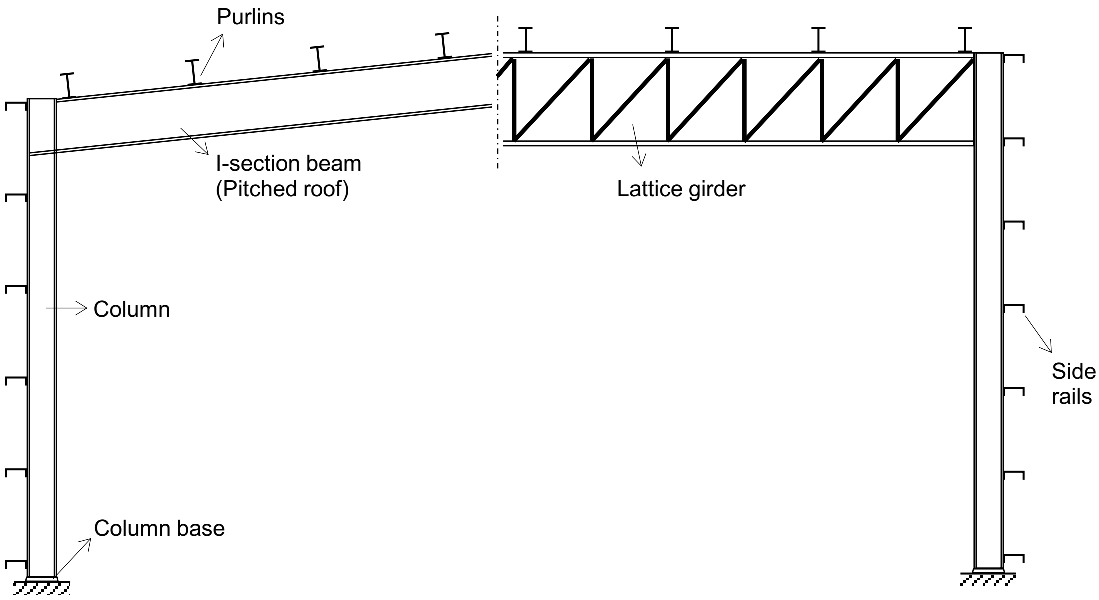


Fig. 2.2 Structural elements of typical single-storey steel building

2.3 Bracing systems

Bracing systems used in single-storey buildings may be divided into 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, and often includes trusses (triangulated system of straight interconnected structural elements), or diaphragms by taking account of the stressed skin action of the roof cladding. Fig. 2.3 shows vertical and roof cross-bracing systems and the load path for the design of these elements. When cross bracings are specified, angles or flats can be used as tension-only members. Profiled roof sheeting may be designed to act as a diaphragm and also stiffens the building considerably.

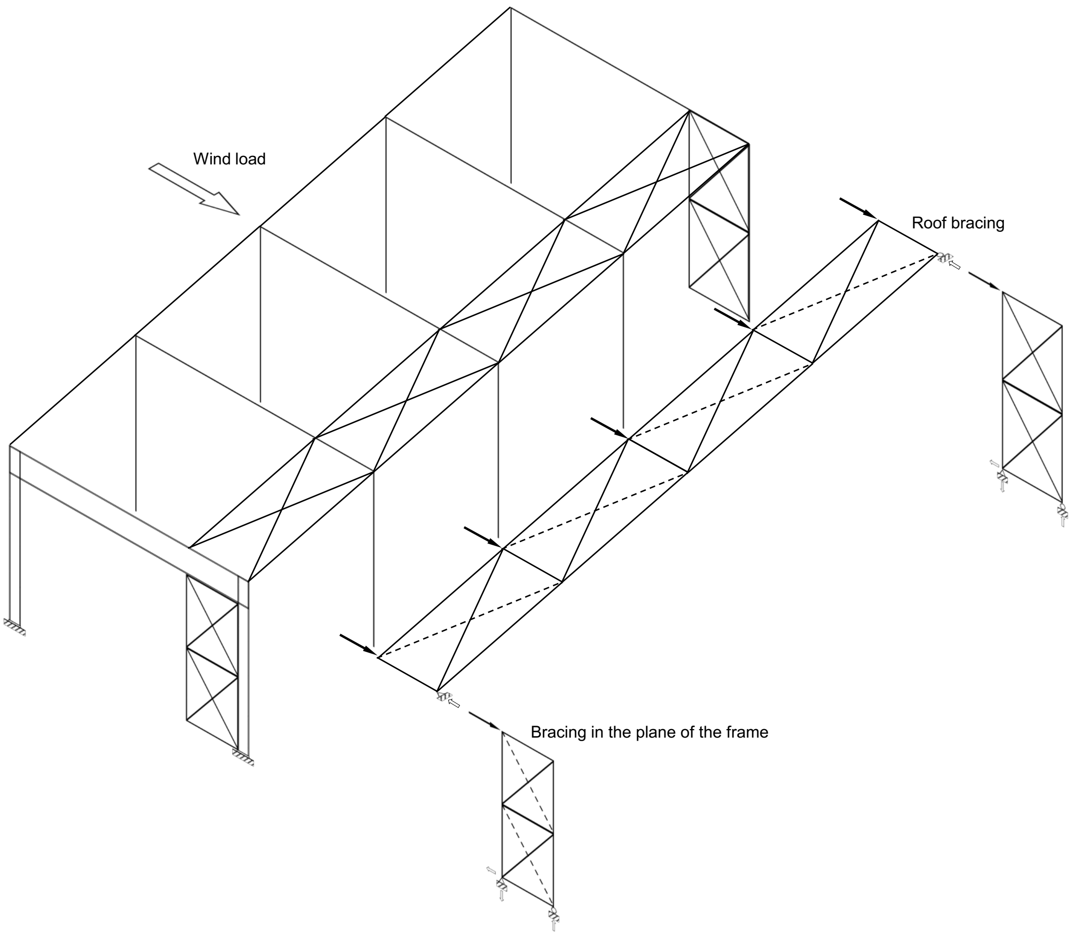


Fig. 2.3 In-plane bracing schemes in the walls and roof

Temporary bracing may be required depending on the construction sequence. Construction may start by a pair of braced central frames of the building in order to reduce cumulative tolerance errors and to resist any destabilising effects during erection of the steel. If a central vertical bracing does not exist, the common procedure is to first erect one of the two extreme pairs of braced building frames.

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

2.4 Secondary steelwork

Secondary steelwork in the roof typically comprises a system of light gauge steel purlins spanning between the beams. Depending on type of cladding elements, 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 structural frames. For typical portal frames applications, a continuous system with overlapped or sleeved solutions is often used.

Roof purlins are cold-formed members, usually of C- or Z-shape (sigma or omega sections are alternative options), that are designed to:

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

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

Wall cladding is often supported by horizontal side rails that span between the columns of the primary frame. Side rails are generally cold-formed C section members. Cold-formed Z-shape members may be used if the side rails system is designed as continuous over the primary structure columns. Side rails are designed to:

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

Vertical restraints are connected to the side rails at discrete locations to prevent lateral torsional buckling (due to bending of the side rails under wind negative pressure) and also prevent the side rails from deflecting excessively under their minor axis prior to the cladding installation. These vertical restraints are typically light steel sections (tubes, angles or channels) for strut elements and steel bars/rods for tie elements.

For column spacing up to 6 m with a typical side rail spacing of between 1.2 and 1.8 m, a single central vertical restraint will normally be sufficient. However, for wider column spacing, two or even three vertical restraints may be required. The cladding stiffens the wall substructure and transfers a significant proportion of the vertical load to the columns by diaphragm action. The cladding also restrains the side rails against lateral torsional buckling in the sagging case and provides partial restraint in the hogging case.

As a guide to steelwork sizes, for a typical 35 m multi-span portal frame, with a height to underside of haunch of 3.5 m and roof slope of 6°, the rafters are typically 450 mm or 500 mm deep and the columns typically 600 mm deep. The total primary steelwork weight is about 30-35 kg/m^2 floor area in this medium span structure. For regions where the snow load is not critical for the design, the total primary steelwork weight can be as low as 25-30 kg/m^2. Mezzanines, including the floor structure and columns, for a typical span up to 6 - 8m, usually require 35-40 kg/m^2.

2.5 Cladding systems

The main function of a cladding system is to provide a controlled internal environment depending on the intended use of the building and this will determine the performance requirements for the building envelope. Some general requirements of the cladding system are:

- Provide the required level of thermal insulation, taking account also of roof lights and junctions,
- Resist wind loading and wind uplift through the fixings to the secondary elements.
- Prevent fire spread,
- Provide an airtight building envelope,
- Include measures for suitable ventilation of a building by mechanical equipment.
- Provide acoustic insulation depending on the building function and nearby roads.
- Stabilise the secondary steel members, and sometimes the primary steelwork, by suitable restraints.

In single-storey buildings, short span cladding (spans up to 2 – 3 m) is usually fixed to the secondary steelwork. As an alternative, long span cladding claddings with spans up to 10 m can be used, which is a common practice in Nordic countries. The thermal insulation requirements in Nordic countries often require the use of thick panels that are structurally strong, so that they can span between the primary structural members without the need of purlins. Long span cladding can be in the form of deep trapezoidal sheeting or panels in roofs and horizontally installed sandwich panels on walls spanning 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.

3 BENEFITS OF REUSING OF CONSTRUCTIONAL STEEL

The construction sector is one of the largest consumers of raw materials and generates large amounts of construction and demolition waste. The goal of today's environmental policies is therefore to reduce those waste streams by recycling and reuse of materials and extending the life of components and structures. The built environment can therefore serve as a material bank with associated embodied energy and carbon in the building materials, fabricated components and structures. Their targeted separation and recovery during demolition can divert over 70% of materials from landfill [34], and it can also contribute to the circular economy goals if the materials, components and structures are used again in new construction.

Reuse of components or entire structures is the most efficient form of material recovery and can be seen as waste prevention or high-level recycling. It offers additional environmental and economic advantages, when compared to other recycling options, but often requires higher initial investment costs. This Section provides guidance on how to estimate and declare various benefits of steel reuse including environmental impacts (such as carbon footprint or waste reduction) and economic impacts (such as lifecycle cost or residual value).

3.1 Impacts of steel reuse

In the case of constructional steelwork and steel-based building components, reuse avoids negative impacts associated with scrap recycling in steelmaking. The avoided scrap can be from individual fabricated components or the whole steel assemblies or steel parts separated from composite elements (e.g. in sandwich panels). The market for reclaimed steel construction products is still small, because the effort associated with their reconditioning and CE-marking often makes the process more expensive than material recycling. Also reusing individual structural steel components is more difficult, because they are typically optimized and fabricated for a specific building design.

Therefore, most of the successful reuse projects are relocations of the whole buildings, repairs and refurbishment or extensions of steel structures, or in-situ reuse. From this point of view, a certain degree of standardisation of section shapes and sizes can be beneficial as well as building material passports and digital twins of the components to avoid unnecessary measurements and testing.

The overall benefit of reuse of constructional steel depends on widespread adoption of this approach in design and construction practices. Designers should understand how to incorporate reclaimed materials in new design applications and how to optimize their designs for deconstruction and reuse. Fabricators should consider salvaged steelwork as one of the material sources. Property owners and developers should be able to consider and communicate the possibilities of relocation of existing steelwork.

The building's or product's lifecycle can be divided into several stages (called Modules) according to CEN TC/350 standards for the LCA (Life-cycle assessment) and LCC (Lifecycle cost) assessment [35]-[37] (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

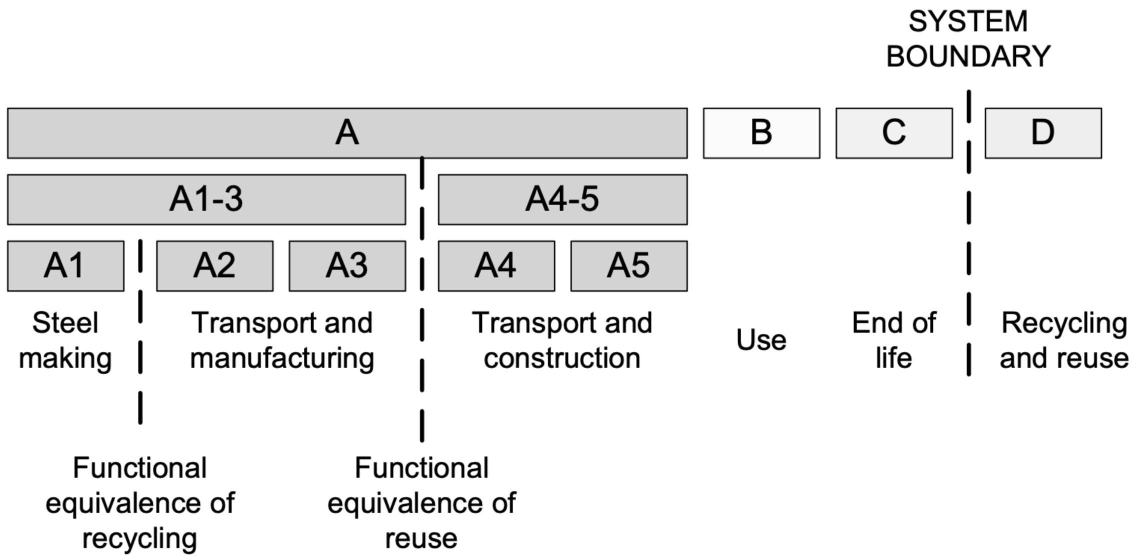


Fig. 3.1 Lifecycle stages of steel and steel-based components

The benefits of reuse can be therefore important in two lifecycle stages:

- **Building with reused products (reuse today)** may decrease the cost of the assembled steelwork and its environmental impacts in the Product and construction stage A. If the products cannot be reused again, Module D will show a burden because of the higher impacts in the next life cycle.

Design for deconstruction and reuse (future reuse) increases the residual value of the building and benefits beyond the current building's lifecycle that are expressed in the Module D. It should be noted that deconstruction and reconditioning of the recovered steelwork may increase the impacts in the End-of-life stage C.

3.2 Reuse scenarios

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

D_0: Reuse of the entire steelwork or its part (e.g. several bays) insitu without disassembly
D_1: Reuse of the disassembled steelwork (may include the envelope)
D_2: Reuse of the fabricated components (e.g. sandwich panels, columns)
D_3: Reuse of the constituent products (e.g. sections, plates)

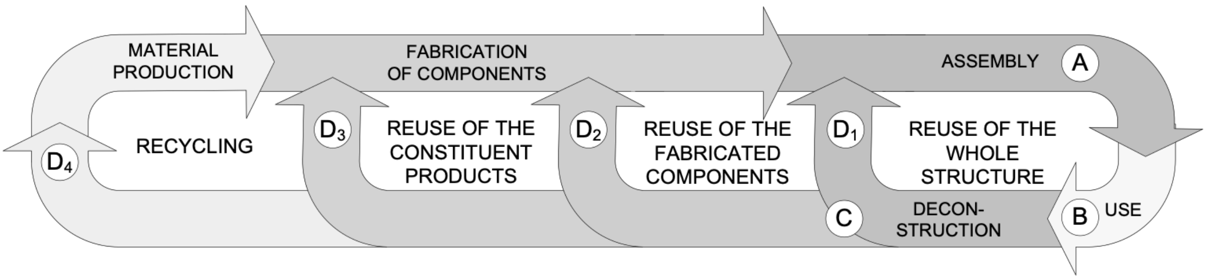


Fig. 3.2 Reuse scenarios in the value chain

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

D_A: In-situ reuse without disassembly
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 different site in the same configuration
D_E: Reuse on different site in different configuration

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

In the case of *in-situ reuse* (D_A) the components are not disassembled and remain connected to the steelwork. They can be repaired, reinforced or coated in order to prevent their disassembly and replacement, and therefore we call this process reuse. Their resistance and serviceability need to be verified according to the current codes. A typical example is the in-situ reuse of the entire primary structure.

The *relocated reuse* (D_B to D_E) means that the components are disconnected and reconditioned either on the building site or in the workshop. In many cases, the building site is used re-developed and so it may be beneficial to consider integrating steel components of the previous building in the new project (D_B and D_C).

Table 3.1 Classification of reuse cases

Case	In situ reuse	Relocated reuse			
		Same site		Different site	
		Same configuration [a]	Different configuration	Same configuration	Different configuration
Entire steelwork	$D_{0\text{-}A}$	-	-	$D_{0\text{-}D}$	-
Disassembled steelwork	-	$D_{1\text{-}B}$	$D_{1\text{-}C}$	$D_{1\text{-}D}$	$D_{1\text{-}E}$
Fabricated components	-	$D_{2\text{-}B}$	$D_{2\text{-}C}$	$D_{2\text{-}D}$	$D_{2\text{-}E}$
Constituent products	-	$D_{3\text{-}B}$	$D_{3\text{-}C}$	$D_{3\text{-}D}$	$D_{3\text{-}E}$

[a] This scenario is unlikely since if the structure was deconstructed, it is unlikely that it would be re-erected in exactly the same configuration on the same site.

Relocated reuse to a different site (D_D and D_E) can be organized using the material dealer (in the case of larger quantities of small components such as structural sections) or can be negotiated directly between the 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 with regards to the transport need are shown in Table 3.1, where index "A" means that the reuse takes place on the same building site and "B" means that the components require transport (e.g. between sites, to the dealer, storage or workshop).

The cladding is a more complex component to be reused. If it is a double skin trapezoidal system, attention should be paid for all the layers. Sandwich panels can be reused if the screw holes are hidden or are reused in the second use. To preserve the protection by the coating 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.

The following case examples show different reuse scenarios and some of the issues that were addressed in terms of reuse:

3.2.1 Relocations

SEGRO warehouse, Slough, UK ($D_{1\text{-}E}$)

The warehouse building built in 2000 was relocated in a different layout in 2015 to enable the construction of a new road bridge. The original brick cladding was replaced by a new composite panel wall system [38].

Agrocolumna warehouse, Copăceni, Romania ($D_{1\text{-}D}$)

The building was constructed in 2004 in Craiova and it consisted of a two-storey office area and a warehouse. In 2012, it was moved to Copăceni (227 km east of Craiova) and one more bay was added to the warehouse [38].

Metis canopy, Otočcu, Croatia ($D_{1\text{-}D}$)

The original structure was erected in Pula and was relocated for reuse in 2011 in Otočcu, 266 km away [38].

Steel industrial kit hall for multiple locations ($D_{1\text{-}E}$)

An existing standard kit structure was used to construct buildings in different locations in Romania between 2008 and 2010. Recently, in 2020, a new complex of buildings reused the elements of one of the existing standard kits [38].

West harbour, Helsinki, Finland ($D_{0\text{-}D}$)

The old warehouse from the West harbour in Helsinki was relocated without disassembly using cranes because of the short distance. The building was then reused as the terminal building [39].

3.2.2 Relocation and conversion of old enclosures

Bus station Schiphol – Nord, Amsterdam, Netherlands ($D_{1\text{-}D}$)

The original building was constructed in 1958 and was used as a hangar by the Rotterdam Airport until the late 1990s. In 2003, the structure was reused as a hangar for 7 years by the Rotterdam Detention Center. In 2015, it was reused again as a bus station in Schiphol [38].

BRE test facility, Cardington, UK ($D_{1\text{-}D}$)

Two sheds in Cardington in the midlands of England were converted from hangars for building zeppelins in wartime. No. 2 shed owned by BRE was initially erected in Pulham, but was moved to Cardington in 1928. The entire 3270 tonne steel structure was dismantled and reassembled [40].

3.2.3 Reuse of other buildings and structures

S-Market, Urjala, Finland ($D_{1\text{-}D}$)

The owner of a retail store chain in Finland decided to replace an existing building in Tampere with a new larger one. At the same time, the need for a new grocery store emerged only 60 km away and created a perfect opportunity for a relocated reuse [38].

Sydney Olympics aquatic centre, Sydney, Australia ($D_{1\text{-}C}$)

Temporary seating for the aquatic’s stadium was deconstructed and re-erected as a permanent grandstand at a football stadium [41].

3.2.4 Partial reuse of the primary structure

NTS building, Thirsk, UK ($D_{1\text{-}D}$)

The original building order was cancelled in 2008 after which, the fabricated steelwork and the elements of the building were stored. Later the steelwork was divided in four parts and sold in auction. The new building was erected in 2017 by reusing one of the lots from the original building [38].

Kingsize Academy, Bradford, UK ($D_{1\text{-}D}$)

Two long span portal frames from existing industrial buildings were used as part of this new secondary school [41].

3.2.5 Reuse of cladding and secondary structure

Mac-Fab Systems department store, Monaghan, Ireland ($D_{2\text{-}D}$)

Steel cladding was reused on a large department store in Monaghan, Ireland [42]

740 Rue Bel-Air, Montreal, Canada ($D_{2\text{-}C}$)

Roof joists from existing industrial buildings were reused in the construction of a new government building on the same site [43].

3.2.6 Reuse of constituent products

The London Olympic Stadium, London, UK ($D_{3\text{-}E}$)

2500 tonnes of surplus oil and gas pipeline tube material was used to build major stadia projects including Old Trafford North & South Stands in Manchester, Emirates Stadium, London and the Reebok Stadium in Bolton [44].

Bedzed, London, UK ($D_{3\text{-}D}$)

Temporary steelwork used at Brighton railway station were later used in the permanent structure of this Award winning environmentally designed mixed-use development in 2002 [43].

3.2.7 Conversion of primary structure into a multi-storey building

HIDROTIM offices, Timisoara, Romania ($D_{0\text{-}A}$)

The building was constructed in the 1960s as a single storey industrial hall with a crane and was converted into a five-storey office building in 2004 [38].

RWTH seminar building, Aachen, Germany ($D_{0\text{-}A}$)

Following the closure of the RWTH heat and power plant in the 1990s, the decision was made to transform it into a seminar building by adapting the structure to meet the new functional requirements [38].

In-situ rehabilitation of a Water Treatment Plant in Brasov, Romania ($D_{0\text{-}A}$)

The building was used as water treatment plant for a local brewery factory, erected in 2003. In 2015 the owner decided to rehabilitate the building, due to the bad thermal insulation and corrosion of some steel components, keeping the function of the building and not interrupting the activity [38].

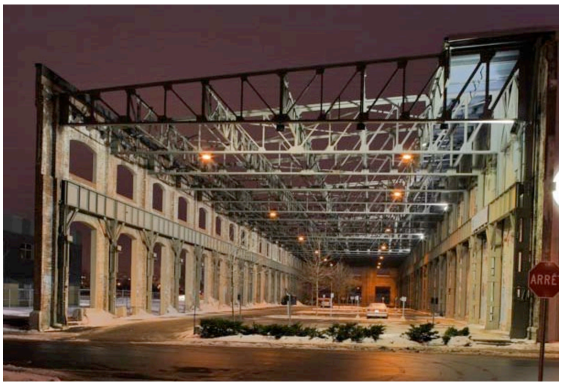
Structural strengthening of a steel structure to enable the removal of 2 columns ($D_{0\text{-}A}$)

The two-storey structure (reinforced concrete columns and steel trusses), built in 2008, located in Târgu Jiu, Romania, and is used as restaurant. In order to reconfigure the upper floor and increase the clear space, two central concrete columns were removed, and consequently the strengthening the steel trusses [38].

3.2.8 Changing the general purpose of the building

Musée d'Orsay, Paris, France ($D_{0\text{-}A}$)

The Orsay railway station in Paris was built for the Universal Exhibition of 1900 and converted into well-known museum in 1978 [45].



Angus Technopôle Building, Montréal, Québec ($D_{0\text{-}A}$)

A former locomotive assembly plant used by Canadian Pacific Railway was converted into light industrial workshops and office spaces for community-focused businesses in 1999 [43].

3.2.9 Major refurbishments

PUT laboratory, Timisoara, Romania (D_{0-A})

The structure was constructed in 1959 and consisted of truss elements. Part of the structure was damaged in 2017 by a storm and it was renovated and reused. [38].

Blue Steel Building, Leeds, UK (D_{0-A})

The existing warehouse structure was refurbished. Its portal frame was raised by 3 m, and the existing purlins, bracing and rafters were reused and a new office block was added with composite decking [41].

Winterton House, East London, UK (D_{0-A})

Originally built in 1968, the residential building was stripped back to its original structure and reclad in 1999. The heavy weight walls were replaced by lighter internal walls to reduce the load on the structure. The new brickwork façade was designed as load bearing and was partially supported by a cantilever steel structure at roof level [41].

3.2.10 Single storey buildings designed for reuse

MEXX DAY hall, Timisoara, Romania ($D_{1\text{-}D}$)

The structure was designed in 2008 as a standard kit to be adapted for different locations and applications. It was constructed in 2009 and relocated for reuse in 2017 [38].

YIT warehouse, UK ($D_{1\text{-}D}$)

The building has been built and re-built first in Finland and then twice in the UK. As YIT's construction sites have changed, Best-Hall has relocated the building, used as a warehouse, from one country to another. The building envelope has been replaced once but the frame structure has remained the same throughout [39].

3.3 Declaring impacts of future recycling and reuse

Reuse can be seen as an alternative production process for the steel components, otherwise produced from virgin materials and recycled scrap in the blast oxygen furnace (BOF) or electric arc furnace (EAF). Therefore, the impacts of reused steel should be allocated in the Product stage $A_{1\text{-}3}$ of the lifecycle assessment. However, this approach does not reward the components prepared for reuse in the future, and may discourage designers and facility owners to invest in demountable and reusable buildings and components.

For this reason, it is also important to declare the environmental impacts of reusable components beyond the system boundary in the Module D. Impacts beyond the product's system boundary are net impacts, and therefore can be both positive (burden) or negative (credits), depending on the balance between the production route of the component and its future recovery scenario. They are calculated as secondary material net flow (e.g. tonnes of steel scrap) multiplied by net impacts (e.g. specific emissions).

Net flows of the material M_{net} are calculated by subtracting from the output material flows at the end-of-life of the product (i.e. recovered steel) all content of secondary material of the product at fabrication, as in Equation (3.1), where the amount of recovered steel used in the manufacturing of the product is the input flow M_{in}, while the amount of steel to be recovered at the end-of-life of the product is the output flow to the calculated system M_{out}.

$$M_{\text{net}} = M_{\text{out}} - M_{\text{in}} \tag{3.1}$$

Fig. 3.3 illustrates material flows relevant to recycling and reuse of constructional steel. The input of steel scrap $M_{\text{in},1}$ includes scrap used for new steel components production S_{A} and scrap for the new steel material needed to repair and re-manufacture reused components, S_{B}. Similarly, the scrap output $M_{\text{out},1}$ is composed of recovered steel scrap from the demolition waste RR_{A} and reusable components that are not reused RR_{B}.

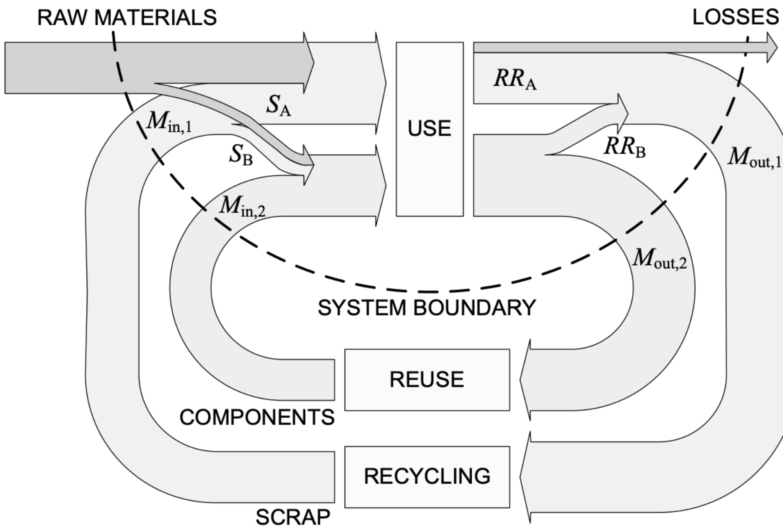


Fig. 3.3 Material flows concept in steel reuse and recycling

Net impacts (e.g. costs or specific emissions and resources consumed per unit of analysis) associated with the material flows are calculated by adding impacts arising from the material recovery X_{MR} and subtracting the impacts arising from the acquisition and pre-processing of virgin material X_{VM}. Virgin material impact (X_{VM}) is calculated from the cradle to the point of functional equivalence, where virgin material could be substituted by secondary material from recovery (i.e. X_{MR}). Functional equivalency can be established for a constituent product, component or assembly. Then the total lifecycle impacts beyond the system boundary X can be expressed as in Equation (3.2):

$$X = M_{\text{out}}\left(X_{\text{MR,out}} - X_{\text{VM,out}}\right) - M_{\text{in}}\left(X_{\text{MR,in}} - X_{\text{VM,in}}\right) \tag{3.2}$$

The net amount of substituted primary material (virgin steel) can be different from the amount of recovered secondary material (steel scrap), and therefore the net flow may be reduced by the yield factor Y representing the efficiency of the recovery process. Moreover, if the product is down-cycled or has a limited number of reuse cycles, the impact of substituted primary production may be reduced by quality factors of the secondary and virgin material Q_{MR} and Q_{VM} respectively. This is not the case in steel recycling because steel produced from recycled scrap has equivalent qualities to steel produced from virgin materials, but it might affect the declaration of impacts of steel reuse. The extended Equation (3.2) is presented as Equation (3.3).

$$X = M_{\text{out}} \cdot Y\left(X_{\text{MR,out}} - X_{\text{VM,out}}\frac{Q_{\text{MR,out}}}{Q_{\text{VM,out}}}\right) - M_{\text{in}} \cdot Y\left(X_{\text{MR,in}} - X_{\text{VM,in}}\frac{Q_{\text{MR,in}}}{Q_{\text{VM,in}}}\right) \tag{3.3}$$

If the unit impacts of the primary production and recovery process are the same at the beginning and end of product's life ($X_{\mathrm{MR}} = X_{\mathrm{MR,in}} = X_{\mathrm{MR,out}}$ and $X_{\mathrm{VM}} = X_{\mathrm{VM,in}} = X_{\mathrm{VM,out}}$), Equation (3.3) can be simplified to Equation (3.4):

$$X = (M_{\mathrm{out}} - M_{\mathrm{in}}) \cdot \left(X_{\mathrm{MR}} - X_{\mathrm{VM}} \frac{Q_{\mathrm{MR}}}{Q_{\mathrm{VM}}}\right) \cdot Y \tag{3.4}$$

When the input flow of the existing product is more efficient than the recovery at the end-of-life stage, Equations (3.3) and (3.4) produce a positive number. This means that the impact X is an overall burden. If the existing product has low recovered material content and it is recovered efficiently at the end-of-life, the impact X is a benefit. If two recovery processes are assessed at the same time, it is recommended to use the extended Equation (3.4) according to [46] written as Equation (3.5), where the flows of each secondary material are treated separately.

$$X = \sum \left(M_{\mathrm{out,i}} - M_{\mathrm{in,i}}\right) \cdot \left(X_{\mathrm{MR,i}} - X_{\mathrm{VM,i}} \frac{Q_{\mathrm{MR,i}}}{Q_{\mathrm{VM,i}}}\right) \cdot Y_{\mathrm{i}} \tag{3.5}$$

This is typical case of steel reuse when the entire steelwork cannot be reused, but the remaining material can be recycled as scrap. Equation (3.5) can be calculated for recycling and reuse streams separately as in Equation (3.6) with the indexes 1 and 2 for recycling and reuse respectively. In the case of steel recovery, quality factors can be neglected and the yield factor is relevant only for the scrap recycling. It is recommended to use $Y = 0.916$ according to the World Steel Association [47].

$$X = \left(M_{\mathrm{out,1}} - M_{\mathrm{in,1}}\right) \cdot \left(X_{\mathrm{MR,1}} - X_{\mathrm{VM}}\right) \cdot Y + \left(M_{\mathrm{out,2}} - M_{\mathrm{in,2}}\right) \cdot \left(X_{\mathrm{MR,2}} - X_{\mathrm{VM}}\right) \tag{3.6}$$

Fig. 3.4 shows graphically the allocation of flows and impacts according to Equation (3.6).

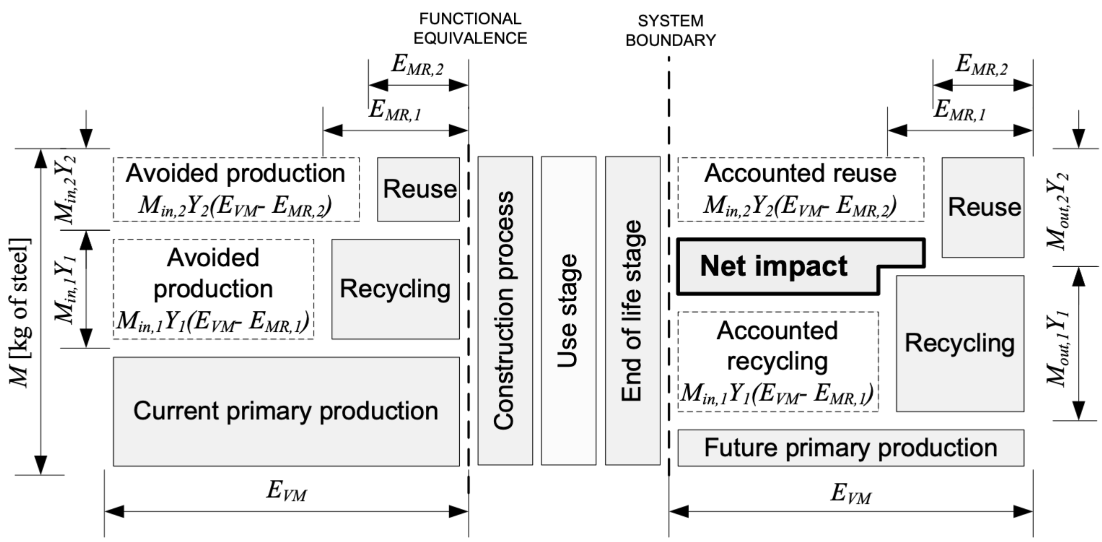


Fig. 3.4 Material flows in steel reuse and recycling

The following sections demonstrate how to declare the environmental and economic impacts of reuse. The methods will be explained by application to a steel-framed building fabricated partly from reclaimed steel sections with the portal frames designed to be reused.

Example 1: Material flows of recycling and reuse

This example shows the calculation of material flows in the hypothetical steel-framed building. The steelwork has total mass 100 t and is fabricated from 10 t of reused steel sections and 90 t steel from the steel mill. The scrap used in the steel mill for production of 90 t new steel is 55 t (close to the EU average). Due to the losses of steel in recycling, 55 t of scrap represents only 50.4 t of the final products (the yield factor of recycling is 0.916 according to [47]) and the remaining 39.6 t is produced from the raw materials. It is estimated that 30 t of the steelwork can be further reused. Then 5 t of the steel will be lost or discarded at the end of life, and the rest will be sent for recycling. The calculation parameters are presented in Table 3.2.

Table 3.2 **Material flows of recycling and reuse**

	Recycling	Reuse
Input of secondary material	$M_{\mathrm{in},1} = 55\ t$	$M_{\mathrm{in},2} = 10\ t$
Output of secondary material	$M_{\mathrm{out},1} = 65\ t$	$M_{\mathrm{out},2} = 30\ t$
Efficiency of the recovery process	$Y_1 = 0.916$	$Y_2 = 1$

3.4 Environmental benefits

Reuse of components generally avoids melting of the material, and therefore the environmental benefit is relatively high compared to the usual recycling option. Although, it is generally impossible to reuse 100% of the recovered steel, the remaining material can be easily recycled and the benefit of recycling has to be considered together with reuse. There are currently several methodologies to declare environmental benefits of combined reuse and recycling.

3.4.1 CEN/TC 350 methodology

The lifecycle information calculated according to the CEN/TC 350 series of standards [35] [36] is divided into four stages:

- A1-3 (Product stage),
- A4-5 (Construction process stage),
- B (Use stage)
- C (End-of-life stage).

Only the impacts of A1-3, B, C and D are mandatory for the Environmental Product Declarations according to EN 15804 [81] (Fig. 3.5). The impacts calculated for the products according to EN 15804 can be further used in the LCA of the building according to EN 15978 [36].

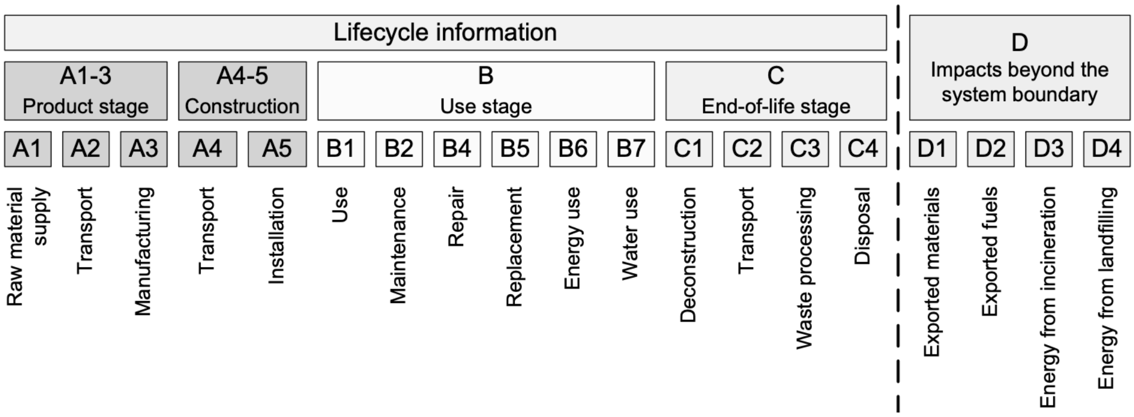


Fig. 3.5 Lifecycle stages in EN 15804 [81]

EN 15804 divides the impacts beyond the system boundary into four sub-modules. Module D1 are burdens and benefits related to the export of secondary materials e_{moduleD1} and are explained in detail in the Annex D of the standard. The calculation of e_{moduleD1} in EN 15804 is presented in Equation (3.7) with the environmental impact of the substituted virgin material $E_{\mathrm{VMSub,out}}$ reduced by the ration of the quality factors of the recycled and substituted virgin material $Q_{\mathrm{R,out}}/Q_{\mathrm{Sub}}$.

$$e_{\mathrm{moduleD1}} = \sum \left(M_{\mathrm{MR,out,i}} - M_{\mathrm{MR,in,i}}\right)\left(E_{\mathrm{MRafterEoW,out,i}} - E_{\mathrm{VMSub,out,i}} \frac{Q_{\mathrm{R,out}}}{Q_{\mathrm{Sub}}}\right) \tag{3.7}$$

In the case of constructional steel, we recommend to neglect the quality factors ($Q_{\mathrm{R,out}}/Q_{\mathrm{Sub}} = 1$) and take into account yield of secondary steelmaking as $M_{\mathrm{MR,out,1}} = M_{\mathrm{out,1}} \cdot Y$ and $M_{\mathrm{MR,in,1}} = M_{\mathrm{in,1}} \cdot Y$.

The point of functional equivalence is recommended to be the same so the impacts of virgin material production $E_{\mathrm{VMSub,out,1}} = E_{\mathrm{VMSub,out,2}}$ show the total impacts of the Product stage (Module A1-3).

Example 2: Global warming potential according to EN 15804

Global Warming Potential (GWP) is one of the specific emissions reported in the Environmental Product Declarations and its value in lifecycle stages A to C is called “carbon footprint” of the product or building in $kgCO_{2e}$. GWP beyond the system boundary is sometimes called “carbon handprint”. In this example, the unit impacts of Product stage $e_{\mathrm{moduleA1-3}}$ is based on the virgin material production, recycling or reuse. Virgin material production and recycling are according to [47] with the added impacts of transport and manufacturing of 0.25 $kgCO_{2e}/kg$ (Modules A_2 and A_3) obtained from the existing Environmental Product Declaration [48], and reuse is considered to be 0 $kgCO_{2e}/kg$ (see Table 3.3). GWP of the construction stage $e_{\mathrm{moduleA4-5}}$ is estimated as 50 tCO_{2e}, and the impact e_{moduleC} of deconstruction is estimated as 70 tCO_{2e} (the deconstruction is more laborious than the assembly because of cleaning and separation of the components). Use stage e_{moduleB} is neglected in this example because of the expected short service life of the single-storey building without the specific maintenance requirements.

Example 2: Global warming potential according to EN 15804 (continuation)

Table 3.3 **Unit impacts used in the example**

	Recycling	Reuse
Unit impact of the recovery process [a]	$E_{\mathrm{MR,1}} = 0.636$ $kgCO_{2\mathrm{e}}/kg$	$E_{\mathrm{MR,2}} = 0$ $kgCO_{2\mathrm{e}}/kg$
Unit impact of the substituted primary production [b]	$E_{\mathrm{VM}} = 2.17\ kgCO_{2\mathrm{e}}/kg$	

[a] 0.386 $kgCO_{2e}/kg$ from [47] plus 0.25 $kgCO_{2e}/kg$ (A_2 and A_3) according to [48].
[b] 1.92 $kgCO_{2e}/kg$ from [47] was increased by 0.25 $kgCO_{2e}/kg$ (Modules A_2 and A_3) according to [48].

The unit impacts combined with the secondary material flows from the 100t steelwork from *Example 1* will result in the burden 88.1 tCO_{2e} in the Product stage $A_{1\text{-}3}$ (see Equation (3.9)) and the benefit of recycling and reuse in Module D 57.5 tCO_2e.

$$M_{\mathrm{MR,out,1}} = 65 \cdot 0.916 = 59.5\ t \text{ and } M_{\mathrm{MR,in,1}} = 55 \cdot 0.916 = 50.4\ t \tag{3.8}$$

$$e_{\mathrm{moduleA1-3}} = \sum M_{\mathrm{MR,in,i}} E_{MR,i} + M_{\mathrm{VM,in}} E_{\mathrm{VM}} =$$
$$= 50.4 \cdot 0.636 + 10 \cdot 0 + (100 - 50.4 - 10) \cdot 2.17 = 88.1\ tCO_2e \tag{3.9}$$

$$e_{\mathrm{moduleD1}} = \sum \left(M_{\mathrm{MR,out,i}} - M_{\mathrm{MR,in,i}}\right)\left(E_{\mathrm{MR,i}} - E_{\mathrm{VM}}\right) =$$
$$= (59.5 - 50.4)(0.636 - 2.17) + (30 - 10)(0 - 2.17) = -57.5\ tCO_2e \tag{3.10}$$

Then the whole lifecycle impact (carbon footprint) of the building is the sum of Modules A, B and C. It is 88.1 + 50 + 0 + 70 = 208.1 tCO_2e with the potential to save 57.5 tCO_2e in the next lifecycle.

3.4.2 Product Environmental Footprint

The multi-criteria assessment methodology called Product Environmental Footprint (PEF) was developed by the European Commission's Joint Research Centre (JRC) [49]. It presents a general assessment of material and energy efficiency of materials that can be reused, recycled, disposed or recovered as energy. The calculation of environmental impacts for different product categories under the so-called Product Environmental Footprint Category Rules (PEFCR) is described in the PEFCR guidance [50] as Circular Footprint Formula (CFF). One example of the PEFCR rules were developed by Eurometaux [51] for metal sheets.

The formula, or its modular version (CFF-M), calculates the whole lifecycle impacts including the end-of-life recovery. The part related to the burdens and benefits beyond the system boundaries presented in Equation (3.11) for materials input and output separately with the material flows R_1 and R_2, unit impacts E_{rec} and E_{V}, and quality factors Q_{S} and Q_{P}. In this calculation, different impacts can be considered in the production and recycling of materials for the current product and in its end-of-life stage. Moreover, impacts of the substituted virgin material E_{V} can be further reduced by the ratio of secondary and primary material quality factors entering the system $Q_{\mathrm{Sin}}/Q_{\mathrm{P}}$ and exiting the system $Q_{\mathrm{Sout}}/Q_{\mathrm{P}}$.

$$input: -(1-A)R_1\left(E_{\mathrm{rec}} - E_{\mathrm{V}}\frac{Q_{\mathrm{Sin}}}{Q_{\mathrm{P}}}\right) \text{ and } output: (1-A)R_2\left(E_{\mathrm{recEoL}} - E_{\mathrm{V}}^{*}\frac{Q_{\mathrm{Sout}}}{Q_{\mathrm{P}}}\right) \tag{3.11}$$

where R_1 and R_2 are the amounts of the recovered material and the material that will be recovered. Allocation factor A (between 0 and 1) determines the amount of impacts beyond the system boundary to be allocated to the lifecycle impacts to produce one “aggregated” result to reflect the market realities. Low value of A means low offer and high demand of recyclable materials, while high value of A means high offer and low demand of recyclable materials. For steel, it is recommended to use the value close to the lower bound $A = 0.2$. Equation (3.12) has shown PEF impacts beyond the system boundary and has form similar to Equation (3.2).

$$X = R_2\left(E_{\mathrm{recEoL}} - E_{\mathrm{V}}^{*}\frac{Q_{\mathrm{Sout}}}{Q_{\mathrm{P}}}\right) - R_1\left(E_{\mathrm{rec}} - E_{\mathrm{V}}\frac{Q_{\mathrm{Sin}}}{Q_{\mathrm{P}}}\right) \tag{3.12}$$

The specific emissions and consumed resources are arising from the recovery of materials used in the manufacturing of the analysed product E_{rec} or from their recovery at the product’s end of life E_{recEoL}. They are subtracted from the specific emissions and consumed resources arising from the acquisition and pre-processing of virgin material in today’s production or at the product end of life, E_{V} and E_{V}^{*} respectively. The formula is able to take into account multiple recycling or reusing technologies if the flows and impacts are calculated according to Equation (3.13) from the flows and impacts of each individual (i-th) recovery process.

$$R = \sum R_{\mathrm{i}} \text{ and } E = \sum \frac{E_{\mathrm{i}} R_{\mathrm{i}}}{R_{\mathrm{i}}} \tag{3.13}$$

Example 3: Global warming potential according to PEF Circular Footprint Formula

The Circular Footprint Formula, using the material flows from *Example 1* and unit impacts from *Example 2* produces the same CO_2 emissions as EN 15804. The total secondary material flows are calculated in Equation (3.14) and (3.15).

$$R_1 = \sum R_{\mathrm{out,i}} = 55 \cdot 0.916 + 10 = 60.4\ t \tag{3.14}$$

$$R_2 = \sum R_{\mathrm{in,i}} = 65 \cdot 0.916 + 30 = 89.5\ t \tag{3.15}$$

The average unit impacts used in the PEF Circular Footprint Formula calculated in Equations (3.16) and (3.17) are also based on the material flows from *Example 1* and unit impacts from *Example 2*.

$$E_{\mathrm{recEoL}} = \sum E_{\mathrm{recEoL,i}} R_{\mathrm{out,i}} / \sum R_{\mathrm{out,i}} =$$

$$= (0.636 \cdot 0.65 \cdot 0.916 + 0 \cdot 0.3)/(0.65 \cdot 0.916 + 0.3) = 0.423\ kgCO_2e/kg \tag{3.16}$$

$$E_{\mathrm{rec}} = \sum E_{\mathrm{rec,i}} R_{\mathrm{in,i}} / \sum R_{\mathrm{in,i}} =$$

$$= (0.636 \cdot 0.55 \cdot 0.916 + 0 \cdot 0.1)/(0.55 \cdot 0.916 + 0.1) = 0.531\ kgCO_2e/kg \tag{3.17}$$

Then the benefit of reuse and recycling in Equation (3.18) is the same as calculated in *Example 2*.

$$X = R_2\left(E_{\mathrm{recEoL}} - E_{\mathrm{V}}^{*}\frac{Q_{\mathrm{Sout}}}{Q_{\mathrm{P}}}\right) - R_1\left(E_{\mathrm{rec}} - E_{\mathrm{V}}\frac{Q_{\mathrm{Sin}}}{Q_{\mathrm{P}}}\right) =$$

$$= 89.5 \cdot (0.423 - 2.17) - 60.4 \cdot (0.531 - 2.17) = -57.5\ tCO_2e \tag{3.18}$$

The aggregated environmental footprint according PEF methodology is then subtracting 80% of this benefit from the remaining part of the calculation and the result is:
208.1 - 0.8·57.5 = 162.1 tCO_2e.

3.5 Economic benefits

3.5.1 Life cycle costs

Economic benefits of reuse can be calculated if the costs arising in the product or building lifecycle are properly assessed. The basic principles of lifecycle cost assessment (LCC) are developed in ISO 15686-5 [52] and the most common LCC method in Europe is described in EN 16627 [37]. However, how the benefits are shared between the actors of the value chain depends mainly on the value of the component or the structure (i.e. how much is the buyer willing to pay for the constructional steel that may be fully functional, but otherwise less suitable than the new product for instance due to aesthetic reasons or because it is optimized for different external conditions).

The value of the steelwork or steel-based components integrated in the building, is for simplicity, illustrated here as the market price of such building or component in a given place and time. As can be seen in Fig. 3.6, the value of the building designed for the specific purpose after fabrication and erection "A_5" depends on the costs of materials, manufacturing and assembly (including transport and storage costs).

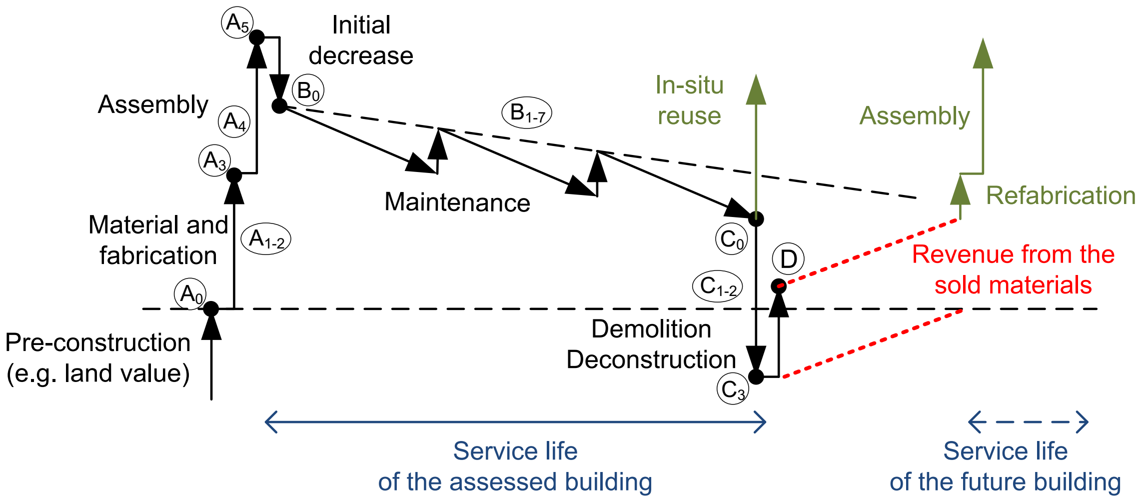


Fig. 3.6 Development of the economic value of the single-storey building or steelwork and steel-based components integrated in such building

It is assumed that immediately after the construction of the building, the value may decrease to point "B_0" in Fig. 3.6. This is caused especially by the customisation choices in building and product design that could not be fully exploited by the new owner. Then the value changes depending on the deterioration of the materials, surface finishes (aesthetic value) and development of the market prices of new buildings in the area. It is generally assumed that the value has decreasing trend until the decision about refurbishment or maintenance action is made to restore the value of the property.

Single-storey steel structures are typically designed for a design life of 50 years according to the Eurocodes and their life can be further extended by testing and re-evaluation of structural integrity, stability and serviceability, and therefore the deterioration of value in the use phase

"B" is expected to be rather slow. However, the real service life of such buildings is usually less than 30 years until they reach point "C_0" (end-of-Life) when the building as such cannot be used anymore.

Deconstruction and separation of structural components can be divided into more stages (see Fig. 3.7), since the components and envelopes can be recovered at different levels:

- No deconstruction at all will lead to in-situ reuse (D_0) without physically removing the component from the structure. The components can still be re-designed and modified.
- Disassembly of the steelwork will allow reuse of the whole structure or its part (D_1) that can act as the whole structure (e.g. single bay of the multi-span building)
- Separation of the components of disassembled structure for reuse of the components (D2), is done typically by opening bolt connections for instance when the sandwich panel, column or truss girder is cleaned, repainted, modified to fit the new design and installed again.
- Extraction and reconditioning of the constituent products, such as sections with separated end-plates and cleats will allow those products to be sold and reused also for different purpose (D3).
- Separation of steel scrap to be recycled is the lowest level of recovery (D4).

Different reuse or recycling process will result different residual value "D" as shown in Fig. 3.7, where the residual values D_0, D_1, D_2 and D_3 are associated with in-situ reuse, reuse of the whole structure, fabricated components or constituent products respectively. D_4 is then the residual value of the baseline scenario with the steel scrap collected for recycling.

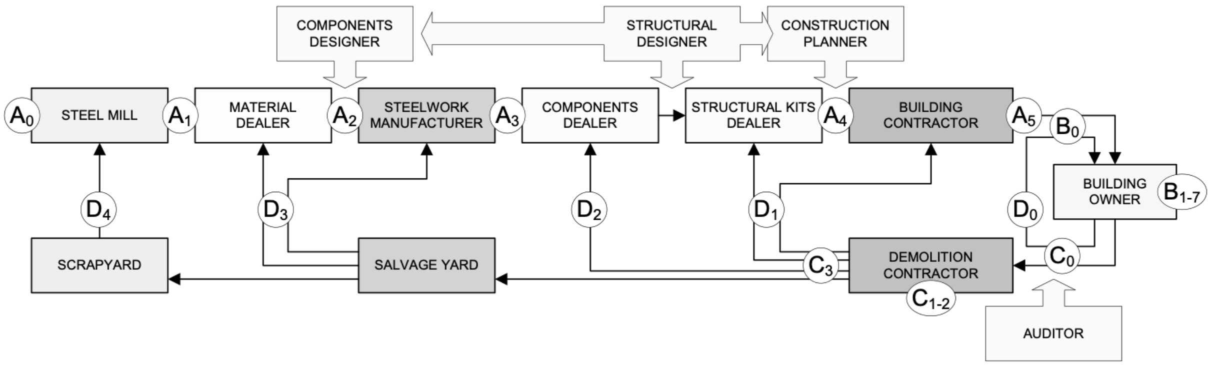


Fig. 3.7 General description of different reuse flows and the participants involved in the particular lifecycle stages

Traditional single-storey buildings are designed and optimized to certain dimensions (such as floor area, height and span) and conditions (e.g. snow load), which decreases their value in the next service life. For instance, it is more likely that the new owner needs to purchase larger reused structure than if he orders a custom-made new one. Therefore, the immediate "initial decrease" of the building value (point "B" in Fig. 3.6). Flexible and modular design is a solution that can minimise such gap as the new owner will have more freedom to re-arrange the structural layout to match his needs.

The estimation of the cost of deconstruction of a single-storey steel building for reuse is difficult, because of the lack of documented case studies. Generally, it is expected that the deconstruction cost is higher than demolition cost and higher than the cost of building assembly. The problematic part of deconstruction is usually not the steelwork itself (because it can be typically disassembled with the same effort as assembled), but the materials and structures attached to it or preventing the access to the steelwork connections. Selective removal of those materials and structures can be a significant barrier to the whole reuse process.

Design for deconstruction is therefore a holistic approach where all the building layers and services are carefully planned so they can be removed in the same “reversible” manner as the structural steelwork itself.

3.5.2 Cost savings according to CEN methodology

In order to calculate the potential costs and revenues related to the substitution of resources beyond the system boundary of the product according to the CEN/TC 350 series of standards, the scenarios for reuse, recycling and recovery of the whole building or its materials need to be described.

EN 16627 [37] describes the Module D as the income relating to exported energy and secondary materials, secondary fuels or secondary products resulting from reuse, recycling and energy recovery that take place beyond the system boundary. Any income which is received by the building owner from the sale of the land, waste processing for reuse, recovery or recycling shall be allocated to Module D. Where a material flow exits the system boundary and has an economic value or substitutes another product, then the incomes may be calculated and shall be based on typical currently available technology and current practice.

Information in Module D according to EN 16627 can be calculated as the potential net income from the reuse of the construction frame or structure of the building after its end-of-life and/or the potential net income for the building owner resulting from the sale of products and materials for reuse, recycling or recovery. Therefore, module D depends on the processes in the lifecycle stages A, B and C. The calculation is based on the quantified mass flows (the amount of secondary products) and its assumed unit price.

In the case of constructional steel, it is recommended that the calculation of net income is consistent with the calculation of net environmental impacts according to EN 15804 [35] and Equation (3.5). The net income will be then expressed the same way as the remaining modules, i.e. positive value will be cost and negative value will be profit associated to the secondary material flows.

The economic assessment shall be calculated excluding VAT, and the VAT treatment shall be reported separately, as it will be dependent on the tax status of the client and project.

If the LCC analysis is used to compare different investment options, results may be expressed as their net present value. The net present value is the sum of the discounted future cash flows, both costs and benefits/revenues.

Example 4: Lifecycle cost benefits according to EN 16627

This example illustrates the calculation of lifecycle costs of the steelwork described in *Example 1*. It is based on the modular approach from the environmental assessment in *Example 2*. The unit costs of different processes are based on the model from [53]. It is assumed that the steelwork manufacturer sells the finished product for 1444 €/t and (for the simplicity) will have no profit or loss if the production uses virgin raw materials or recycled scrap. With the assumption that the scrap price is 200 €/t, we can write the unit costs $C_{\mathrm{VM}} = 1\,444\,€/t$ for steel produced entirely from virgin materials and $C_{\mathrm{MR,1}} = 1\,244\,€/t$ for 100% recycled steel. Then, depending on the reuse scenario, the costs associated with the production of the same product from reused steel (to the point of functional equivalence) may vary. For instance, the costs may be $C_{\mathrm{MR,2}} = 920\,€/t$ for steelwork from 100% reused constituents. With the knowledge of material amounts entering and leaving the system from *Examples 1* and *2*, the Equations (3.5) to (3.6) can be followed to calculate properly Modules $A_{1\text{-}3}$ and D.

$$c_{\mathrm{moduleA1-3}} = \sum(M_{\mathrm{in,i}} Y_i) C_{\mathrm{MR,i}} + M_{\mathrm{in,VM}} C_{\mathrm{VM}} =$$

$$= (55 \cdot 0.916) \cdot 1244 + 10 \cdot 920 + (90 - 55 \cdot 0.916) \cdot 1444 = 129\,055\,€ \qquad (3.19)$$

$$c_{\mathrm{moduleD1}} = \sum(M_{\mathrm{out,i}} - M_{\mathrm{in,i}})(C_{\mathrm{MR,i}} - C_{\mathrm{VM}}) Y_{\mathrm{i}} =$$

$$= (65 - 55)(1244 - 1444) \cdot 0.916 + (30 - 10)(920 - 1444) = -12\,312\,€ \qquad (3.20)$$

The calculated benefits (net revenue) 12.3 k€ show the potential savings in the lifecycle costs of a steelwork manufactured from the reused constituents. It can be significantly improved if the calculation takes into account that the fabricated components are reused at the end-of-life stage without reducing them to the constituent products. The costs of reconditioning of such components is significantly lower $C_{\mathrm{MR,3}} = 460\,€/t$ in the cost model from [53]. Then the third material loop has to be added to the calculation (see Table 3.4) and the result of the modular calculation according to Equation (3.5) is presented in Equation 3.21.

Table 3.4 **Material flows of recycling and reuse**

	Scrap	Constituents	Fabricated steelwork
Input of secondary material	$M_{\mathrm{in,1}} = 55\ t$	$M_{\mathrm{in,2}} = 10\ t$	$M_{\mathrm{in,3}} = 0\ t$
Output of secondary material	$M_{\mathrm{out,1}} = 65\ t$	$M_{\mathrm{out,2}} = 0\ t$	$M_{\mathrm{out,3}} = 30\ t$
Efficiency of the recovery process	$Y_1 = 0.916$	$Y_2 = 1$	$Y_3 = 1$

$$c_{\mathrm{moduleD1}} = \sum(M_{\mathrm{out,i}} - M_{\mathrm{in,i}})(C_{\mathrm{MR,i}} - C_{\mathrm{VM}}) Y_{\mathrm{i}} =$$

$$= (65 - 55)(1244 - 1444) \cdot 0.916 +$$

$$+(0 - 10)(920 - 1444) +$$

$$+(30 - 0)(460 - 1444) = -26\,112\,€ \qquad (3.21)$$

The example demonstrated the reuse benefits in lifecycle costing with the modular approach. Equation (3.20) showed the conservative assumption that the end-of-life reuse will be the same as in the beginning of service life when the constituent products were recovered from the existing buildings. If the components are designed for future reuse, the benefits can be higher (see Equation (3.21)), but the calculation needs to be expanded into three different material loops (recycling scrap, constituents and fabricated products). The reuse benefits can be also expressed as the residual value of the steelwork, but this aspect is not covered in the Example 4.

3.5.3 Residual value

In case of reuse, the residual value of the building or components for its owner is the revenue obtained for the sold secondary materials such as steel scrap or reusable components (the value of the recovered material). It can be expressed as net present value for the purposes of planning initial investment.

The calculation of the revenue for the reusable components can be based on two assumptions:

- The value of the steelwork or components will correspond to the market value of the new steelwork or components after re-fabrication and relevant certification.
- The steelwork or components will be sold as second-hand products with lower performance (e.g. aesthetic), and therefore their value should be lowered. In this case, the reduction is estimated as 20% [54].

It should be noted that in any case, the reused steelwork will have to be CE-marked according to EN 1090.

Part 1: Recommendations for existing single-storey buildings

4 RECOMMENDATIONS FOR EXISTING SINGLE-STOREY BUILDINGS

4.1 General approach

The overall process from reclamation to reuse of steel components is summarised in the flowchart in Fig. 4.1. The scope is limited to buildings first constructed 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 the building components that can be reused. Pre-deconstruction audits are described in Section 7.5.

From this initial building inspection, a recommendation is made as to whether the steel components can be reused, or if demolition is the more sensible option. If the steel products can be salvaged, it is important to define the anticipated 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 about the assessment of the reusability of reclaimed elements is given in Section 6.1. The materials should then be sampled and, if needed, tested according to the protocol in Appendix A. The structural reusability of the existing elements is then re-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 (see Section 7.5.4), and all elements labelled and batched. Often, the components have to be cleaned to remove coatings and accumulated dirt or subjected to other reconditioning processes (see Section 7.6). Finally, the structural design and verification of the reclaimed steel members and other components is carried out for the chosen reuse scenario (see Section 8).

4.2 Design procedure

The design of structures made from reclaimed steel members follow the same principles of limit state design and verification by 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 theory [5].

Fig. 4.2 gives a general overview of the Eurocode-based design philosophy. The approach for general 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 for the building and its stability system.

- The design values of the effects of actions, based on the relevant standards, and the measured geometric data are obtained.
- The design values of the material strengths are determined for the structural steel members, see Section 6.2.1.
- Scheme design of the structure is completed based on this input information and various options are presented to the client
- At this stage, it will be apparent whether the possible use of reclaimed steel components is both practical and viable.
- Final design of the structure is then performed taking account of client feedback to the proposed scheme 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 satisfy the new design requirements, the structural system will have to be modified or strengthened accordingly.

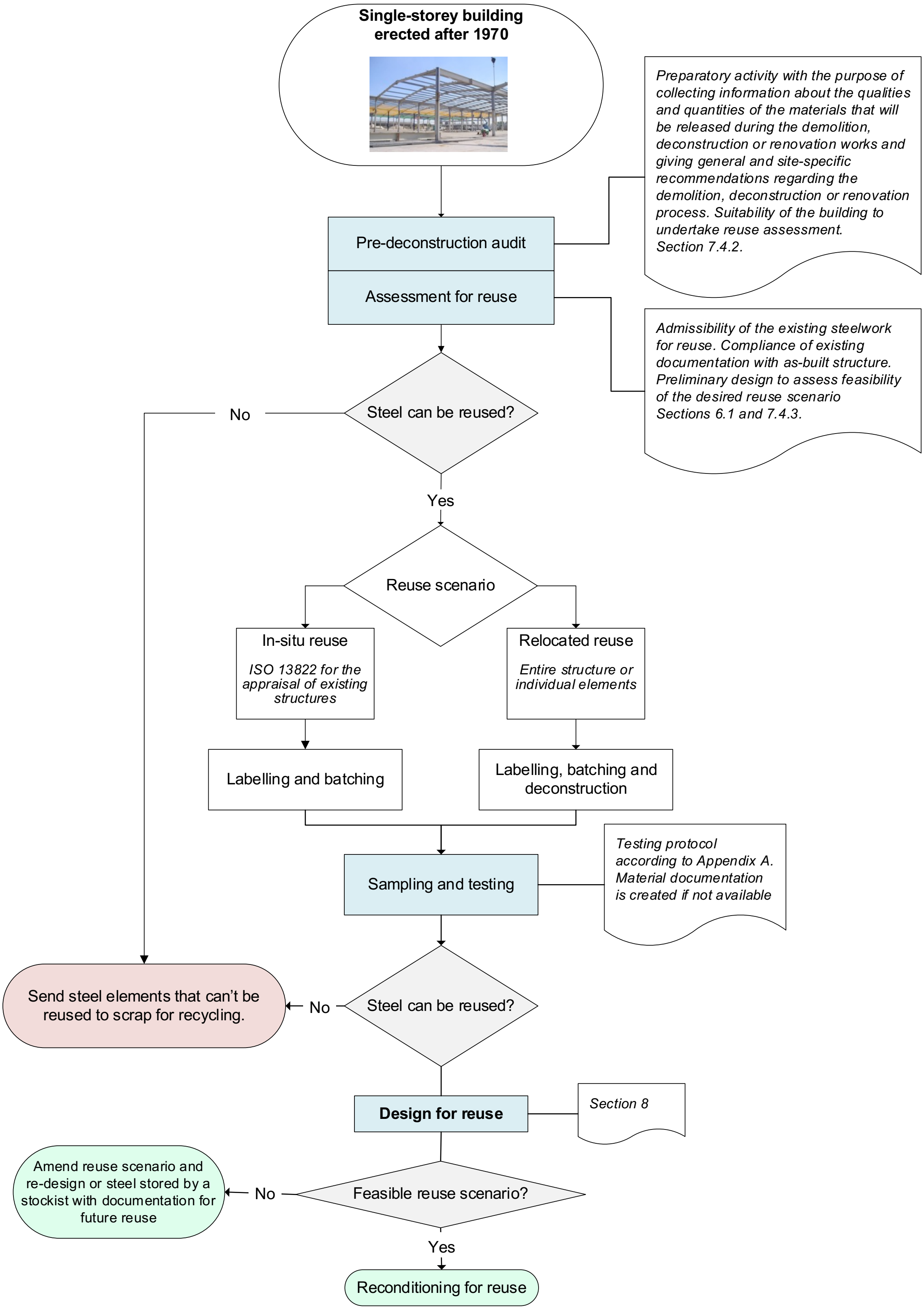


Fig. 4.1 Overall process: from reclamation to reuse and design of steel products

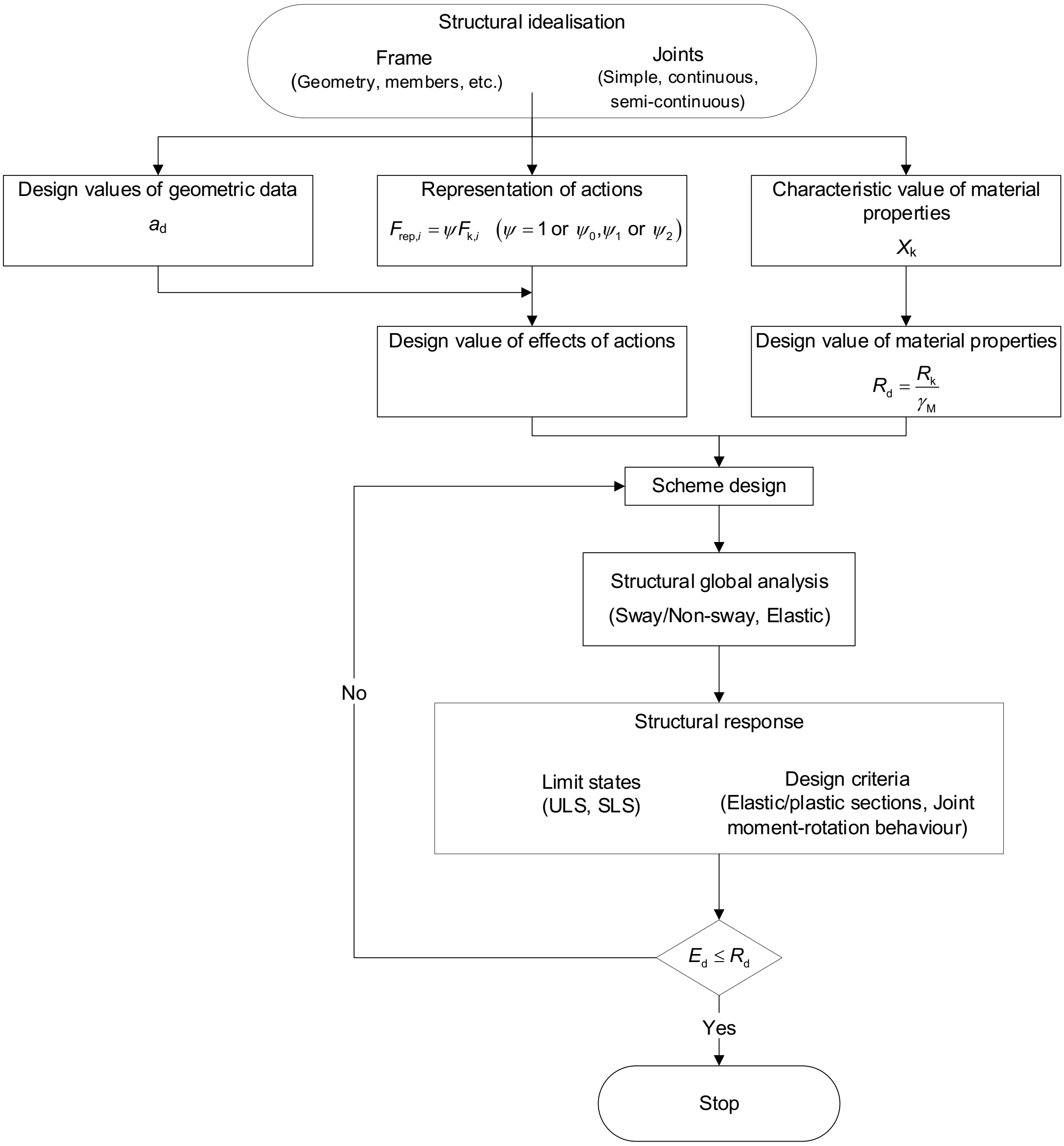


Fig. 4.2 Procedural overview of frame design

5 HISTORICAL REVIEW OF CODES OF PRACTICE AND PRODUCT STANDARDS FOR STRUCTURAL STEELWORK

5.1 General

A knowledge of the history of structural steel is important if reuse of steel members and other steel components is to be widely adopted. During the 1970s, that 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 was required in order to meet country-specific standards. This is demonstrated in Fig. 5.1 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. 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 single-storey structure from which the steel members are to be salvaged, was originally designed and specified based on the standards given in Fig. 5.1 or EN 1993. Design was based on the Limit State principles so that the probability of each limit state being reached is substantially constant for all members in a structure, and is also at an acceptable low level.

5.2 Hot rolled structural steels

5.2.1 Product standards

The designation of steel products in EN 1993 is in accordance with EN 10025-2:2004 [15]. Fig. 5.1 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. The material properties for structural steel are defined in Clause 3.2.6 of EN 1993-1-1, and these properties do not change over time: The elastic modulus of all grades of steel is E = 210000 N/mm^2, and Poisson's ratio $\nu = 0.3$, The Coefficient of linear thermal expansion α = 12×10^{-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 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.

			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-NX, *X* = 1, 2, 3, 4 ,5	BS 5950 (after 1985) BS 449 (before 1985)

Fig. 5.1 National product and structural design standards before 2004

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 may be enhanced by minimising the sulphur levels, and toughness may 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.

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

5.2.2 Codes of practice and standards for design

The first European standard for the design of steel structures was issued in 1992, as a Pre-standard ENV 1993-1-1 [55]. It was intended to be a framework for preparing 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 are widely applied in all European countries and have generally replaced all National Structural Design Standards, see Fig. 5.1. Eurocodes also provide a means of ensuring public safety throughout the EU.

Another important part of the Eurocode 3 is the way in which this is integrated with product standards to allow CE marking to support the Construction Products Regulation (CPR) [56].

5.3 Practice for 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 [57], followed by ENV1993-1-3:1996 [58]. This European document has been 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* [11]. EN1993-1-3:2006 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 EN1993-1-1 and EN1993-1-5. EN1993-1-3 permits only design by the limit states method (LSD).

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

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

The design provisions for these particular applications are often complex but may be useful for design engineers since they include detailed methodologies not available in other standards or specifications.

Cold-forming technology enables the production of unusual sectional configurations. However, from the point of view of structural design, the analysis and design of such unusual members may be very 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 code procedures. Numerical FEM analysis is always an alternative for the design, but for many practical situations, modelling can be very complicated. For such complex design problems, modern design codes permit the use of testing procedures to evaluate structural performance. Testing can be used either to replace design by calculation or combined with calculation. Only officially accredited laboratories are able to perform such tests and produce the relevant certification.

6 EVALUATION OF STRUCTURAL REUSABILITY

6.1 Parameters influencing reusability

An important aspect when assessing structural steel for reuse is that it should be damage-free when salvaged from its previous use. Therefore, the structural members should not have significant imperfections or permanent deformations nor have corroded in terms of material loss other than surface effects, and should not have not been subjected to extreme events such as impact, or fatigue due to repeated loading (for example from machinery) and fire damage.

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 due to the effects 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 preparation and application of surface paint systems or metallic coatings by means of thermal spraying or galvanising.

Ageing of material is the gradual deterioration (due to time or use) mostly 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 of 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 material property changes 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 at 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 undergoing 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 by slow plastic deformations at high temperature, typically during a fire. The creep strength in the range of temperatures where creep applies is always lower than the material yield strength.

6.2 Structural steel for reuse

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

The definition of structural steel in this context is the steel from members made from hot-rolled sections and their end connections, including trusses made from rolled members. Fabricated (built-up) 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 additional verifications for the specific reuse scenario. Secondary members made from cold-formed sections may also be reused as they are or cut down lengths, although they are not normally considered under the definition of structural steel.

6.2.1 Classification of reclaimed steelwork

Reclaimed steel should be classified based on the verification of its material performance requirements (adequacy assessment), and quality assurance requirements (reliability assessment) into 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 re-certification.
- **Class C**: steel materials classified as the most conservative grade in accordance with structure age and location (unidentified steel).

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

6.2.2 Material performance requirements

EN 1090-2 for the execution of steel structures (i.e. fabrication and erection) allows non-conforming structural steel members for constituent products to be specified. The following mechanical properties have to be determined according to EN 1090-2 clause 5.1 [4]:

- Strength, i.e. yield strength, f_y, and the tensile strength, f_u,

- Elongation after fracture, ε_f, that gives information on how much the material deforms,
- Heat treatment delivery condition.

The nominal yield strength should be in the range of 235 N/mm^2 and 460 N/mm^2. The minimum nominal tensile strength should be in the range of 360 N/mm^2 to 550 N/mm^2. The ductility requirements for design to EN 1993 are presented in Table 6.1 (recommended values that may be modified by the NAs).

Table 6.1 **Ductility requirements (CEN recommended values)**

Yield ratio f_u/f_y	Elongation at failure	Ultimate strain ε_u
≥ 1.10	$\geq 15\%$	$\geq 15\dfrac{f_y}{E}$

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 has to 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 should the reclaimed material need to be recertified, due to absence of original certificates. The steel weldability shall be declared as follows [4]:

- Classification in accordance with the materials grouping system defined in CEN ISO/TR 15308, 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.

6.2.3 Quality assurance requirements

Reclaimed steel has to meet certain quality and safety requirements in order to be *re-certified* to ensure its reliability 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 for product conformity, quality and traceability.

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 proper identification and quality assurance system. Suppliers and fabricators who intend to reclaimed structural steel materials have to establish an in-house quality assurance system to ensure the traceability of such materials. Each steel member shall be marked with a unique identification number of which quality control checks are introduced and recorded. Such 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, then it is possible to trace back the reclaimed steel components in order to check that they meet the relevant material specifications and reliability requirements.

New steel materials are sourced with valid factory production control certificate and manufacturer test certificate based on the delivery specification, whereas reuse of the material is permitted with satisfactory verification against its reusability.

6.2.4 Adequacy and reliability assessments

Generally, reclaimed steel is assessed for adequacy and reliability, which are closely interrelated. Steel usually has to be certified, i.e. actual material properties have to be evaluated against the material performance requirements. In the absence of such certificates, material testing through appropriate sampling and testing method(s) should be carried out to demonstrate the adequacy of the material. The reliability assessment is to ensure that the steel products are manufactured under stringent quality assurance system and that it meets the quality assurance requirements.

Materials can be classified on completion of these assessments, which are obviously interrelated, according to the system proposed above and in accordance with the flow chart shown in Fig. 6.1. The classification is necessary to establish whether the reclaimed steel material can be allowed for structural use to EN 1993 with or without any restrictions.

Class A reclaimed steel material can be designed to EN 1993, as appropriate adequacy and reliability assessment are justified by the 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 e.g. confirm the grade of the reclaimed steel. See also section 8.2.3.

Class B material complies with the material performance requirements through a comprehensive material testing (see Appendix A), and have approved quality assurance by re-certification. The testing procedure comprehends a combination of non-destructive and destructive tests, e.g. to EN ISO 6892-1, together with inspection of geometric tolerances. More conservative values for the partial factor γ_{M1} for designs to EN 1993 are recommended, as some uncertainty in the member and section imperfections are recognised (discussed later).

Finally, reclaimed steel material should be classified as C if it remains unidentified steel, free of damaging defects, and may be permitted to be used for non-safety critical structures, e.g. agricultural buildings, or members that do not require CE marking. In this situation, it should be assumed that the steel is of the weakest grade of structural steel in use at the time of its first use. Relevant material product standards and design codes based on the structure erection date can be used. See also section 8.7.

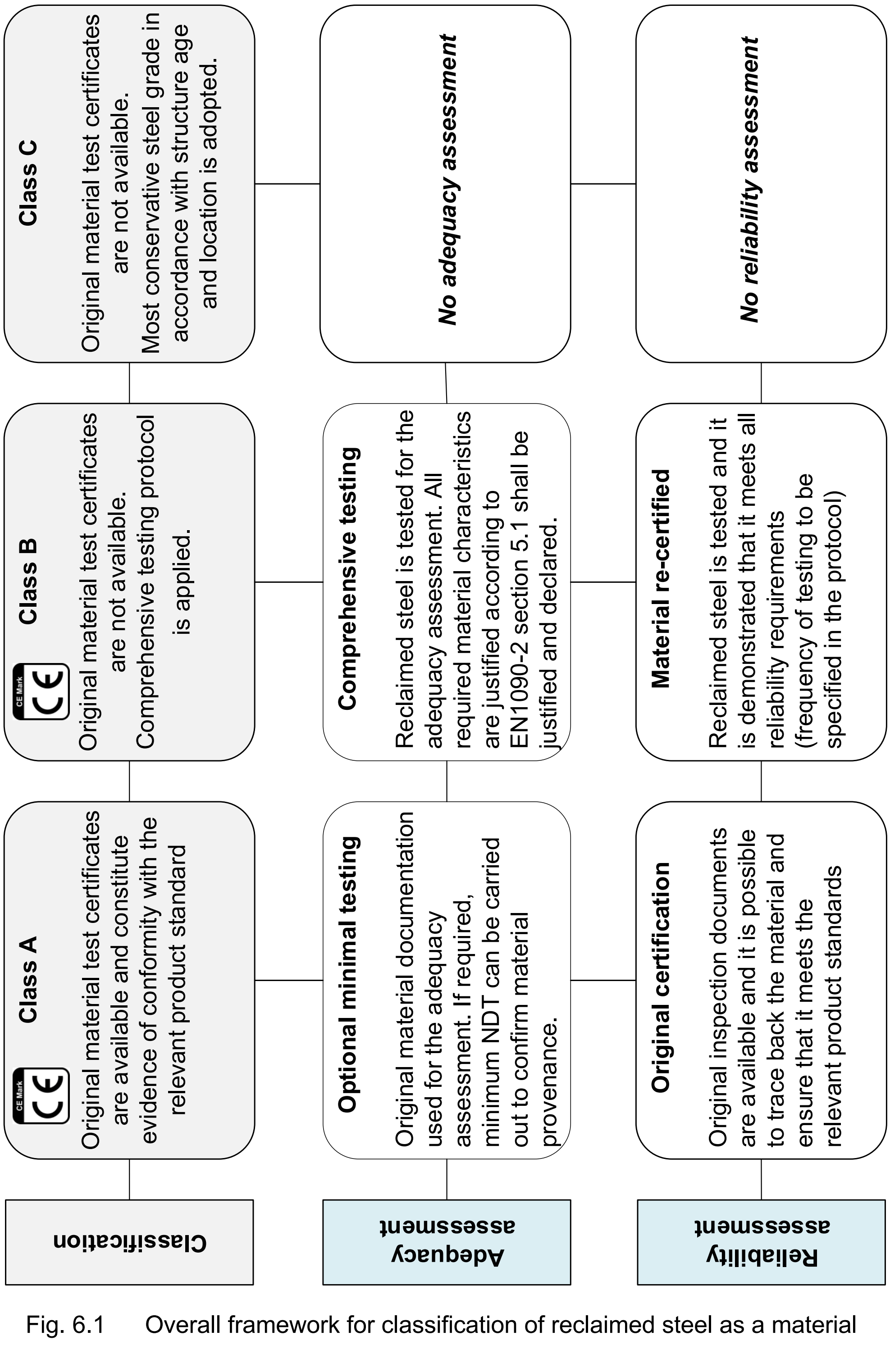


Fig. 6.1 Overall framework for classification of reclaimed steel as a material

The adequacy assessment will be based on existing documentation or testing procedures. For Class C, no adequacy assessment is undertaken, which means that material characteristics (i.e. mechanical and chemical properties) are assumed based on the steelwork age and location. Fig. 6.2 summarizes the adequacy assessment framework for reclaimed steel products.

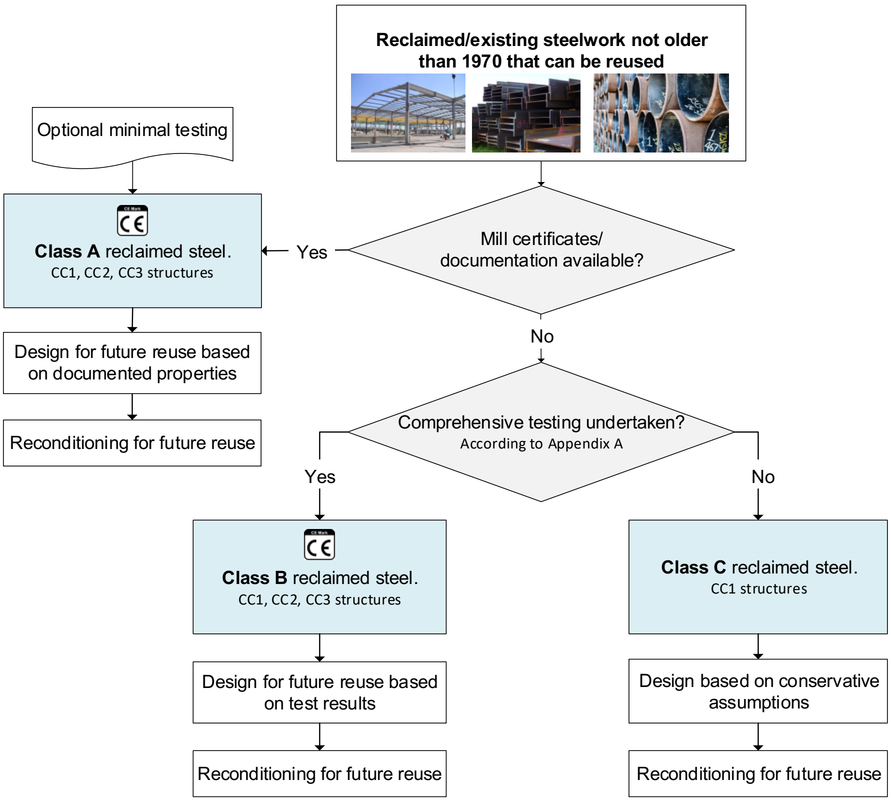


Fig. 6.2 Overall framework to undertake the adequacy assessment of steelwork

After undertaking the adequacy assessment of the reclaimed steelwork, a reliability assessment is required to ensure that the reclaimed product can be used in structural design according to EN 1993-1-1. The basic requirement of this assessment is essentially related to the fact that EN 1993 relies on mean values of yield and tensile strengths to specify the partial factors for cross sections and members resistances which meet the reliability requirements according to EN 1990. To undertake such assessment, test results must meet certain minimum values for yield and tensile strengths.

6.2.5 Alternative specification of source material

Unused, unfabricated steel components might be placed on the market having been manufactured to an alternative material standard, for example steel manufactured to a non-European material or manufacturing standard. This unused material would be expected to have appropriate original certification/documentation declaring the material properties. A declaration of the material properties must be provided by the stockholder.

If the steel can be shown to comply in all respects with a weldable structural steel reference Standard (as listed in Section 1.2.2 of EN 1993-1-1), and tolerances within the limitations of EN 1090-2, the steel can be used in design, using the procedures specified in EN 1993-1-1 and without modification of the γ_{M1} value as proposed for reclaimed steelwork, as long as the steelwork was never erected.

6.2.6 Certification for reuse

CE marking applies to the manufacture of structural steel components, which are produced from constituent products (i.e. steel sections, mechanical fasteners and welding consumables).

The properties of supplied constituent products have to be documented in a way that enables them to be compared to the specified properties and ensure that they conform to the relevant product standard. These documents include inspection certificates, test reports, declaration of compliance as relevant for structural steels so that their provenance is fully established.

The structural engineer should then specify a testing programme from the required list of properties in Section 6.2.2, bearing in mind that it is not mandatory to fully characterise all properties. The number of tests and sampling locations will depend on the particular circumstances and should ultimately be decided by the engineer taking account of factors including:

- The likely variation in material properties within and between parts of the structure;
- The probable critical members' locations;
- The possible errors in the test procedure and associated deviations in the results obtained.

Materials evaluation usually involves a combination of on-site/off-site non-destructive testing (NDT) and testing of samples using Destructive Testing (DT). EN 10025-1 states that, for “new” steels, the following samples shall be taken from one sample product of each test unit:

- One sample for tensile testing;
- One sample sufficient for one set of six impact test pieces if the impact test is required for the quality, as specified in EN 10025-2 (or any other relevant part of EN 10025).

The procedure consists of the following parts:

- Batching the products: categorise members by groups, e.g. according to size. Assign a unique identifier to each member in each group, e.g. using consecutive integer identifiers;
- Sampling within each group: select samples of members for testing. Samples are selected randomly: each member of the group has the same probability of being sampled;
- Testing the samples using destructive techniques;
- Statistical judgement of the results;
- Decision regarding acceptance.

The quality control can be either total or statistical. If the control is total, all of the reclaimed steel products are tested, using NDT and DT (or a combination of the two). The acceptance rules imply that the statistical analysis of the results will be judged as good (accepted) or bad (not accepted). This requires the specification of some kind of accepted measurement error. The statistical parameter “Coefficient of Variation” (CoV or V_x) is a measure of the dispersion of results within a population and can be used as an acceptance rule. If the control is statistical, only a limited number of products are tested. The statistical control is adequate, unless it is not possible to provide a statistically adequate sampling. Other cases exist that may justify total control, e.g. different material grades are found within the same group of elements (test unit), or in cases where material re-certification is required.

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).

Reclaimed structural steel products for use in new design situations have to be traceable to a CE marked Type 3.1 or Type 2.2 Inspection Certificate (see EN 1090-2), which are essentially documents that contain the chemical and mechanical properties of the steel that assures that the steel product meets the specified properties. This poses some problems when the original material certificates are not available and, as a consequence, the material has to be re-certified. This implies classification of material as Class B (see above).

Inspection documents previously/also known as mill or test certificates, are supplied with all new rolled steel sections and plate supplied to the steelwork contractor. EN 10204 [24] defines the different types of inspection documents that include Type 2.1, 2.2, 3.1 and 3.2 certificates.

An important distinction exists between specific and non-specific inspection certificates:

- Non-specific inspection is defined (in EN 10204) as inspection carried out by the manufacturer in accordance with his own procedures to assess whether products defined by the same product specification and made by the same manufacturing process, are in compliance with the requirements of the order or not. Types 2.1 and 2.2 are based on non-specific inspection. The products inspected are not necessarily the products actually supplied;
- Specific inspection is defined as inspection carried out, before delivery, according to the product specification, on the products to be supplied or on test units of which the products supplied are part, in order to verify that these products are in compliance with the requirements of the order. Type 3.1 and 3.2 inspection documents are based on specific inspection. A type 3.2 certificate means that products where tested by a third-party accredited entity.

The type of inspection document required for (new) hot-rolled structural steels is presented in Table B.1 in EN 10025-1. Only the steel manufacturer can provide an inspection document to EN 10204. However, clause 12.2.1 of EN 1090-2 states that:

"*Documents supplied with constituent products in accordance with the requirements of Clause 5 shall be checked to verify that the information on the products supplied matches those in the component specification. These documents include inspection certificates, test reports, declaration of compliance as relevant for plates, sections, hollow sections, welding consumables, mechanical fasteners, studs etc.".*

Test reports and declarations of compliance for the reclaimed steel can be provided by a stockholder or other entity responsible to undertake the adequacy and reliability assessment of the reclaimed steelwork. EN 1090-2 also gives requirements for inspection documents for metallic products in Table 1. It is clear that the inspection is intended to guarantee a minimum characteristic yield strength, where an inspection document 3.1 is required.

For reclaimed steel, as material characteristics are justified for a group of reclaimed elements, the documentation created for that group will provide the same level of reliability as a 3.1 certificate. If destructive tests are performed by an external accredited laboratory, a document equivalent to a certificate 3.2 can potentially be issued.

Clause 5.2 in EN 1090-2 states that for execution class EXC3 and EXC4, constituent products shall be traceable at all stages from receipt to hand over after incorporation in the works. This traceability may be based on records for batches of product allocated to a common production process, unless traceability for each individual constituent product is specified. For EXC2, EXC3 and EXC4, if differing grades and/or qualities of constituent products are in circulation together, each individual constituent product shall be designated with a mark that identifies its grade and its quality. Same principles must be applied for groups of reclaimed steel. Methods of marking shall be in accordance with that for components given in 6.2 of EN1090-2. If marking is required, unmarked constituent products shall be treated as nonconforming products.

Cl 6.2 of EN 1090-2 addresses identification of steel components and states: “At all stages of manufacturing each piece or package of similar pieces of steel components shall be identifiable by a suitable system. Identification may be achieved as appropriate by batching or by the shape and the size of the component or by the use of durable and distinguishing marks applied in a way not producing damage”. A similar procedure shall be applied when dealing with reclaimed steel elements.

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

This section summarizes the steel properties that need to be assessed for reclaimed hot rolled steel elements according to EN1090-2 clause 5.1 (including hollow sections – Table 6.2). Further commentary for these properties is also provided.

Table 6.2 **Material properties to be declared according to EN 1090-2 clause 5.1**

Property	**To be**	**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 o dimensions and shape	Yes	Based on dimensional survey.
Impact strength or toughness	If required	If required, determined by destructive tests. Conservative assumption as the
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**	**Procedure**
Classification in accordance with the materials grouping system defined in CEN ISO/TR	-	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	Yes	Determined by non-destructive and destructive tests.

Strength

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 structural 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 EN 1090-2 clause 5.1, which needs to be determined by a destructive tensile testing. Based on historical data, there are no concerns that structural steel reclaimed from buildings erected after 1970 will not meet the design requirements according to Eurocode 3 (see Table 6.1) - [59] to [61]. The minimum elongation requirement for reclaimed steel shall be taken from Table 6.1 and not from the reference product standard.

Tolerances on dimensions and shape

Reclaimed elements can be checked against geometric tolerances according to the relevant product standard (see Table 6.3). 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 6.3 are not met, as long as the design considers measured section properties rather than tabulated standard section sizes. Member bow 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. If through thickness properties are required, reclaimed plate must be tested as specified in EN 1993-1-10 [13].

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 27 J at 20°C can be assumed if no testing is performed (JR subgrade) - [59] to [61]. If material toughness must be determined, destructive tests are required in accordance with the requirements of 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 to EN 10219 or hot finished 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. Available material documentation may allow for the heat treatment delivery condition specification (see section 6.2.5).

Declaration of chemical composition

Chemical composition is necessary to establish the durability but most importantly the weldability of the 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 section 7.2 or EN 10219-1 section 6.6), from which the carbon equivalent value (CEV) can be calculated.

6.2.8 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 re-fabricated, reclaimed structural steelwork can be CE Marked in accordance with EN 1090.

In addition to careful control of the fabrication process, material properties must be declared according to EN 1090-2 clause 5.1 if no material certificates/documentation is available. When using reclaimed steel, declaring such properties according to EN 1090-2 clause 5.1 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 make sure that they meet the fabrication requirements of EN 1090-2 (see section 7.6.5 and Appendix A).

6.3 Constituent products

6.3.1 General

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 fabrication and construction of another structure. This may include hot-rolled and cold-formed steel profiles. Such products will need a new Declaration of Performance (DoP) according to the corresponding harmonised standard because it will be marketed as new product.

Steel sections, plates, and bars used as members must be supplied with dimensions and tolerances that comply with the standards from Table 6.3 and structural hollow sections with those listed in Table 6.4. Table 6.5 provides limiting plate thicknesses for UK practice assuming steelwork to be welded with “moderate” and “very severe” details according to references [62] and [63] for a stress level equal or more than 0.5 x $f_y(t)$. Alternatively, a fracture mechanics approach that conforms to the recommendations given in EN 1993-1-10 may be used to determine the toughness requirements.

Table 6.3 **Rolled steel sections, plates or bar: material and dimension standards**

Form	Dimensions	Tolerances	Material quality: Non-alloy steels	Material quality: 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 6.4 **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.

Table 6.5 **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 publication is limited to reuse of reclaimed steel, for structures where fatigue is not a design consideration, the limiting thickness values proposed by SCI P419 [64] may be used. The background document to EN 1993-1-10 [65] 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 fracture mechanics approach, but reduces the calculated crack growth for applications where fatigue is not a design consideration. Table 6.6 follows the same format as Table 2.1 of EN 1993-1-10, but adopts a reduced crack growth. The values in Table 6.6 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.

The inspection documents (or test certificates) constitute sufficient evidence that the mill product satisfies a required grade and subgrade. At the steel manufacturing plant, the quality control system puts 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 assembling members [66]. The inspection document for each batch of steel is the most important document to 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 in order to provide the adequate knowledge of the materials needed for reliable evaluation (see Appendix A). Unidentified reclaimed steel may be used in non-safety critical structures, for example in agricultural buildings (example of a Class C reclaimed steel according to section 6.2.1).

Table 6.6 **Limiting thickness values when fatigue is not a design consideration [41] [64]**

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

6.3.2 Selection and acceptance criteria

The following procedure is proposed for 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 original building, should be provided for all members.
- All products to be reused should come from a structure first constructed 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 corrosion. (Elements need to be visually exposed, and therefore any fire protection should be removed). In the case of structural hollow sections, the weld seam has to be inspected for any defects.
- Coatings containing toxic substances, e.g. lead, cadmium, asbestos, and surface scaling needs to be removed by preparing the surfaces to EN ISO 8501-1 [67].
- 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 6.3 and Table 6.4. 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. Flange thickness should be measured at ¾ points along the member, each at top left half-flange and bottom right half-flange, and web thickness should be measured at ¾ points along beam central axis;
- 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. 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. Tolerances on older sections may be different and therefore some straightening may be required, e.g. see Table 6.7 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 Section 6.2.1.

Table 6.7 **Review of geometrical tolerances for individual members**

Products	**Tolerances**				
	BS4 UK (1962) [68]	Dorman Long UK (1964) [69]	NSSS UK (1994) [70]	EN 1090-2 EU (2018) [4]	STAS 767 RO (1988) [71]
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	

6.3.3 CE marking

Marketing of construction products in the EU is regulated by the CPR [56], and for the supply of structural steel products, came into force on 1st July 2013. It requires that structural construction products “placed on the market” (available for sale) after this date are CE marked to indicate appropriateness for use in construction in the EU where a European Harmonised Standard (hEN) or a European Technical Assessment (ETA) exists for the product. It places duties on importers, distributors and manufacturers to ensure that these CE marking and associated obligations are met. CE Marking can be applied on constituent products, individual fabricated steel components or the whole structural kit.

A “CE” mark indicates that a product is consistent with its DoP as made by the manufacturer. CE Marking can be either affixed to the product, issued with accompanying documentation or made available on demand through electronic means.

By making a DoP, the importer, distributor or manufacturer is assuming legal responsibility for the conformity of the product with its declared performance.

There are two forms of CE Marking, which are applicable to structural steelwork:

- Material and product standards – relating to the manufacture and properties of the product,
- Execution standards – relating to the design and manufacture of load bearing components and structures.

The constituent products should be evaluated by checking the geometry and the structural steel properties, see Section 6.2, and their durability. EN 1090-1 refers to the methods and instruments to be used to check that the geometry of the constituent products complies with all tolerances.

Fig. 6.3 shows an example of CE marking of constituent products, in this case, a cold-formed structural hollow section. In this example, it was issued by SSAB (Nordic and US-based steel company) and it contains:

- the CE mark and underneath a four-digit number, which is the number of the Notified Body that assessed SSAB for CE marking,
- the grade and quality of the steel material,
- the mechanical and chemical properties of the section,
- tolerances on dimensions and shape, and
- durability (indirect) evaluation by specifying surface protection requirements.

In the case of CE Marking to an execution standard, the manufacturer will ensure that its products meet the specified performance characteristics that are defined as essential to the application of the products in the field of construction. In order to do this, the manufacturer should:

- Identify the requirements in terms of defined essential performance characteristics and required values to be met. For structural steel components, these requirements are defined in clause 4 of EN 1090-1.
- Use specified test methods that can evaluate whether products conform to the specified requirements. For structural steel components, these evaluation methods are defined in clause 5 of EN 1090-1.
- Implement a system for controlling regular production. For structural steel components, the system for evaluation of conformity is defined in clause 6 of EN 1090-1.
- Mark its products in the correct way using a suitable classification and designation system. For structural steel components, the marking system is defined in clauses 7 and 8 of EN 1090-1.

01234

Any Co Ltd, PO Box 21, B-1050

03

01234-CPD-00234

EN 10025-1

Hot rolled structural steel products.

Intended uses: Building constructions or civil engineering.

Tolerances on dimensions and shape: Plate EN 10029 Class A

Elongation	
Tensile strength	
Yield strength	: **Steel S355J0 – EN 10025-2**
Impact strength	
Weldability	

Durability: No performance determined

Regulated substance: No performance determined

0416

SSAB

SSAB Europe Oy, Harvialantie 420,
13300 Hämeenlinna
07

ref. S235JRH_20150812

EN 10219-1:2006

Cold formed welded structural hollow section:

EN 10219-1:2006	**S235JRH**
Essential characteristics:	
Yield strength Rp0.2 min	**235**
Tensile strength Rm T<3 mm	**360-510**
Tensile strength Rm T>=3 mm	**360-510**
Elongation A% min*	**24**
Impact strength min J / Temp	**27 / -20 °C**
Weldability CEV max	**0.35**
Tolerances on dimensions and shape	**EN 10219-2, Clause 6**
Durability	**Suitable for hot dip galvanizing**

*For section sizes D/T < 15 (round) and (B+H)/2T < 12,5 (square and rectangular) the minimum elongation is reduced by 2

Fig. 6.3 Template and example of CE marking of constituent products

6.4 Structural components or entire primary structure

6.4.1 General

Steel structures and steel construction products are, in general, highly demountable. Provided that attention is given to deconstruction at the design stage, there is no technical reason why nearly all the steel building stock should not be regarded as components for future use in new applications. In some single-storey building sectors, e.g. industrial and agriculture, reuse of 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 organized combination of connected components designed to carry loads and provide adequate rigidity.

To facilitate greater reuse, it is important that designers not only use steel but also do what they can to optimise future reuse. Steps that can be taken to maximise the opportunity for reusing structural steel include [72]:

- Use bolted connections instead of welded joints to allow the structure to be easily disassembled;
- Use standard connection details including bolt sizes and spacing of holes;
- Ensure easy and permanent access to connections;
- Where feasible, try to ensure that the steel is free from coatings or coverings that will prevent visual assessment of the condition of the steel;
- Minimise the use of fixings to structural steel elements that require welding, drilling holes, or fixing, by using clamped fittings where possible;
- Identify the origin and properties of the component for example by bar-coding or e-tagging or stamping and keep an inventory of products;
- Use long-span beams as they are more likely to allow flexibility of use and to be reusable by cutting the beam to a new length.

Fig. 6.4 presents the overall framework for reuse process of a steel structure or structural components parts of it.

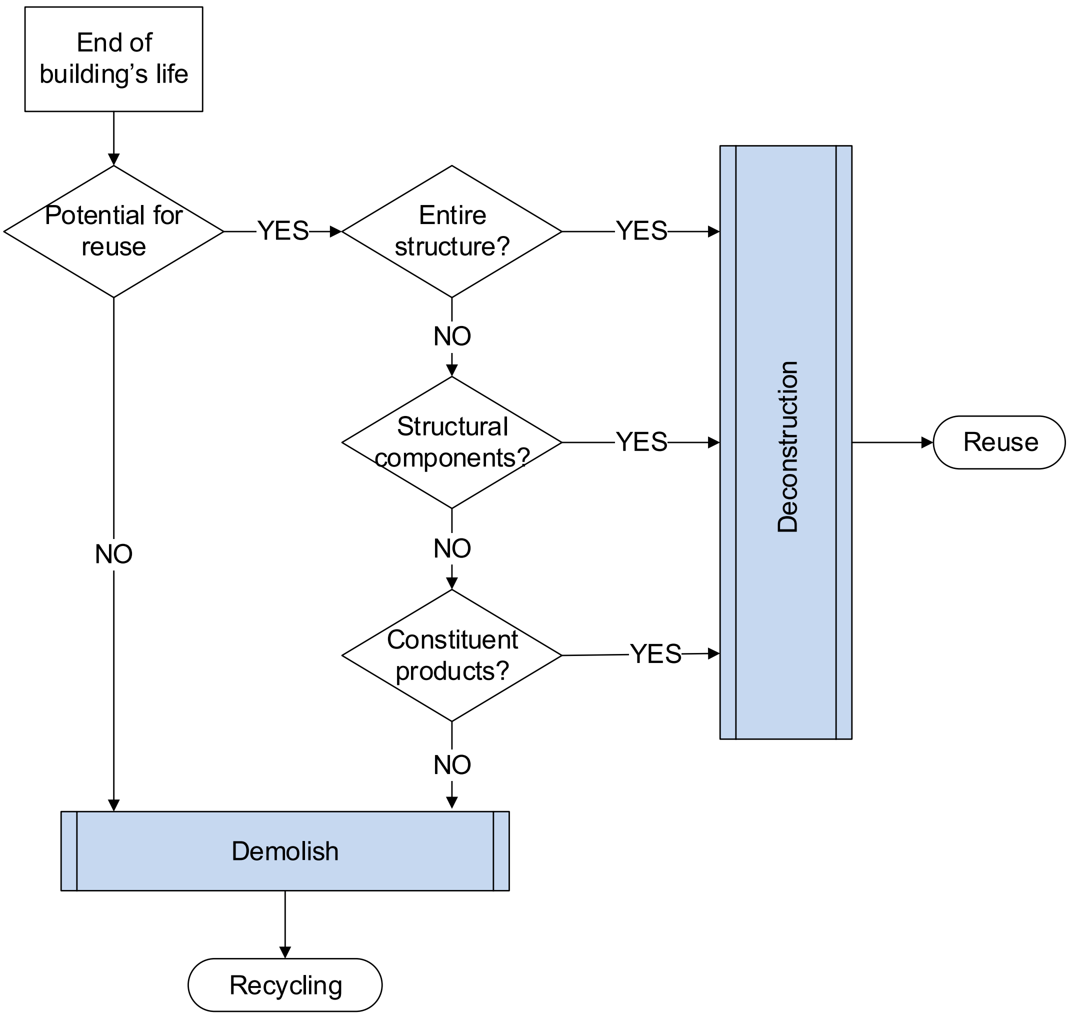


Fig. 6.4 Overall framework for reuse process of steel structure/components

6.4.2 Selection and acceptance criteria for reclaimed structural components

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 is proposed:

- The structural components or entire primary structure should be a part of a single-storey steel building. The building structure should have been erected after 1970 and should have not been exposed to extensive dynamic loading and other extreme actions.
- Each element shall be dismantled in the reverse order of construction. 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.
- Structural steel components shall 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 manner 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.
- First, the individual structural members are evaluated (see section 6.3). The term “evaluation”, in this context, is as defined in EN 1090-1:
- The term 'evaluation method' is used for all kinds of methods used to demonstrate compliance with the requirements, e.g. physical testing, measurements of geometry and structural calculations whether assisted or not by physical testing.
- The reclaimed steel components should not have areas of accelerated localised corrosion or show other evidence of localised section loss. If corrosion affects the characteristics of the components, they should be redesigned according to the new dimensions or, if are beyond economic repair/reconditioning, shall be scrapped.
- For all reused components, or entire primary structure, documentation showing the location and building structure where the components were recovered from, including date of construction of original building, should be provided.
- All reclaimed steel should be certified to the section properties and classified according to the system proposed in Section 6.2.1 of this document.
- Careful visual inspection of every reclaimed member, and assessment against the tolerances should ensure that the element has not undergone plastic deformations and therefore the residual strains, and reserves of ductility, are no different to that of ‘new’ steel. All dimensions (if the initial drawings are missing) of the components/structure shall be measured to check they meet all the tolerances, at the level of cross-section, member and/or structure. Cross-section dimensions should comply with EN 10034 [19] for wide flange H- and I- members, EN 10219-2 [28] specifies tolerances on shape dimensions of cold-formed structural hollow sections, while EN 10210-2 [26] specifies tolerances on shape dimensions of hot finished ones. Straightness of all members should comply with the tolerances given in EN 1090-2. The structural component or module or the primary structure should comply with the geometric tolerances in Annex B of EN 1090-2 (see B.2 manufacturing tolerances and B.3 for erection tolerances).

- All welds should be 100% visually inspected throughout their entire length for surface imperfections in accordance with EN ISO 17637 [73]. The visual inspection should be carried 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. See also section 7.6.5.
- Existing bolts from previous applications should not be reused;
- If the existing coating contains toxic substances (e.g. lead, cadmium and / or asbestos), remove existing coatings and surface scaling by preparing the surfaces to EN ISO 8501-1 [67]. Steel paint that contains lead carbonate and lead sulphate can be encapsulated with other paint.
- The reused steel component/detail/structural component or module/primary structure can be CE marked to EN 1090-1 [3].

Reclaimed steel structural components, detail, module, or primary structure can be CE marked according to EN 1090-1. Nevertheless, 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. 6.5.

The flowchart in Fig. 6.5 identifies three possible classes, after checking the eligibility and compliance with 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. Deep 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 the compliance with EN1090-2 and the harmonised standards. Steel materials meet performance requirements through limited testing (see Fig. 6.1) and with approved quality assurance from original certificates. The welding has to pass the visual and other NDT inspection. The reclaimed structural component can be reused in designed according to EN 1993-1-1 with some restrictions: (i) plastic global analysis is not allowed when 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 steel structural components.
- Class RSC3: the structural component has been CE marked in the first life according to EN 1090-1, but in this case an existing CE marking is not available. The structural components have to conform with EN 1090-2 standard. Steel materials and welding can be evaluated through limited material testing and recertified. The reclaimed structural component can be reused in designed according to EN 1993-1-1 with the same restrictions as for Class RSC2.

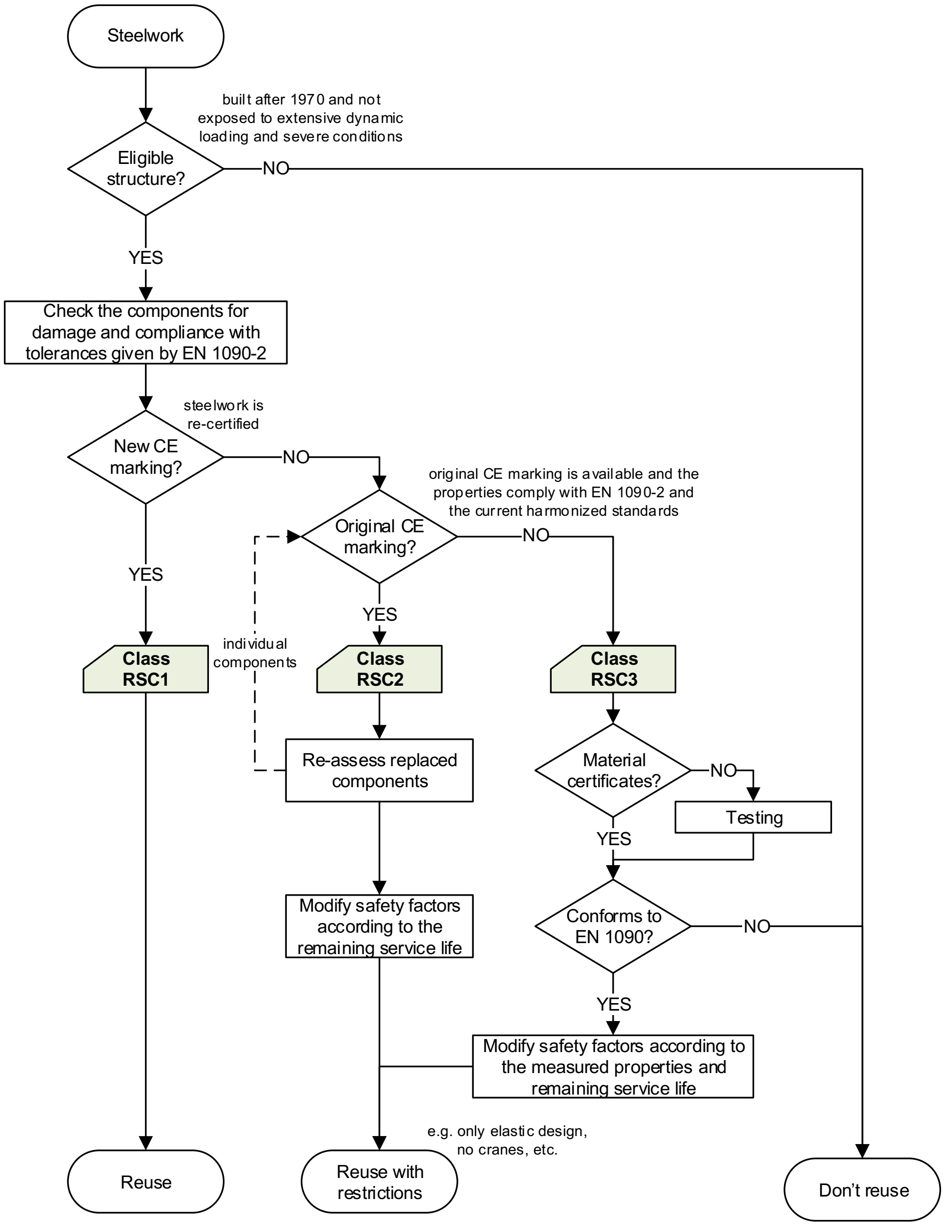


Fig. 6.5 Overall framework for verification of the reusability of components or entire primary structures

6.4.3 Execution classes. Potential issues related to CE marking

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 specified performance characteristics that are defined as essential to the application of the products in the field of construction.

For any project, the required quality of fabrication or execution class (EXC) must be specified. EN 1090-2 [4]: requires the execution class to be specified for the works as a whole, an individual component and 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 to 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. Execution class is defined as a 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.

According to EN 1090-2 there are four classes EXC1, which is the least onerous to EXC4, which is the most onerous. For the four execution classes EXC1 to EXC4 requirement strictness increases from EXC1 to EXC3 with EXC4 being based on EXC3 with further project specific requirements. It is down to the designer to select the EXC required for the structure, an individual component, or a particular detail of a component. EN 1090-2 states that EXC2 should apply if no execution class is specified.

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

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

In terms of reliability management, the selection of execution class should be based on either the required consequences Class (CC) or the reliability Class (RC) or both. The concepts of reliability Class and consequences Class are defined in EN 1990 [5].

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) should be based on Table 6.8.

Table 6.8 **Selection of execution class (EXC)**

Reliability Class (RC) or Consequence Class (CC)	**Type of loading**	
	Static, quasi-static or seismic DCL[a]	**Fatigue**[b] **or seismic DCM or DCH**[a]
RC3 or CC3	EXC3[c]	EXC3[c]
RC2 or CC2	EXC2	EXC3
Rc1 or CC1	EXC1	EXC2

[a] Seismic ductility classes are defined in EN 1998-1:
Low = DCL; Medium = DCM; High = DCH.
[b] See EN 1993-1-9.
[c] EXC4 may be specified for structures with extreme consequences of structural failure.

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:

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.

Specification of a higher Execution Class for the execution of a structure or component or detail should not be used to justify the use of lower partial factors for resistance in the design of that structure or component or detail.

PROGRESS focussed on the reuse of single-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 DCL type of loading. Consequently, Execution Class 2 (EXC2) is the most appropriate for the majority of single-storey industrial buildings.

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 are to 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.

Special care is needed if existing connections are to be re-used. In particular, any welding should be subject to careful inspection and test. Visual inspection of 100% of the welds is recommended.

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) 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 aspect 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, the 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) utilization 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 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 the entire primary structure, the contractor or the stockholder of the stock, is responsible for specifying the EXC for the structure (the works as a whole) and for components and details, where it is appropriate to specify an Execution Class different to that specified for the structure. Where different, the Execution Class for a component or detail should not be lower than that specified for the works as a whole. The EXC for a component or detail should be clearly identified in the new specification if it is different to the Execution 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 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 contractor or the stockholder has to draw up the product technical documentation required by the Regulation for the assessment of the product’s conformity to the relevant requirements. Together with the EC Declaration of Performance, the technical documentation must be made available when requested by the appropriate authorities.

The contractor or the stockholder of components falling within EXC2, 3 and 4 must have, or have access to, a Responsible Welding Co-ordinator (RWC) and all welding must be controlled by the RWC.

The Declaration of Performance (DoP) is a legal declaration made by the contractor or the stockholder that the product was manufactured in accordance with, and conforms to, the requirements of the standard EN 1090-1. The contractor, his authorised representative or the stockholder must produce a DoP before placing the product on the market.

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.

The contractor or the stockholder is responsible for creating a CE Mark and applying it to their product. It must be affixed visibly, legibly and indelibly on one or more of the following locations: the product, an attached label, the packaging, on the accompanying commercial documentation (such as delivery note) or in a technical specification.

The format of the CE Mark, as well as the information that needs to be included to ensure traceability, is detailed in the Annex ZA of EN 1090-1 and presented in Fig. 6.6.

CE 01234
AnyCo Ltd, PO Box 21, B-1050 **18** 01234-CPD-00234
EN 1090-1 Rafter of a steel portal frame – M 123 Tolerances on geometrical data: EN 1090-2. Weldability: Steel S235JR according to EN 10025-2. Fracture toughness: 27 J at 20°C. Reaction to fire: Material classified: Class A1. Release of cadmium: NPD. Emission of radioactivity: NPD. Durability: Surface preparation according to EN 1090-2, preparation grade P3. Surface painted according to EN ISO 12944-5, S.1.09. Structural characteristics: Design: NPD. Manufacturing: According to component specification CS-034/2006, and EN 1090-2, execution class EXC2.

Fig. 6.6 Proposal of CE marking information of products properties by material properties and geometrical data

6.5 Cold formed structural steelwork elements

6.5.1 Introduction

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 is dependant 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.

Each manufacturer produces its own specific shapes, the 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 members and sheets to be galvanized should also be suitable for galvanizing.

EN 1993-1-3 [11] specifies the materials in Table 5.1a conform to harmonized product standards, while the materials in Table 5.1b conform to EN or ISO product standards. For other steels the suitability for cold-forming shall be demonstrated by a bend test in accordance with EN ISO 7438 [74] or by an equivalent test.

For an effective design, when spans are around 6.0 m to 7.0 m, continuous purlins are usually made with sections lapped and bolted at intermediate supports. Alternatively, double-sections can be used for strengthening 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 success rate of reclaiming secondary light gauge steelwork is likely to be much lower in comparison with primary hot rolled steelwork. This is due to the fact that cladding is usually fixed with a considerable number of connectors, which may hinder the disassembly process as well as damaging the structural elements during the process.

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, accounting for the recommendation from EN 1090-4 [75]. Alternative specifications of source material similar to section 6.2.5 are also possible. 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 elements is found or required for future application. Therefore, the assessment of execution processes may be only related with geometric tolerances of the cold formed steelwork according to EN 1090-4 and EN1993-1-3 recommendations.

6.5.2 Classification of cold formed reclaimed steelwork for reuse

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

According to Section 6.2.1, reclaimed steel should be classified based on the verification against (i) material performance requirements (adequacy assessment) and (ii) quality assurance requirements (reliability assessment), framing the steel in on one of the following classes, i.e. class A, class B or class C. For Class C reclaimed steelwork, as a wide range of steel grades are likely to be available, it is not recommended to assumed a yield and tensile strengths of more than 120 MPa and 260 MPa respectively.

Cold formed members are usually protected against corrosion by means of metallic coatings as specified in EN 10346 (designation of coating mass Z, ZM, ZA or AZ) and, if necessary, by means of an additional organic coating as specified in EN 10169. The provisions of EN 10346 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.

6.5.3 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 single storey steel building and should have not been exposed to any extreme actions;
2. Each element should be dismantled in the reverse order of construction. 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 1090-1:2009;
5. All reclaimed steel should be certified to the section properties and classified according to the system proposed in 6.2.1 of this document;

6. If the initial drawings are missing, all dimensions of the components/structure shall be measured to check 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. 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 [76] 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 EN10143 [77] and apply over the whole length.
7. Bolts from the previous application can be reused;
8. The product surface shall be visually inspected for verification of conformance with the requirements in 7.4 to 7.6 of EN10346 [30]. The coating surface can vary and change to a dark appearance by oxidation. The available coating masses should conform with Table 11 of EN10346. NDT tests have to be performed to check the coating thickness. If necessarily the methods described in Annex A (Z, ZF, ZA and AZ) or Annex B (AS) of EN10346 shall be used;
9. The reused steel component or structural component can be CE marked according to EN 1090-1:2009.

6.5.4 Material performance requirements

The re-certification of non-constituent light gauge cold-formed elements 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 product recertification:

- Yield strength or 0,2 %-proof strength (ReH/Rp0,2);
- Tensile strength (Rm);
- Elongation after fracture A80 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, coating mass / coating thickness should be evaluated.

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 re-fabricated, reclaimed structural steelwork can be CE Marked in accordance with EN 1090.

In addition to careful control of the fabrication process, material properties must be declared according to EN 1090-4 clause 5.1. When using reclaimed steel, as referred for hot rolled profiles, this may be a stockholder's responsibility.

The purpose of declaring material properties is so that the material used in construction meets the appropriate standard and that properties required by design are confirmed, *e.g.* the required material strength assumed in the member verifications has actually been provided.

The test regime for cold-formed steelwork is intended to allow the necessary material properties according to EN 1090-4 clause 5.1 to be declared, based on dimensional survey, by non-destructive tests, by destructive tests or by making conservative assumptions.

For the scope the current design guide, the nominal yield strength for cold formed elements shall be in the range of 220 N/mm^2 and 450 N/mm^2. The minimum nominal tensile strength should be in the range of 300 N/mm^2 to 510 N/mm^2. The ductility requirements for design to EN 1993 are presented in Table 6.1 (recommended values that may be modified by the NAs).

6.5.5 Adequacy and reliability assessment

As for hot rolled elements, adequacy and reliability assessments are required for cold formed steelwork to ensure that the reclaimed product can to 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.

6.5.6 Product properties do be declared for cold formed reclaimed elements

This section summarizes the steel properties that need to be assessed for reclaimed cold formed steel elements according to EN1090-4 clause 5.1 (Table 6.2). Further commentary on these properties is also provided.

Yield strength, tensile strength and elongation

According to EN 10346 [30], 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 (see section 6.2.7)

Geometric tolerances and limitations

The geometric tolerances on dimensional shape shall comply with EN 10143 [77]. EN 1993-1-3 specifies minimum thicknesses for cold formed 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 metallic coating nearby 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 detail.

Metallic coating composition, designation and layer mass

The composition of the metallic coating needs to be specified according to EN 10346. Section 3 from 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 Appendix A.

Chemical composition

For cold forming products, EN 10346 may be used, where in Table 2 of the same standard the chemical composition for steels for construction is presented. The intent of this declaration is to enable the carbon equivalent value (CEV) to be calculated, which is a key measure of weldability. If the reclaimed cold-formed steelwork is not to be welded, chemical composition doesn’t need to be assessed. See Appendix A for more detail.

Table 6.9 **Material properties to be declared according to EN 1090-4 clause 5.1**

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 A80 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.

6.5.7 Durability

The process of disassembling 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 the subsequent life cycles is reduced, thus reducing the durability of the steelwork system. It is therefore necessary that the test protocol will assess the remaining/available coating mass for the reclaimed cold-formed steelwork.

6.6 Cladding

Composite panels are covered by EN 14509 – *Self-supporting double skin metal faced insulating panels. Factory made products* [78]. Specifications. For panels constructed after 2004, the key information is available on a tape at 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* [79].

In this study metal faced insulated sandwich panels are considered to be the main form of cladding. Recommendations for the evaluation of the potential for reusing sandwich panels are proposed. For the evaluation of safety aspects for reuse, the rules in EN 1990 (safety factors) and rules in harmonized product standard EN 14509 for type testing essential properties are used. A basic requirement for a limited amount of testing is that the name of manufacturer is known and a copy of original declared values (values given by the manufacturer) is also known. This might limit the use of reduced testing program for panels older than 25 years, because of the lack of common 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 rules in EN 14509 is recommended.

The evaluation of potential to reuse sandwich panels are:

- 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.

6.6.1 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 mechanical properties are the values declared by the manufacturer at the time of delivery of the panels. This reference level is further called as 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 and 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 is that the ageing affects mostly the cross panel tensile strength and panel shear strength. Results from testing dismounted panels at the end of 1990s indicate that the ageing rate of wrinkling strength is approximatively the 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 EN 14509 section A1. 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.

6.6.2 Tensile strength and density

The characteristic value of tensile strength is compared to originally declared value. If there is degradation in the level of less than 10%, the panels can be reused using the 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.

6.6.3 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.

6.6.4 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.

6.6.5 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.

6.6.6 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 EN 14509 section A.16.

6.6.7 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, to 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 %.

6.6.8 Tolerances

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

6.6.9 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).

6.6.10 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.

6.6.11 Certification for reuse

A summary of the evaluation and certification procedures for reuse of sandwich panels is presented in Table 6.10.

Table 6.10 **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*, A*1*): 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 x Declared value, and: 2. Shear strength Actual value ≥ 0.9 x 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 x 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 x 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 where fulfilling the requirements in the project for reuse.

7 PRACTICAL IMPLEMENTATION OF STEEL REUSE

7.1 General

Structural steel sections are robust, durable and dimensionally stable elements that are generally bolted together to form structural assemblies. Of the range of steel products used in construction, they are considered to be among the most suitable steel products for reuse as opposed to the current common practice of recycling by remelting. Reusing structural steel yields significant environmental savings compared to recycling and has the potential to be cheaper than using new steel.

We know that reusing reclaimed structural steel is technically feasible but it remains a niche or small-scale activity. Reuse case studies and consultations with the supply chain, confirm the technical viability but also confirm the many, real and perceived, technical and non-technical barriers to reuse; particularly to more mainstream reuse. These include barriers across the supply chain, in particular the additional cost and longer procurement and construction programmes, and other barriers specific to individual actors.

Reusing reclaimed steel is not a new idea; in fact, anecdotal evidence suggests that the practice was more prevalent in the past but has declined over the last few decades. There are several reasons for this the most significant including new development programme constraints and pressures and tougher health and safety requirements in relation to demolition activities, in particular, working at height.

It is clear that the circular economy agenda is still in its early stages of development and it will take time for policy to develop. A key point is that the EU mandatory target for recovering 70% of construction and demolition waste by 2020 has already been achieved in most EU member states. This EU target focusses on high volume and problematic waste streams, particularly concrete, timber and masonry, and is more a measure of 'landfill avoidance' rather than a policy to encourage higher value, closed-loop recycling or reuse.

The additional time/programme and cost of using reclaimed steel are significant barriers identified across the supply chain. Extra time, in general, incurs additional cost and there are many barriers centred around the increased programme associated with reclaiming and reusing structural steel including:

- The current approach of 'just-in-time' supply (of new steel) by stockholders and steelwork contractors;
- The additional design, procurement and testing/certification time required compared to using new steel;
- Increased automation in steelwork fabrication which is far less efficient when using reclaimed steel sections;
- Insufficient time within new development programmes to allow for deconstruction and recovery of the steel elements rather than demolition.

A statistical analysis of a survey of the barriers to structural steel reuse in the UK [80] identified the following barriers ranked in descending order of importance:

- Availability of reclaimed sections particularly of the desired size, volume and in the right location;
- Issues relating to the quality, traceability and certification of reclaimed sections;
- Additional cost associated with using reclaimed sections;
- (Lack of) supply chain integration; particularly communication and sharing information through the supply chain and trust (and risk sharing) between actors;
- Additional time required within construction programmes to allow for using reclaimed steel; in general, additional time incurs addition cost;
- Reclaiming and reusing structural steel is a relatively uncommon practice and many organisations simply do not have the skills or experience to do it;
- The perception that reclaimed steel is somehow inferior to new steel sections.

Consistent with other parts of this guide, practicality of reuse is reviewed below in the context of:

- Reuse of existing buildings;
- Design for future deconstruction and reuse;
- Whole structure reuse;
- Component level reuse.

Reuse of existing buildings – whole structure reuse

Section 3.2 includes several cases of successful whole structure reuse. All reuse projects are project specific, as are new construction projects, and consequently detailed, practical issues need to be addressed at the project level. Resolving these issues is generally straightforward through the knowledge and skills of the designer or steelwork contractor, by following the guidance and procedures outlined in the guide and/or consulting specialists where required. It should be recognised that resolving project-specific issues will take time and money however, key to this is good planning so that, as far as possible, any issues are foreseen and costed as part of the economic assessment of the chosen reuse scenario.

Key high-level factors for success include:

- Co-ordination and good communication throughout the supply chain;
- An enlightened client who understands the benefits (economic and environmental) and the challenges of reuse;
- A good understanding of risk allocation along the supply chain;
- Early communication so that decision-making is not dictated by others;
- A main contractor and a steelwork contractor with a ‘can-do’ attitude.

In terms of process Fig. 4.1 outlines the recommended stages in the process and provides references to more detailed sections of the guidance.

Whole buildings for reuse are generally procured via companies that specialise in reused agricultural or industrial buildings or via personal contacts, for example, within the agricultural sectors. In addition, some property companies offer previously used buildings for use, often via auction.

Reuse of existing buildings – component level reuse

There is an existing, small market for the reuse of structural steel components. This includes:

- Trading via construction material exchange platforms;
- Some steel stockists hold reclaimed sections alongside new sections;
- Via eBay and other mainstream, online trading platforms.

This market is mainly unregulated and generally involves small quantities of relatively small section sizes suitable for small domestic-scale projects.

The practicalities of reusing steelwork at the component level mainly revolve around:

- Procuring sufficient sections of the right size, at the right time and within an acceptable travel distance;
- Establishing the mechanical and chemical properties of the steel.

Design for future deconstruction and reuse

Practical issues in relation to design for future deconstruction and reuse are relatively straightforward. Recommendations of enhancements to current SSB design practice, are given in Part 2 of this guide and apply to both whole structure and component level reuse.

Key points include:

- Standardisation of structural elements and connections;
- Design loading assumptions to facilitate relocated reuse of the structure;
- Connection detailing to facilitate reuse including haunch and apex connections and column to foundation connections;
- Attachment of secondary steelwork and cladding to the primary structure.

7.2 Economic considerations

Economic assessment of the viability of reusing steelwork is challenging and is project, location and time specific. In the absence of regulation or other drivers, reuse is reliant on the usual commercial drivers at play within the construction industry, i.e. lowest capital cost. However, it can be deduced, from the cases included in Section 3.2, that reusing the building or the building structure, was cheaper than building the same structure using new steel for these projects. Section 3.5 describes a theoretical assessment of the life cycle costing (LCC) of reusing steelwork.

For the case of whole building or whole structure reuse, it is recommended that a detailed, project specific comparison is made between reusing the building and the new build cost. This should include:

- additional deconstruction costs over and above the cost of demolition;
- testing and certification of the steelwork where required;
- storage and transport of the sections prior to their reuse;
- re-fabrication costs that will be project specific.

A further important factor in assessing the economic viability of reusing structural steel is difference in cost between new steel section and scrap steel sections. New steel and scrap prices are volatile and also vary geographically as shown in Fig. 7.1 which shows the variation in price of new steel sections (Long products - medium sections - Europe) and grade OA steel scrap (UK OA plate and girder scrap) between 2000 and 2016. Prices have been adjusted for inflation on a constant UK (GBP) 2016 basis. The difference between the scrap and new steel price has varied between £187 and £658 per tonne (average £312 per tonne with a standard deviation of £90 per tonne) over the period 2000 to 2016.

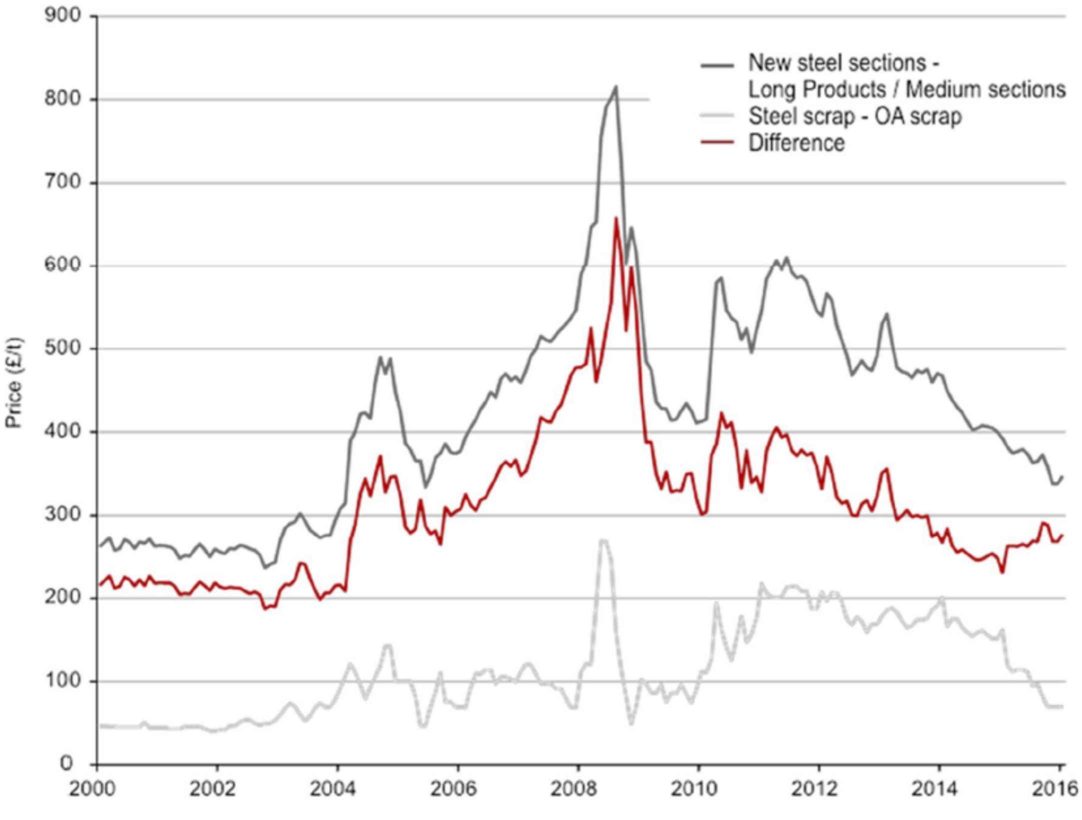


Fig. 7.1 Variation in price of new steel sections and grade OA scrap steel in the UK (2000-2016) [81]

Fig. 7.1 shows the volatility of new steel and scrap prices. Although there is clearly a correlation between new steel and scrap prices, the ratio (scrap: new steel price) varied between 18% and 43% over the period. The average ratio is 26%. The general trend between 2000 and 2016 is for the price of scrap, as a proportion of the new steel cost, to increase.

We know that reclaiming and reusing structural steelwork incurs additions costs (see above), however, in simple terms, if these additional costs are less than the price difference between new steel and scrap then, theoretically at least, reusing steelwork should be economically viable. In reality, there are many additional barriers and less tangible costs that this simple assessment does not take into account. In particular, these include time and production efficiency penalties from sourcing and processing reclaimed steelwork.

Table 7.1 shows an overview of cost information gathered by SCI and the University of Cambridge in 2016-18 [82]. The data were obtained from a series of interviews from organisations across the UK steelwork supply chain. Most cost elements costs are the same for new and reused steel but additional costs for deconstruction, reconditioning, transport and testing are included for reused steel.

Table 7.1 **Overview of cost ranges of various operations for the fabrication and erection of new and reused structural steel elements [82]**

Element	**New steel (£/t)**		**Reused steel (£/t)**	
	Min	**Max**	**Min**	**Max**
Raw materials	600	750[1]	266[2]	305[2]
Deconstruction	-	-	120	165
Reconditioning	-	-	100	200
Fabrication	325	455	325	455
Construction	120	167	120	167
Fire protection	180	270	180	270
Engineering	56	79	56	79
Transport	22	25	66	75
Testing	-	-	145	175
Total cost	**1303**	**1746**	**1378**	**1891**

[1] 25% has been added to the minimum cost to account for less common section sizes, etc.
[2] Reclaimed section costs are based on a scrap value of 26% of the new steel cost plus a flat rate margin for £110 per tonne.

The testing cost is based on the assumption that each structural element is tested. The cost (per tonne) was derived by assuming a test cost of £50 to £60 and that, on average, structural steel elements weigh 341kg. In practice, for typical portal framed buildings, the members will be significantly longer and heavier (5 to 6 times heavier than 341kg) and, as proposed in Appendix A, statistical sampling and ND testing, should mean that only a subset of elements require testing. In this case, the testing costs could be significantly reduced on a ‘per tonne’ basis.

As shown in Table 7.1 the cost of reusing reclaimed structural steelwork is estimated to be around 6 to 8% more than the cost of new steel. shows the cost difference between new and reused steel; negative values (on the vertical axis) indicate that the cost of new steel is less than reclaimed steel, positive values indicate that the cost of new steel is more than reclaimed steel. The four series represent that cost of scrap steel as a proportion of the new steel cost.

Other than for the steel (raw material) costs, the cost data in Fig. 7.2 is based on the average of the ranges shown in Table 7.1.

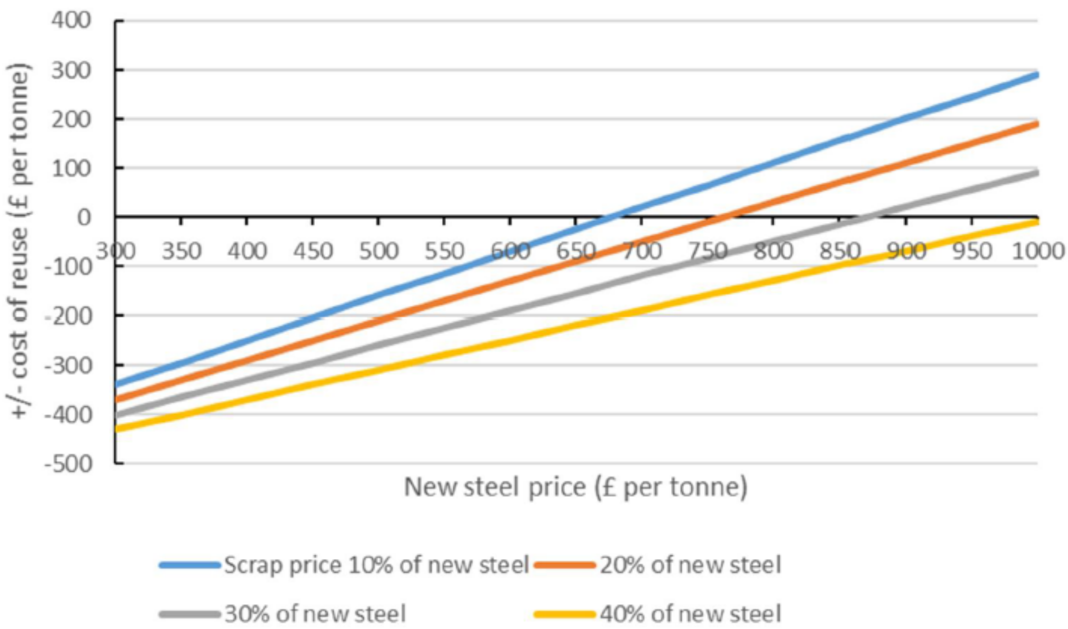


Fig. 7.2 Cost differential between new and reused steelwork

Fig. 7.2 shows that reuse theoretically becomes more economically viable as the cost of new steel rises and as the scrap price reduces, relative to the cost of new steel. With reference to Fig 1 therefore, 2008 appears to be the most favourable time within the assessed timeframe, for steel reuse.

Based on the assessment presented here and the assumptions made, it appears that there is a small cost penalty to reusing reclaimed structural steel although given some of the uncertainty and variability in the cost data, the difference is potentially not significant. As case studies of successful reuse projects have shown, when the additional costs in Table 7.1 are not incurred or are less that assumed on a particular project, the balance tips in favour of reuse.

7.3 Circular economy business models

The macro economic benefits of the circular economy have been quantified/estimated as part of several high-level circular economy studies. We know that steel structures are inherently reusable, however we also know that there are currently many barriers that prevent or hinder mainstream reuse, particularly within existing supply chain configurations. These barriers, some real some perceived, include increased cost and production inefficiencies which appear at odds with the macro benefits.

There are several observable trends influencing policy and opinion with respect to resource use, waste and the need to develop more circular economy business models. These trends include:

- Growing awareness that current and predicted global resource consumption demand is unsustainable;
- Public awareness on specific waste, resource and pollution issues, for example, plastic waste;
- The need to decouple economic growth and resource consumption;
- The need to reduce greenhouse gas emissions;
- (Extended) Producer responsibility for waste and for their products at their end-of-life.

While there are initiatives at the EU level to encourage and support the circular economy, to reduce C&D waste and to improve resource efficiency, there are no specific legislative or regulatory drivers or mandates requiring or encouraging the reuse of structural steel. Without legislation therefore, it falls upon industry to develop and devise new ways of working and new business models to facilitate greater steel reuse. New circular economy business models are emerging in certain sectors, notably the digital and retail sectors, but their applicability to the construction sector is less obvious.

It is important to remember that constructional steel already has excellent end-of-life recycling credentials and, as such, is not seen as a priority for improvement. Nevertheless, structural steel is a durable, robust and inherently reusable product group and therefore provides the opportunity to move from the closed recycling loop to a reuse loop. If successful CE business models cannot be identified for structural steel then it is unlikely that CE business models can be defined for other major construction product groups.

The circular economy is most commonly described as a system that is ‘regenerative by design’. It aims to minimize the input of new materials in the production system, as well as the amount of waste that is created throughout the entire process. To deliver a more circular economy, particularly in the absence of legislation, there is consensus that we need new circular business models.

In terms of business models for steel reuse, it is important to distinguish between:

- Reuse of constructional steel today;
- Design to facilitate future deconstruction and reuse of steel buildings and components.

This distinction is important since the opportunities and potential business models differ quite fundamentally. For example, the properties and provenance of steel sections recovered today is uncertain requiring mechanical and physical testing to prove their material properties prior to reuse. Whereas, future design for deconstruction and reuse can be facilitated by means of capturing and securely storing relevant product information electronically.

With reference to generic circular economy business models, as applied to single-storey, steel-framed buildings, Table 7.2 summarises the existing level of take-up and identifies possible future opportunities for new business models.

It is important to recognise that within construction, the product (of greatest interest to the customer) is the building not its constituent parts such as a brick or a steel beam. Greater focus should therefore be placed on CE business models for buildings rather than at the construction product level. Although leasing and other ownership models are common in the property sector, further work is required to capture circular economy benefits in new business models for property.

The reuse of structural steel elements or whole steel building structures, is really just a variation on, or adaptation to, the existing, current business model for the supply of new structural steelwork. While steel reuse has been studied several times and the barriers to its widespread uptake have been well documented, the model itself has shown to be technically feasible and, as many examples demonstrate, also commercially viable, particularly at the whole building or whole structure level.

The business models for the reuse of entire building structures and for individual structural components are quite different. The existence of both models in the market demonstrates the viability of both models albeit generally at small-scale (in the case of components) and in certain niche markets (in the case of whole building structures), for example, temporary buildings. The challenge is more about increasing demand for steel reuse, supply will respond to the demand and economies of scale will improve the viability of new business models.

In terms of an improved business model to facilitate widespread steel reuse some reconfiguration or consolidation of existing supply chains, for SSBs, makes good sense. With reference to Fig. 7.3, which represents the current structural steel supply chain (red loop 1), and particularly the green loops shown, the following reconfiguring of the supply chain would appear to offer a more viable business model:

- Recovery of the structural elements for reuse eliminates the need for the scrap merchant (loop 3);
- The demolition contractor or the steelwork contractor is able to store reclaimed stock so that the requirement for the stockist is avoided (loop 2);
- The demolition contractor has both space to store the reclaimed stock and is able to fabricate and paint the reclaimed steelwork (loop 4).

By consolidating supply chain partners in this way, profit margin (and risk) is shared between fewer organisations making the business model potentially more commercially viable.

Table 7.2 **Summary of the applicability of different generic circular economy business models**

Business model	**Existing situation**	**Future opportunities/prospects**
Hire & Leasing	Existing market for relocatable buildings in many building sectors (temporary applications) Many temporary works component examples	More challenging for longer-life or semi-permanent buildings Possible model for retail/distribution sectors Not viable for permanent works components
Servitization	Some existing building services models	Possible models for whole buildings but not building sub-systems, e.g. structure. Possible model for envelope systems.
Incentivised return	Increasing uptake in retail sectors but not common in construction. Construction schemes generally limited to uncontaminated construction waste. End-of-life waste generally excluded. No specific requirement for steel construction products (only packaging, electrical and electronic goods) which are already highly recycled	Buy-back scheme to guarantee future supply for steel makers – commodity futures trading Incentivises traceability and product development to facilitate future reuse
Reuse	Small-scale, niche markets at both building and product level Proven technical and economic viability Constructional steel's high recycling rate means that reuse is not currently an objective for legislators	Need to increase demand (supply will follow) - Legislation would help this Reconfiguration of current supply chain to facilitate reuse Capture new building information to facilitate future reuse
Supporting services	Limited available information and support Limited trading via material exchanges which don't provide the required steel properties to facilitate reclamation and reuse	Designers need skills and support including advice on warranties and risk Testing and certification of existing reclaimed steelwork for reuse Secure capture and storage of BIM data to facilitate future reuse

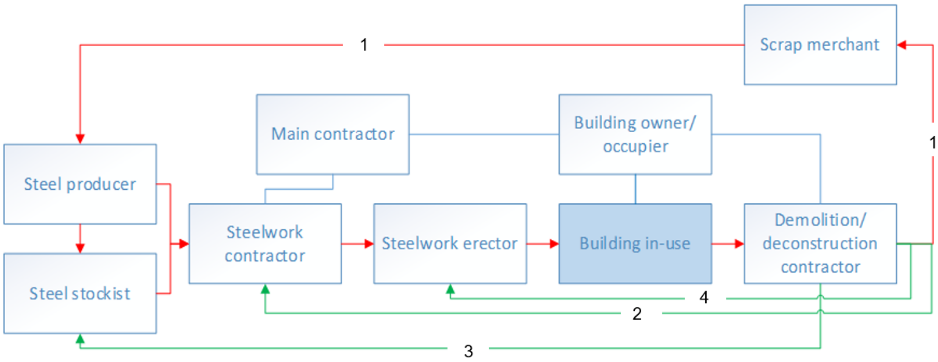


Fig. 7.3 Configuration of stakeholders in the supply of new structural steelwork

Loop 3 above is likely to provide the most viable business model for reusing structural steel at the component level. By way of example, Fig. 7.4 shows the site of Cleveland Steel and Tube (CST) in the UK. CST sells surplus offshore pipe into the construction sector. So, although the model is not strictly the reclamation of previously used sections, the business model has many features of a potentially viable business model for reclaimed, open sections. Key to the success of their business includes:

- A large inventory of stock available to order on-line
- Significantly reduced lead times (compared to new products)
- On-site shot-blasting, fabrication and painting

CST's success is, in part, based on consolidation of the existing supply chain.

Fig. 7.4 Cleveland Steel and Tube UK – stockyard of reclaimed offshore tubes

Crucial to the viability of any business model for reusing structural steel sections is to increase demand and to ensure sufficient supply to meet this demand. One model for doing this is a hybrid, buy-back model in which the demolition contractor is either mandated or incentivised to reclaim (intact) structural sections meeting certain specified criteria and returning these to a stockist of reclaimed sections. If not mandated, this could be incentivised through payment by the stockholder; with the price linked to the current scrap value, i.e. the price paid being current scrap price +10 to 20%. The actual price could be linked to the condition of the reclaimed sections, their age, provenance, transport distance, etc. Buy-back criteria could be mandated or set by individual stockholders. This could include, for example:

- Hot-rolled sections greater then 6m length
- In good condition without signs of corrosion
- Relatively free of attachments (details to be defined)
- Uncontaminated by any potentially hazardous coatings.

Such a model requires a suitably located stockholder site (relative to the demand) and of sufficient size to hold significant stock for a potentially long timeframe during which the stockholder can either sell the reclaimed steel for reuse or, as the scrap price varies, release unsold sections for recycling. The analysis presented in Section 7.2 is relevant to this model.

As an alternative to a physical inventory of stock, a virtual or cloud-based model may be an alternative, viable business model.

7.4 Adaptation to new locations

There are three possible levels for reusing an existing structure, i.e.: (i) the entire primary and/or secondary structure; (ii) the components of the primary structure, e.g. trusses or 2D portal frame, and components of the secondary structure or envelope; (iii) constituent products (such as IPE, UB, HEA, HEB, UC, L, U, RHS or CHS profiles, as Z-, C- Σ- and Ω- cold-formed profiles).

At the building systems level, kit constructions offer the greatest opportunities for reuse. From this point of view industrial single-storey steel buildings can be regarded as the best solutions. Portal frames come in a variety of different shapes and sizes, with flat and pitched roofs. A large variety of framing systems exists on the market, offering standard solutions. Portal frames can be deconstructed from the building and refurbished and reused on the same or an alternative building.

The guide focusses on the following structural solutions for the main structure, considered as the most commonly used in practice, i.e.:

I. single-storey steel framed buildings made of hot-rolled steel profiles;
II. single-storey steel framed buildings with members made of welded steel plates and variable cross-section;
III. single-storey steel framed buildings with hot-rolled steel profile columns and steel truss girders.

From the point of view of the secondary structure, the document will focus on systems built using thin-walled cold-formed steel profiles used both for purlins and side rails. These are also considered as the most commonly used in practice.

Purlins and side rails can be reused for the same or smaller span and/or spacing or combinations, e.g. smaller span – greater spacing. If they are treated as continuous with overlapping, either the overlapping should be considered higher either should be cut to the new dimensions. They can be framed in reuse cases (ii) or (iii) from above.

The cladding systems most commonly used and thus of interest are:

- cladding systems using sandwich panels with various insulations layers (PUR foam, PIR foam, mineral wool layer);
- cladding systems using built-up systems (internal + external layer of trapezoidal sheet containing in between the secondary structure and insulation layers);
- systems made with deep trapezoidal steel sheeting for roofing and liner trays for wall cladding.

Other parameters that should be considered are the building location and configuration. There are three locational scenarios:

- Reuse in-situ, i.e. the components are retained and not deconstructed (scenario of an existing structure);
- Reuse on the same site, i.e. the components are deconstructed and re-erected either in the same configuration and/or same or different location;
- Reuse on a different site.

The following aspects should be considered:

- the chance of being able to remove a component without damage depends particularly on the installation method and building typology;
- if the dismantling process is difficult and/or time-consuming, reuse of the component may not be economically viable;
- if possible, the cleaning and reconditioning could take place in a workshop, as it may become too expensive or simply not feasible to undertake such operations on site;
- the structural components identified as reusable must meet the corresponding demands presented in Section 6 of this guide;
- the envelope is essential in ensuring the comfort and energy saving of the system taken as a whole and has to provide as much as possible a uniform “enclosure” of the steel structure in order to avoid and/or control thermal bridging.

Fig. 7.5 presents possible ways for integration of reclaimed steel, from entire structure to structural component or to constituent products, integrating the steps from the Section 6 of the guide.

In-situ reuse consists in retaining the existing structure or structural components without disassembling. An alternative could be the deconstruction of the structure and after, the structural components can be then re-assembled on the same site in the original layout or different layout. In both cases, if necessarily, strengthening of elements and connections, adding or removing components can be done.

In the case of relocated reuse of the whole structure, in the same or different layout, the deconstructed structure can be either transferred directly to a new owner or location, sold to a stockholder or back to the producer. Alternatively, the structure can be leased by the producer/dealer and collected after the deconstruction. If necessarily, strengthening of elements and connections, adding or removing components can be done. In order to reduce the costs, a better alternative could be to reduce the bays of the structure and to modify the bracing system and longitudinal elements.

The second level is relocated reuse of structural components. Similarly, as in the previous case, components of primary or secondary structure and envelope can be reused directly, via material dealer, or a manufacturer. The success rate of reclaiming the cladding system, as it is, is very low in comparison with primary or secondary steelwork. This is due to the fact that cladding is usually fixed with a considerable number of connectors. New hybrid solutions for claddings, by reusing the existing components, can be built, that focus on their contribution to the improvement of the overall performance of buildings made of reclaimed elements.

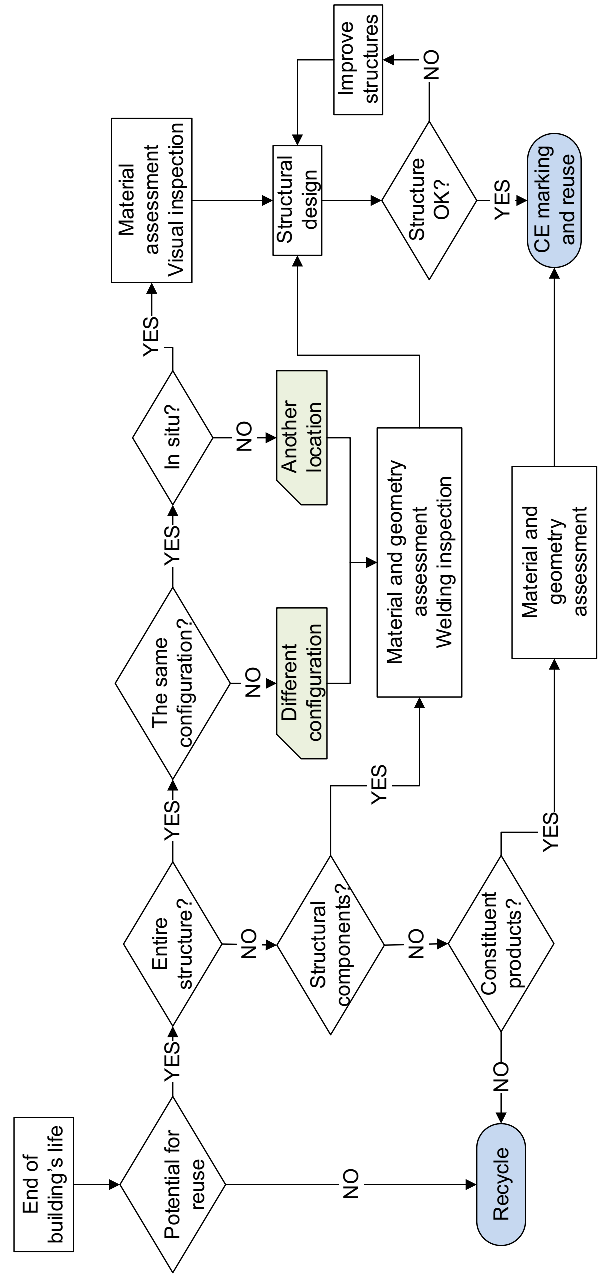


Fig. 7.5 Integration of reclaimed steel in different reuse scenarios

Re-fabrication of individual components by using constituent products represent the third level. Usually, the individual components cannot be used in the same design configuration they need to be remanufactured (usually involving separation of different materials, welded parts and so on). The modification of the constituent product can be made by the product manufacturer, material dealer, or demolition contractor and then sold to the manufacturer or dealer.

7.5 Pre-deconstruction audit, assessment for reuse and deconstruction

7.5.1 Introduction

It is necessary to understand the type and number of elements and materials that will be demounted to issue recommendations on their further handling. This will contribute to better deconstruction waste management, safer deconstruction techniques and increased chances of reclaiming structural steel from existing buildings. From a reuse point of view, it is important to evaluate the reusability of the steelwork based on a building inspection and documentation research. Based on these procedures, the adequacy/feasibility of the adopted reuse scenario can be evaluated. Undertaking these measures, clear recommendations about the end-of-life scenario of the building can be made, which may avoid any unnecessary costs.

7.5.2 Pre-deconstruction audits

The pre-deconstruction audit is an essential tool for clients and design teams with an emphasis on reuse of structures and materials. This audit summarizes the knowledge about building systems, components and materials, in particular what can be reused, what is waste, and recommendations for reconditioning of reclaimed products and waste management. One of the main outcomes of the audit is the inventory of elements and materials, their location, reusability, etc. Recommendations on the building decontamination and safe removal of hazardous waste, e.g. asbestos, should also be included.

According to the EU guideline [83], the waste/material audit consists of the following steps: research, field survey, condition evaluation and recommendations (see section 7.5.3), and its results shall be recorded in a report. The document provides guidance on best practices for the assessment of construction and demolition waste streams prior to demolition or renovation of buildings and infrastructures, called “waste audit”. The aim of the EU guidance is to facilitate and maximise recovery of materials and components from demolition or renovation of buildings and infrastructures for beneficial reuse and recycling, without compromising the safety measures and practices outlined in the European Demolition Protocol [84]. This protocol states that:

- Any demolition, renovation or construction project needs to be well planned and managed in order to reduce environmental and health impacts while providing important cost benefits.
- Waste audits (or pre-demolition audit as defined in the European Demolition Protocol) are to be carried out before any renovation or demolition project, for any materials to be re-used or recycled, as well as for hazardous waste.
- Public authorities should decide upon the threshold for pre-demolition audits (which is currently highly variable in the EU).

- Waste audits take full account of local markets for C&D waste and re-used and recycled materials.
- A good waste audit must be carried out by a qualified expert (the auditor).

The scope of the EU guideline and protocol [83] [84] includes waste from construction, renovation and demolition works. It excludes the design phase, as well as excavation and dredging soils. They include good practice from across the EU.

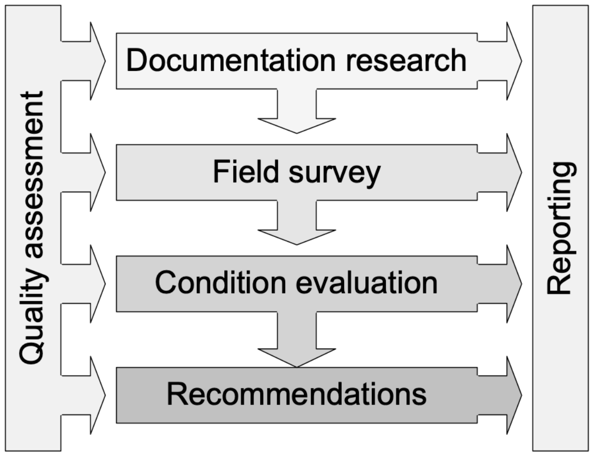


Fig. 7.6 Pre-deconstruction audit process

Step 1: Documentation research

Information about the structure and materials that may be reused, refurbished and recycled should be collected and analysed. The documentation research will inform the field survey in terms of the level of inspection required (see data collection and documentation section below).

Step 2: Field survey

Field survey should be carried out to gather further information or to confirm the compliance of the documentation research with the as built structure, concerning the types and amounts of materials and the condition of the steelwork and cladding.

Step 3: Condition evaluation

The information gathered on the condition of the materials and structure during the field survey should be recorded. The total volume of elements that can be salvaged and reused can then be assessed, as well as point out the best waste management option for other elements/products. Recycling should always on the top of the waste hierarchy.

Step 4: Recommendations

Based on the information gathered on the potential for reuse and recycling, targets can be set for different waste management methods, e.g. overall amount of materials reused on or off site, closed/open loop recycled off site. A report is finally produced.

7.5.3 Assessment for reuse

General

An existing building is targeted as a possible source of reclaimed elements or as an existing structure with a possible new future application. Classifying a structure as such may imply that, with visual inspection of the building, the condition and quantitates of existing steelwork may justify the costs of a careful deconstruction in comparison with demolition, or allow for a possible refurbishment option for a new building application. The candidate building may be then subjected to a two-step assessment process as described below. An overview of the reclamation process from source building to design is presented in Fig. 7.7.

Step 1: preliminary assessment: documentation research and limited inspection

The preliminary assessment should be performed by an experienced chartered engineer, with possible contributions from personnel with fabrication and erection expertise. The preliminary assessment is intended to evaluate the feasibility of reclaiming and reusing the existing steelwork. This assessment needs to consider the expected reuse scenario. The main purpose of this step is to avoid effort and costs of a detailed inspection, deconstruction, documentation and testing of an existing building/steelwork which offers limited opportunities for reuse.

In a first phase, a study of the available collected documentation is envisaged prior to any field assessment. Available engineering reports, including any previous inspection, maintenance or records of possible modifications shall be also reviewed. Specifications (including possible original Welding Procedure Specifications), shop drawings, erection drawings, and construction records shall be reviewed when available.

The date of the original construction date must be identified, as well as the likely materials to be found in the existing building when documentation is not available. Using a similar process, the likely standards that were used for the structural analysis and design shall be identified.

Whenever the structural elements are not visually exposed, measures should be specified to expose a sufficient number of elements for the preliminary assessment (only a representative/limited number of elements need to be exposed for this step - see Table 7.3).

An evaluation of the structural concept and the adequacy of the existing steelwork and design for the possible future reuse scenario shall be undertaken. Mechanisms and nature of possible structural failure shall be evaluated and documented. The preliminary assessment will make sure that any field operation is appropriate and safe for the personnel.

A quantitative/empirical evaluation of the existing steelwork is undertaken within this step. A photographic record and a representative field survey may be created. Sketches can be used to describe the structural concept and relevant details. Preliminary calculations may be undertaken based on existing documentation or based on preliminary material properties assessed by a minimal testing (based only on non-destructive tests – see Appendix A) to assess the adequacy of the proposed reuse scenario. As an alternative, the minimum yield strength according to building location and age may be used for such preliminary calculations (say for a representative frame).

All collected data must be compiled into a report, where any concern/issue found should be documented. Based on the date of construction, the expected steel properties (or preliminary test results) as well as the possible/likely design codes must be identified. The report shall define any urgent safety measures required for the existing building.

Building documentation is extremely valuable for the reuse of existing steelwork. However, it is necessary to ensure that the collected information is updated and refers to the as-build structure. The consistency of the collected documentation must therefore be assessed not only for building geometry and section sizes but also for details (e.g. joints). Table 7.3 proposes guidance to check the compliance of existing building documentation. For structures with detailed construction drawings, a limited on-site inspection is recommended. For other cases, where no documentation is available, a comprehensive on-site inspection is recommended.

Structural members should be checked against available documentation. If building documentation is not available, the members should be compared against relevant catalogues according to be building age and location. Geometric tolerances should be allowed for while assessing the cross-sectional dimensions according to current standards (see Table 6.3).

For the cases where documentation is available, but no compliance is found according to the on-site inspection, the percentage of details and members/cross sections to be checked must include those where no documentation is available (see Table 7.3).

Table 7.3 **Details and members to be checked – preliminary assessment**

<table>
<tr><th>Detailed construction drawings/documentation available</th><th>Percentage of details to be checked for geometry</th><th>Percentage of members checked for cross section dimensions</th><th>Building dimensions and structural solution</th></tr>
<tr><td>Yes
Limited on-site inspection</td><td>10% (min 3 different detail types) – details selected randomly</td><td>10% (min 3 different sections up to all different section) – members selected randomly</td><td rowspan="2">Inspection for a typical portal frame:
✓ Span;
✓ Eaves height;
✓ Apex height;
✓ Frames spacing;
✓ Vertical and roof bracings arrangements;
✓ Eaves struts;
✓ Fly bracings;
✓ Etc.</td></tr>
<tr><td>No
Comprehensive on-site inspection</td><td>25% (min 5 different detail types) – details selected randomly</td><td>25% (min 5 different sections up to all different section) – members selected randomly</td></tr>
<tr><td colspan="4">Note: percentages to be applied to a group of elements with same geometric/cross section and load history/structural application (see section Appendix A for the definition of a group of elements); by way of example, for a portal frame, the three types of details to be checked can be column-base, apex and eaves connections. Different sections can be columns, rafters and vertical or roof bracings.</td></tr>
</table>

The preliminary inspection should assess the condition of the existing steelwork. The inspection should look for evidence that may indicate that the steelwork is not suitable for reuse, so that a more detailed inspection (and costs) avoided. Problems in a considerable number of components such as evidence of fire exposure, damage due to impact loadings, evidence of exposure to abnormal load (say earthquake, snow or wind), section loss due to corrosion, excessive/plastic deformation of the structural elements, plastic local deformations in connecting elements (plates), signs of local (plate) or member buckling (evidence of plastic loading history), fabrication defects, etc., will limit the reusability of the existing steelwork (see sections 6.1 and 6.3.2). For the preliminary assessment, the percentages of members and details proposed in Table 7.3 may be used to assess the steelwork condition. For the comprehensive assessment, all members and details must be visually inspected for defects.

Step 2: comprehensive assessment: detailed survey and inspection

The comprehensive assessment shall be undertaken if the existing steelwork was classified as "*good for reuse*" by the preliminary assessment. The consistency of the collected information and steelwork condition shall be re-assessed according to Table 7.4.

Table 7.4 **Details and members to be checked – comprehensive assessment**

<table>
<tr><th>Detailed construction drawings/documentation available</th><th>Percentage of details to be checked for geometry</th><th>Percentage of members checked for cross section dimensions</th><th>Building dimensions and structural solution</th></tr>
<tr><td>Yes
Limited on-site inspection</td><td>25% (min 5 different detail types) – details selected randomly</td><td>25% (min 5 different sections up to all different section) – members selected randomly</td><td rowspan="2">Inspection for a typical portal frame:
✓ Span;
✓ Eaves height;
✓ Apex height;
✓ Frames spacing;
✓ Vertical and roof bracing arrangement;
✓ Eaves struts;
✓ Fly bracings;
✓ Etc.</td></tr>
<tr><td>No
Comprehensive on-site inspection</td><td>100%</td><td>100%</td></tr>
<tr><td colspan="4">Note: percentages to be applied to a group of elements with same geometric/cross section and load history/structural application (see section Appendix A for the definition of a group of elements); by way of example, for a portal frame, the three types of details to be checked can be column-base, apex and eaves connections. Different sections can be columns, rafters and vertical or roof bracings.</td></tr>
</table>

For the comprehensive assessment, it is recommended that a detailed evaluation of the condition of all existing structural elements condition is undertaken. Such inspection must seek to identify any defects/problems as described for the preliminary assessment.

The evaluation of the coating condition (including blistering, rusting, cracking, flaking or chalking) should be undertaken. Coating toxicity should also be assessed and documented, justifying that the existing system can be reused (see section 7.6.2). Depending of the reuse scenario, structural elements may need to be exposed, which means that careful deconstruction (and partial demolition of non-structural elements) may be required.

Steelwork that can be reused should be sorted into groups/test units according to Appendix A, and should be permanently and physically identified/labelled using a reliable method. Each label should be unique (e.g. number, barcode, QR code or RFID) so that the original reclaimed element source and locations can be traced back to its origin.

If adequate documentation is not available, as-built drawings and a possible 3D BIM model can be produced (depending on the reuse scenario for the reclaimed steelwork). This is clearly not the case if individual elements are intended to be reclaimed for future reuse applications.

The geometric assessment of the overall structure in-service performance and geometry can be undertaken thorough laser scanning, LIDAR or photogrammetry, which offer better precision and more reliable data in comparison with other techniques. These techniques will allow for an easy evaluation of the in-service deformed shape of the building.

After the comprehensive assessment, only reusable elements are grouped into test units according to Appendix A. The necessity of testing is related to the available documentation. Based on the reclaimed steel classification (A, B or C), appropriate testing procedures shall be implemented according to Appendix A. If any structural element is classified as non-reusable in the current condition, it can be repaired/refurbished or sent for recycling.

Data collection and documentation from building inspection

The building owner, designers, fabricators and on-site contractors, and neighbouring building owners can be contacted to collect available information.

The process should try to provide answers to the following aspects:

- Collect drawings, CAD drawings, 3D BIM models, mill certificates, photographic evidence etc. for the as-build structure;
- Collect information about the design such as calculation notes, loading history, etc.;
- Records of interventions (e.g. expansions, modification, etc.);
- Records from any possible incident in the building/area: fire, earthquakes, etc.;
- Inspection and maintenance records;
- Date and place of construction of the original building;
- Building owner's manual (O&M manual);
- Identify fabricator, erector, designers, architects and other actors.

The following data should be recorded from the existing structure and steelwork in a report:

- A description of the structure and its use. This should include a description of how the building is stabilised;
- The age of the structure, which may be from records, or local/anecdotal information;
- A preliminary listing of the steel members;
- A preliminary inspection of the members for damage, obvious repairs, significant corrosion, etc.;
- Any evidence of plasticity or excessive and permanent deformations.

Overview of the steel reclamation process

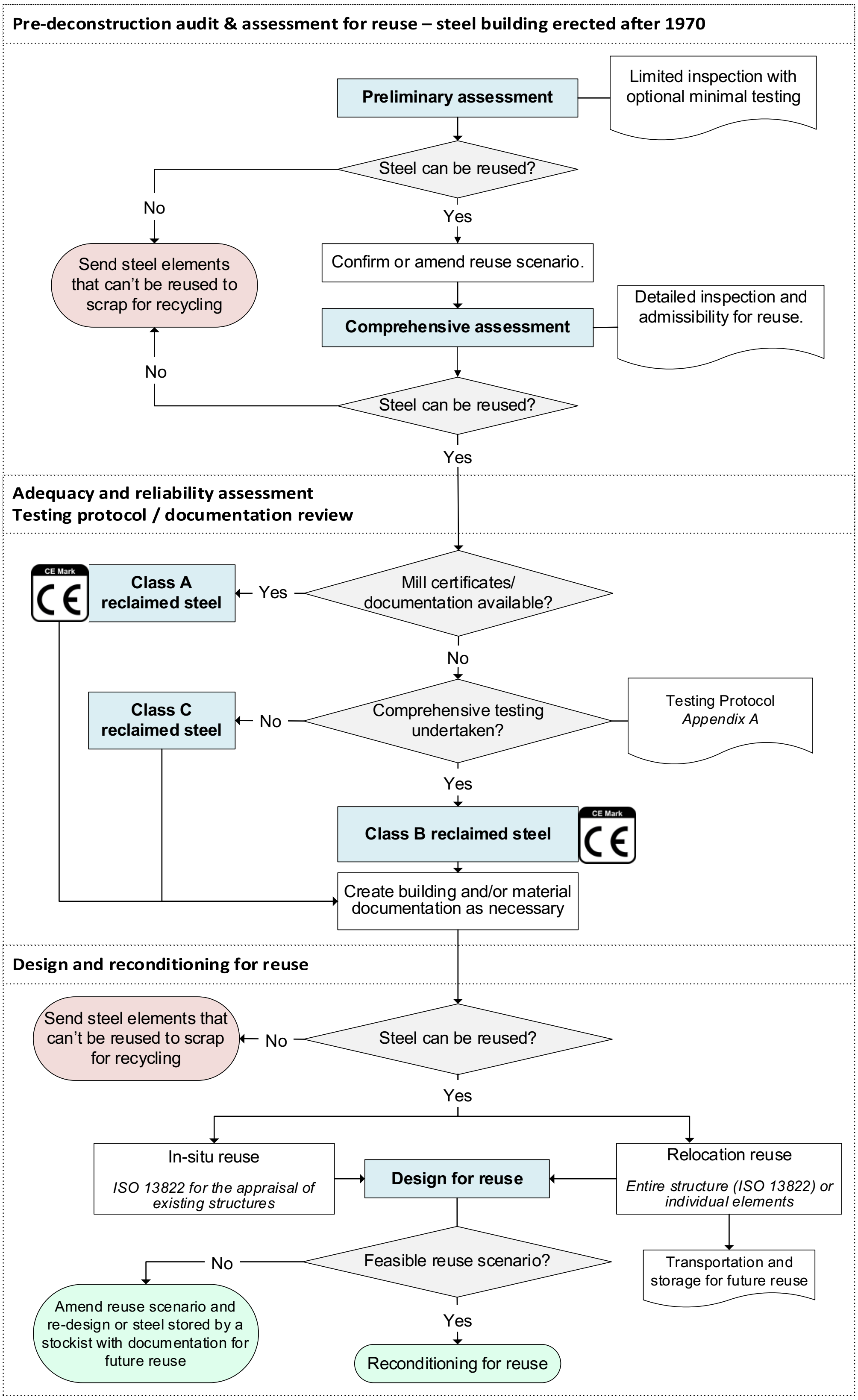


Fig. 7.7 Overall process: from reclamation to reuse

7.5.4 Safe demounting techniques

The demolition industry has undergone major transformation within the last few decades. Traditionally, demolition was a labour intensive, low skill, low technology and poorly regulated activity, dealing mainly with the disassembly and demolition of simply constructed buildings. More recently, it has followed the trend of all major industries and mechanized the processes by replacing manual labour with machines. This is because of the increased complexity in building design, financial pressures from clients, health and safety issues, regulatory and legal requirements and advances in plant design, in particular high-reach excavators with specialist attachments. The industry now employs fewer, but more highly skilled operators and very expensive specialized equipment.

Traditionally, much of the demolition contractors' income was from the sale of salvaged and recycled materials. Today income is mostly generated from the contract fee; demolishing the building as quickly and as safely as possible. Nevertheless, substantial amounts of materials and components are recovered or reclaimed but is generally downcycled and therefore is not used to its fullest potential.

The selection of demolition method is dependent on several factors concerning the physical aspects of the building to be demolished, safety and economic issues, i.e. the location of the building, the type of structure and materials involved, the space available on-site for segregation and storage, health and safety of operatives undertaking demolition work, permitted levels of nuisance and also, on the time and money available.

Demolition practice is building and location specific and will be subject to local regulatory requirements. Although there are a range of methods for different building types and scenarios, demolition of single-storey, steel-framed buildings are generally done using excavators to destructively pull down the building. The structural steel members are then sheared into suitable lengths for handling, transportation and recycling using hydraulic shear attached to an excavator boom. If shearing is not possible, beams are manually cut by burning and either dropped to the ground or supported using a crane or excavator and them lowered to the ground.

Fig. 7.8 Steel shear attachment pulling-down and shearing a steel structure

Deconstruction

Deconstruction with a view to reclaiming elements of the building for reuse is currently rarely done. Despite the environmental and the potential economic benefits, relative to demolition, there are two key barriers to deconstruction. These are:

- The additional time (and associated cost) required relative to traditional demolition techniques. Demolition is generally part of the redevelopment of an existing site and redevelopment programmes frequently do not allow the additional time required to deconstruct existing buildings.
- The health and safety implications of deconstruction in particular linked to working at height.

Buildings now reaching their end-of-life, were not constructed with thought of how they could be deconstructed and their components reclaimed for reuse in a new building application. However, for conventional steel-framed, single-storey buildings, the form of construction makes the process relatively straightforward, i.e. reversing the construction sequence and deconstructing the following elements in sequence:

- Non-structural elements/equipment;
- Flashing elements;
- Cladding;
- Secondary structure;
- Primary structure;

The suggested deconstruction protocol relies on the deconstruction of single bays, rather than the deconstruction of an entire building layer. The general process can be summarized in the following steps:

- Before any deconstruction procedure, all non-structural elements and equipment must be removed/detached from the structure;
- Deconstruction of the building will start in one end of the building, where flashing and cladding elements are removed in the vicinity of the frame that will be first deconstructed;
- Based on the secondary steelwork detailing, it is likely that at least two roof cladding bays need to be removed to allow for the secondary steelwork deconstruction;
- For southern European countries, it is likely that rafters will not be able to resist their own self-weigh without any intermediate flexural and lateral buckling restraint; rafters will need to be supported by a lifting crane while the secondary steelwork connected to the rafters is deconstructed;
- After removing secondary elements, with the rafters supported, the unbolting of the eaves connections takes place and the pair or individual rafters is deconstructed;
- The apex connection can be then easily deconstruction on the ground level;
- After removal of a pair of rafters, side rails and wall cladding can be removed for the bay without rafters;

- Columns must keep an out of plane restraint (say a pair of side rails, or eaves strut) before the column is supported by the lifting equipment; when the columns is supported, out of plane restraints can be removed;
- Individual columns can then be safely individually deconstructed;
- A similar procedure is repeated until the whole building is deconstructed;
- Salvaged members must be labelled before transportation form the site; it is recommended that the labelling sequence reflects the deconstruction sequence.

More detailed guidance is provided in the Deconstruction protocol developed during the PROGRESS project.

7.6 Reconditioning of reclaimed products

7.6.1 General

All fabricated steelwork should conform to the requirements of EN 1090-2 or EN1090-4. Product specific standard and/or performance requirements (such as for a cladding system) may need to be followed while dealing with existing or reclaimed products. The following section provides guidance to some issues that can be encountered when dealing with existing steelwork.

7.6.2 Existing coating systems

Reclaimed steel will usually have an existing protective coating (paint) to provide corrosion resistance, and in some cases, an intumescent coating for fire resistance. In most situations, it is envisaged that any existing coatings on reclaimed steelwork should be entirely removed prior to fabrication. The reuse of steelwork with its original protection is likely to be limited to situations when the entire structure is dismantled, relocated and reconstructed, largely in its original form.

If the reuse of steelwork with its existing corrosion protection is contemplated, the following issues should be considered:

- Existing corrosion protection systems are likely to need remedial works after dismantling the structure, and after any fabrication activity,
- Existing corrosion protection systems might contain hazardous substances, prohibited under current legislation (see below),
- Although corrosion protection systems for internal steelwork might be more durable than originally anticipated, the original level of protection is likely to have diminished and to be less than recommended under current requirements;
- The reuse of steelwork with existing corrosion coating may be limited to a building owner decision to accept the risk of undertaking such practice.

Intumescent coatings are highly sensitive to humidity, and the coating thickness is defined based on the degree of structural utilization of the member and the paint properties. The disassembly process, transportation and storage are likely to cause damage to the coating and an accurate assessment of the paint properties is not feasible. For these reasons, no reliance should be placed on existing fire protection coatings. However, most of the steelwork on single-storey buildings do not require fire protection (examples of the UK and France). Furthermore, intumescent paints were not used before the late 1990s.

Old paint coatings may contain hazardous chemicals, e.g. lead, chromium that should be removed. If the coating is a bright colour, especially red, yellow or green, and the steel originates from a building in the 1970s, there is a good possibility that the coating system is based on a lead pigment. Testing of existing coating to determine chemical properties is therefore important, for example by on-site X-ray fluorescence testing.

Dry abrasive blasting is one of the most efficient methods of surface preparation for total coating removal and is capable of removing all contaminants from the surface, including paint, rust and mill scale. Abrasive blasting is effective on almost any configuration of steel. It also imparts a surface profile (roughness) into the substrate to promote coating adhesion, and it is one of the most productive methods of surface preparation. A number of different abrasives are used for abrasive blasting. The most typical are expendable abrasives, i.e. sands and slags.

Recyclable abrasives, such as steel, aluminium oxide, and garnet, can also be used. Recyclable abrasives minimise the amount of waste generated because the usable abrasive can be reclaimed for reuse. The waste must then be evaluated through the End-of-Waste criteria [85]. The decision about the hazardous nature of the product is in many cases based on the allowed concentrations of the regulated substances, see EU REACH legislation [86].

In some cases, over-coating can be an option. *Overcoating* is defined by some as spot cleaning and priming degraded areas, cleaning intact paint, and applying a lead-free system over the existing paint. The structural members to be over-coated and the existing coating system must be carefully inspected to ensure the suitability of over-coating. This process does not require extensive surface preparation. The surfaces to be over-coated may be low-pressure power washed or hand washed using a mild detergent and water solution. The wash water should be collected and tested to ensure that it does not breech the hazardous criteria for lead contamination.

For the evaluation of degradation of coatings, ISO 4628 can be used:

- ISO 4628-1: Part 1: General introduction and defect designation system [87];
- ISO 4628-2: Part 2: Assessment of degree of blistering [88];
- ISO 4628-3: Part 3: Assessment of degree of rusting [89];
- ISO 4628-4: Part 4: Assessment of degree of cracking [90];
- ISO 4628-5: Part 5: Assessment of degree of flaking [91];
- ISO 4628-6: Part 6: Assessment of degree of chalking by tape method [92] ;
- ISO 4628-7: Part 7: Assessment of degree of chalking by velvet method [93];
- ISO 4628-8: Part 8: Assessment of degree of delamination and corrosion [94];
- ISO 4628-10: Part 10: Assessment of degree of filiform corrosion [95].

In order to remove the paints, the steel surface needs to be cleaned and the steel substrates prepared, in accordance with the following international standards:

- EN ISO 8501: Visual assessment of surface cleanliness [96];
- EN ISO 8502: Tests for the assessment of surface cleanliness [97];
- EN ISO 8503: Surface roughness characteristics of blast-cleaned steel substrates [98];
- EN ISO 8504: Surface preparation methods [99].

New paint coating system shall be defined according to EN ISO 12944, which is divided into the following parts:

- ISO 12944-1: Paints and varnishes — Part 1: General introduction [100];
- ISO 12944-2: Paints and varnishes — Part 2: Classification of environments [101];
- ISO 12944-3: Paints and varnishes — Part 3: Design considerations [102];
- ISO 12944-4: Paints and varnishes — Part 4: Types of surface and preparation [103];
- ISO 12944-5: Paints and varnishes — Part 5: Protective paint systems [104].

7.6.3 Reclaimed steel members with corrosion

High levels of corrosion are not accepted as the geometric properties of the cross section may be compromised. However, small levels of localized corrosion may be accepted if the geometric properties of the cross section are not diminished by more than 5% of the minimum thicknesses [61] specified by the product standard or manufacturers tables (see Table 6.3). The 5% allowance shall be added to geometric tolerances.

The evaluation of the effects of corrosion shall be measured after implementing appropriate steel surface treatment according to EN ISO 12944-4 [103].

7.6.4 Bolt holes and welded parts in reclaimed steel

The reuse of members with holes for the structural bolts is permitted if all geometric and design requirements according to EN 1993-1-1 and EN 1993-1-8 [12] are fulfilled.

If bolt holes are located within the critical cross-section and reduce the cross-section by more than 15%, the net cross-sectional properties should be used in member verification [61].

If present, larger holes, e.g. for the passage of services, must be assessed on an individual basis during member verification.

In general, it is recommended that redundant welded fittings, e.g. stiffeners or cleats, need not to be removed.

See also sections 6.3.2 and 7.6.6

7.6.5 Existing connections

Special care is needed if existing connections are to be re-used. Any welding should be subject to careful inspection and adequate testing.

As a general recommendation, at least the same amount of weld testing required by EN 1090-2 (Table 24) should be applied to reclaimed steel elements. Visual inspection of 100% of the welds is mandatory. Further guidance is provided in Annex A.

7.6.6 Remedial works on existing steelwork

Additional fabrication work to the reclaimed steel components may be required, as follows (see also section 6.3.2:

- Re-straightening sections to meet specified tolerances;
- Splicing to create longer lengths, which may require welding or a bolted solution. Type of solution or even acceptance of the solution may be limited by any client visual/appearance requirement;
- Possible repairing unused holes: client may not accept pre-existing and visually exposed bolt holes in the re-used sections;
- Removing unused attachments and brackets: client may not accept pre-existing and visually exposed bolt holes in the re-used sections; (additional costs).

7.6.7 Claddings

In general, the reuse of sandwich panels is possible for two scenarios: same site, different configuration and different site, different configuration. Up to now, sandwich elements are fixed directly to the substructure by push-through mounting by direct or indirect connections. In order to promote the reuse of sandwich panels, a new method of fastening sandwich panels could be the one-sided fixation. One-sided fixation means a screw which is only fixed on the outer metal sheet of a sandwich element. This allows an over-cladding of sandwich panels (see Fig. 7.9).

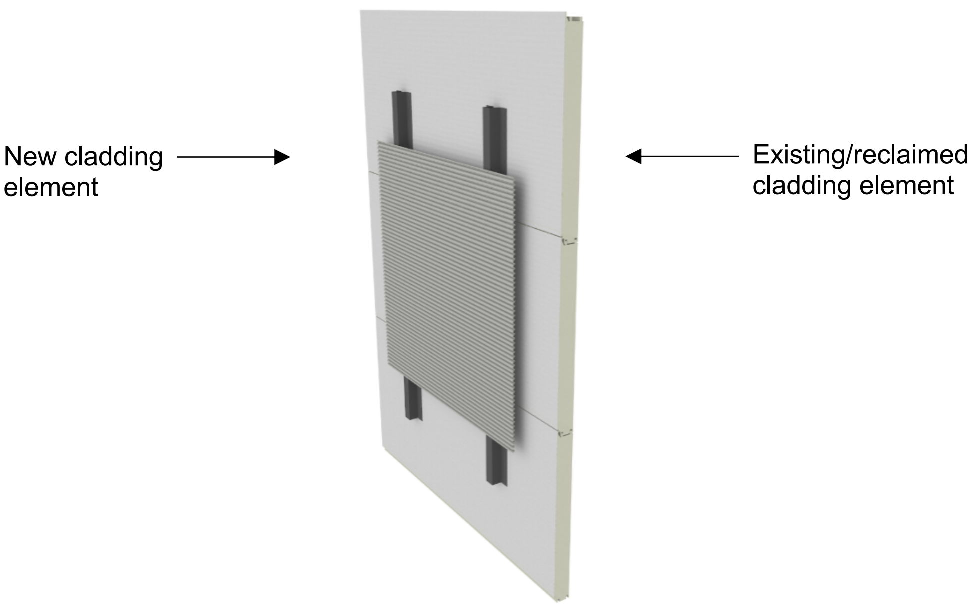


Fig. 7.9 One-sided fixation of sandwich panels

This kind of fixation offers potential for promoting the reuse of sandwich panels for the following reasons:

- In case of a planned renovation of the building envelope, the old sandwich panels could be reused with the aid of the one-sided fixation by fixing distance profiles from the outside, which allow the sandwich panels to be covered with new facade elements.

- The sandwich panel is not penetrated and therefore, there is no weakening of the building physical properties of the sandwich panel.
- The one-sided assembly process brings advantages in the construction process.
- If the one-sided fixation is used at the inner metal sheet to fix the sandwich panel to the substructure, there is a further advantage: The outer metal sheet shows no changes. After the dismantling only holes are visible from the inside, the outer sheets remains untouched. This would greatly facilitate the reuse of the sandwich panel.

8 STRUCTURAL ANALYSIS AND DESIGN FOR EXISTING STEELWORK

8.1 General requirements

This section discusses several considerations that may affect the design of structures using reclaimed steel members. The principles of Limit States 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 given in EN 1990. For the remaining intended working life, the structure shall be designed and constructed to:

- 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 a 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 length of the working 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 procedures 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. In fact, the designer relies on ductility for a number of aspects of design, including redistribution of stress at the ultimate limit state (ULS), in the design of bolt groups and in the fabrication process for welding, bending, and straightening. For designs using reclaimed steel, ductility and toughness reductions can be neglected because, under normal conditions for building construction, strain demands applied in service are less than 1.5% and therefore this will not affect the structural behaviour significantly.

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.

8.2 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 actions, Q_d, are obtained from the characteristic values, Q_k, based on a 50-year reference period and a 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:

$$\beta = -\Phi^{-1}\left(P_f\right) \tag{8.1}$$

where Φ^{-1} is the inverse standardised normal distribution.

The general actions on single-storey buildings are defined in EN 1991-1-1. 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 for snow loads, and Note 4 in Clause 4.2 in EN 1991-1-4 for wind loads (see section 10.1.6).

EN 1990 defines three Consequence Classes (CC), depending on the consequences of failure or malfunction of the structure, which are associated with three different Reliability Classes (RC) as follows:

- CC1: *low* consequence for loss of human life, and economic, social or environmental consequences *small or negligible*, associated with RC1 ($\beta_{50\text{-year}} = 3.3$),
- CC2: *medium* consequence for loss of human life, economic, social or environmental consequences *considerable*, associated with RC2 ($\beta_{50\text{-year}} = 3.8$),
- CC3: *high* consequence for loss of human life, or economic, social or environmental consequences *very great*, associated with RC3 ($\beta_{50\text{-year}} = 4.3$).

It is also noted that designs with the partial factors given in the Eurocodes generally leads to a RC2 structure with a β value greater than 3.8 for a 50-year reference period.

The design working life of the structure is not explicitly linked to the consequence class in EN 1990, and can be understood as an assumed period of time for which a structure is to be used for its intended purpose without any major repair being necessary. Clause 2.3(1) of EN 1990 gives the following categories together with indicative design working life for permanent structures:

- Category 3, with a notional design working life of 15~30 years,
- Category 4, with a notional design working life of 50 years,
- Category 5, with a notional design working life of 100 years.

Structures designed to the Eurocodes should perform and remain fit for the appropriate working life. Typical buildings are designed for a working life of 50 years, i.e. category 4, for a *normal degree of reliability* and RC2 ($\beta_{50\text{-year}} = 3.8$). If the design working life is limited, it may be reasonable to specify a *lower than normal degree of reliability*, $\beta_{50\text{-year}} < 3.8$, but $\beta_{50\text{-year}} \geq 2.5$, which is the limit value for human safety according to ISO 13822 [105]. Likewise, if the design working life is increased, say to 100 years, then $\beta_{50\text{-year}} > 3.8$, corresponding to a *higher than normal degree of reliability*. It should also be highlighted that these β indices and the corresponding probability of failure are only notional values that do not necessarily represent actual failure rates.

Gulvanessian et al. [106] clearly explain that the β indices are used as operational values for code calibration purposes and 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*.

Clause 2.2(6) of EN 1990 states that the 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 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.

8.2.1 Partial factors for actions

The partial factors for actions allow for the variability of loading in which loads may be greater than expected, and also self-weight loads that act to counteract overturning may be less than intended.

A multiplication factor K_{FI} may be applied to the partial factors for unfavourable actions in fundamental combinations for persistent design situations, see Clause 6.4.2.2(3) of EN 1990 and Table 8.1. The notation in the table is as follows, in which γ_F are the recommended values:

$G_{k,j,sup}$	is the upper characteristic (superior) value of permanent action *j;*
$G_{k,j,inf}$	is the lower characteristic (inferior) value of permanent action *j;*
$Q_{k,1}$	is the leading variable action;
$Q_{k,i}$	is the accompanying variable action *i;*
$\psi_{0,i}$	is a combination factor (for variable action *i*);
ξ	is a reduction factor for unfavourable permanent actions, defined in the National Annexes for use in a country.

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. [107] and [108]. This is justified by the fact that, for existing structures, a shorter design life is often assumed and accepted. Likewise, for designs with reclaimed steelwork, it is reasonable to consider the option to assume a shorter design life, to, say, 15-30 years (category 3 above), which corresponds to RC1. This leads to a multiplication factor of 0.9. It is recommended, however, that the fundamental combinations of actions are assessed based on Eq. (6.10), top line in Table 8.1, as highlighted, which leads to a higher value of reliability as compared to Eqs. (6.10a) and (6.10b). See section 8.2.2.

Table 8.1 **Design values of actions for strength (STR) using Eq. (6.10), orEqs. 6.10a and 6.10b in EN 1990**

CC/RC	Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions (i > 1)
		Unfavourable	Favourable		
1 (K_{FI} = 0.9)	**Eq. 6.10**	**1.215 $G_{k,j,sup}$**	**1.0 $G_{k,j,inf}$**	**1.35 $Q_{k,1}$**	**1.35 $\psi_{0,i}$ $Q_{k,I}$**
	Eq. 6.10a	1.215 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.35 $\psi_{0,1}$ $Q_{k,1}$	1.35 $\psi_{0,i}$ $Q_{k,I}$
	Eq. 6.10b	$\xi \times$ 1.215 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.35 $Q_{k,1}$	1.35 $\psi_{0,i}$ $Q_{k,I}$
2 (K_{FI} = 1.0)	Eq. 6.10	1.35 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.5 $Q_{k,1}$	1.5 $\psi_{0,i}$ $Q_{k,I}$
	Eq. 6.10a	1.35 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.5 $\psi_{0,1}$ $Q_{k,1}$	1.5 $\psi_{0,i}$ $Q_{k,I}$
	Eq. 6.10b	$\xi \times$ 1.35 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.5 $Q_{k,1}$	1.5 $\psi_{0,i}$ $Q_{k,I}$
3 (K_{FI} = 1.1)	Eq. 6.10	1.5 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.65 $Q_{k,1}$	1.65 $\psi_{0,i}$ $Q_{k,I}$
	Eq. 6.10a	1.5 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.65 $\psi_{0,1}$ $Q_{k,1}$	1.65 $\psi_{0,i}$ $Q_{k,I}$
	Eq. 6.10b	$\xi \times$ 1.5 $G_{k,j,sup}$	1.0 $G_{k,j,inf}$	1.65 $Q_{k,1}$	1.65 $\psi_{0,i}$ $Q_{k,I}$

8.2.2 Feasible scenarios to adopt a lower design life time

In the previous section, it was suggested that the combination factors for actions could be slightly reduced while designing with reclaimed steel by assuming a lower expected structure life time. It is recommended that such option require 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). The examples where the lower partial factors for a notional design working life of 15-30 years can be used are: (i) existing buildings (in situ reuse) or (ii) the cases where the whole building is relocated to a different location.

For the cases where a new structure is designed, while promoting the practice of steel reuse, it is possible to adjust the influence areas of the reclaimed elements (for example, by adjusting the spacing of a floor beam or frame) so that the load level is acceptable according to the standard reliability requirement for a notional design working life of 50 years according to EN 1990 (standard design process).

8.2.3 Partial factors for resistance

The partial factors on resistance defined in EN 1993-1-1 are summarised in Table 8.2. 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 8.3. The values in these Tables are given for new steels, and were obtained from test data obtained between 1969 and 1980 for steel produced to EN 10025, see [109] and [110], and later, in 2002 [111].

The use of reclaimed steel has been restricted to single-storey 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 vary from the strength used in calculations. Thus, the steelwork designer can safely adopt the same values from Table 8.2 for γ_{M0} and γ_{M2} in designs using reclaimed steel.

Table 8.2 **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 8.3 **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**
$a = \min\left(1.10, \frac{0.9f_u}{f_y}\right)$; [(i)] For nonlinear analysis, assume γ_{M0} = 1.10															

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 *column curves*, which give buckling resistance as a function of member non-dimensional slenderness. Although the salvaged members have to meet all geometrical tolerances, it is recommended to increase γ_{M1} for stability verifications in designs with reclaimed steel in order to provide an additional margin of safety. This leads to a recommended value of $\gamma_{M1,mod} = K_{\gamma M1}\ \gamma_{M1}$ with $K_{\gamma M1}$ = 1.15 (the derivation of $K_{\gamma M1}$ is presented 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 $K_{\gamma M1}$ 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. See also section 8.7.

8.3 Structural (static) analysis

Global structural analysis for ULS shall be carried out in accordance with the basic principles from EN 1993, 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 analysis, depending on the horizontal flexibility of the structure, which dictates if ignoring second-order effects may lead to an unsafe approach due to underestimation of internal forces and moments due to those effects. For structures sensible to global second order effects, it is recommended that global effects P-Δ are accounted for by a geometric non-linear analysis (usually undertaken using software) or using the amplification factor according to EN 1993-1-1 section 5.2.2 (5)b.

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

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 (see section 8.4).

The serviceability limit state, SLS, defines 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 8.6).

8.4 Ultimate limit states

8.4.1 Design of members: resistance of cross-sections

The rules set out in Clause 6.2 of EN 1993-1-1 may be applied without restrictions in the design checks for cross-section resistance taking into consideration the cross-section classes from Clause 5.5. 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 (see Section 8.2.3).

8.4.2 Design of members: stability

For stability verifications of members, account should be taken of local imperfections, in accordance with Clause 5.3.4 of EN 1993-1-1. Usually, this is treated implicitly within the procedures for checking individual members in Clause 6.3. In the case of members using reclaimed steel, it is recommended to substitute γ_{M1} for $\gamma_{M1,mod}$ (see Section 8.2.3).

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 considered [61]. The relative (or non-dimensional) slenderness, however, should always be determined for the gross cross-section.

8.4.3 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}.

8.4.4 Design of frames

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

In simple construction, the joints between 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 account of the moments that are transferred among the members. This can be performed from a global elastic.

Clause 5.2.2 of EN 1993-1-1 permits all forms of geometrical and material imperfections in a second-order global analysis of frames. This approach requires specialist software and is seldom used in practice. Option b) of Clause 5.2.2(3) 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 5.3.2.

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 5.3 of EN 1993-1-1, 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 lead 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 to Clause 5.3.3 of EN 1993. Imperfections are taken into account 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 5.3.3(4).

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 in order to reduce the cumulative tolerance errors.

8.4.5 Design of secondary structural elements

Secondary steelwork in single-storey buildings is typically in the form of cold rolled 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 into the bracing system (see section 9.4.6).

Section 10 of EN 1993-1-3 gives guidance on the design of purlins and side rails. Because these elements are usually proprietary sections, suitable sections have been developed and tested by manufacturers, who provide design data in the form of design tables or software.

8.4.6 Connections design

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 [61]. However, it is recommended that the existing welds should be carefully inspected (see also section 7.6.5).

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 [112] and [113].

8.5 Seismic design considerations

Designers should note that the seismic design of single storey buildings do not usually require special design consideration according to EN1998-1 section 6 [30]. Single story building structures are generally treated as low ductility class systems (DCL), which means that the design requirements from EN1993-1-1 are sufficient. The trade-off in this practice is that a lower 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 DCL concept is assumed in design, there are no concerns in utilizing the reclaimed steel elements for structures subjected to seismic action, but a behaviour factor equal to 1 is recommended in the design of the reclaimed steelwork.

The recommendations provided in the current publication 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). For such cases, it is only recommended to allow for reclaimed steel elements if those elements are used at least under one of the following conditions: (i) as members of the gravity or secondary load resisting systems (not part of the lateral load resisting system, such as pin-ended floor beams), or (ii) as elements that are part of a DCL structure.

The assessment and testing procedures proposed in section 7.5.3 and Appendix A, respectively, are in agreement with the requirements proposed in EN1998-3 [114] for existing buildings. For the cases where 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 [115] [116].

8.6 Serviceability limit states

8.6.1 Deflections and displacements

Serviceability limit state conditions (deflections, displacements, vibrations) are generally not codified. The maximum allowable deflection/displacement in portal frames will depend on many factors, such as appearance, building use, or cladding type (for which manufacturer's recommendations and guidelines should be followed). As a result, acceptable limits should be agreed between the client, designer, and competent authorities.

The calculation of vertical and horizontal deflections/displacements is based on the characteristic load combination according to EN 1990, which may not include permanent loads. Different limits for serviceability (deformations, displacements) can be found in different practices/countries. The following range of limits are often used across Europe:

- For vertical deflections of rafters and beams, a deflection of $L/180$ to $L/250$ (where L is the beam span) for flexible cladding in roofs that are accessible only for maintenance, up to a deflection of L/360 for other claddings and/or roof applications;
- For horizontal displacement limits at the eaves, a maximum of $H/150$ for flexible cladding to $H/300$ for brittle cladding is considered (H is the building height at the eave).

Elastic analysis is used to determine the deflections of the frame at the serviceability limit state. The frame is often pre-set such that the deflections under the permanent actions are not significant. The degree of pre-set is partly a matter of calculation and partly a matter of experience (the steelwork contractor should be consulted if pre-setting the frame is being considered).

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

Cladding

Limits on differential deflection between adjacent portal frames are necessary to prevent the fixings between the sheets and the frame from becoming overstrained, resulting in tearing of the sheeting, and leakage.

Portal frames cladding in steel sheeting deflect significantly less than the deflection calculated for the bare frame. This is due to the sheeting acting as a stressed skin diaphragm, which provides a considerable stiffening effect to the structure. The actual deflection depends on the building proportions and cladding type, but reductions in horizontal deflections of over 50% (from those calculated for the bare frame) are typically found on as build structures.

Gables

A sheeted and/or braced gable end is very stiff in its own plane and the deflections can be ignored. The calculated differential deflections between the end frame and the adjacent frame (at the ridge and at the eaves) can be very high. This differential deflection will always be modified by the presence of the roof sheeting and roof bracing, particularly if the roof bracing is located in the end bays.

Masonry

When brick or blockwork side walls are constructed such that 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 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 pitch roofs and flat roofs, the possibility of ponding of water on the roof should also be considered. The recommended minimum roof slop is 3^{o} after the vertical deflection is taken into account. 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.

Cranes

Where crane girders are supported directly by portal frames, the need to control deflections at the crane level is likely to result in stiffer sections for the frames. The deflection and displacements should be determined in agreement with the client and the crane manufacturer. For such structures, the use of fixed bases is often used as a mean to more efficiently control the deflections and displacements. Recommendations from EN 1993-6 [117] should be followed for the design.

Assessments of serviceability limits based on cladding performance (UK)

The recommendations presented in Table 8.4 and Table 8.5 represent the current practice in the UK for the limits of deflections and displacements to assess the serviceability performance of portal frames - [118]. Only deflections due to variable actions are considered.

Table 8.4 **Recommended deflection limits for portal frames: horizontal deflections (UK)**

Horizontal deflection at eaves		
Type of cladding	*Absolute deflection*	*Differential deflection relative to adjacent frame*
Side cladding		
Profiled metal sheeting	$\leq h/100$	-
Fibre reinforced sheeting	$\leq h/150$	-
Brickwork	$\leq h/300$	$\leq (h^2 + b^2)^{0.5} h/600$
Hollow concrete blockwork	$\leq h/200$	$\leq (h^2 + b^2)^{0.5} h/500$
Precast concrete units	$\leq h/200$	$\leq (h^2 + b^2)^{0.5} h/330$
Roof cladding		
Profiled metal sheeting	-	$\leq b/200$
Fibre reinforced sheeting	-	$\leq b/250$

Table 8.5 **Recommended deflection limits for portal frames: vertical deflections (UK)**

Vertical deflections at ridge	
Type of cladding	*Differential deflection relative to adjacent frame*
Profiled metal sheeting	$\leq b/100 \; and \; \leq \; (b^2 + s^2)^{0.5} h/125$
Fibre reinforced sheeting	$\leq b/100 \; and \; \leq \; (b^2 + s^2)^{0.5} h/165$

The calculated deflections for the criteria in Table 8.5 and Table 8.4 are those due to:

- Wind actions;
- Imposed roof loads;
- Snow loads;
- 80% of combined (wind and snow loads).

Feedback on these recommendations indicated that some of these values are more stringent than necessary. The values of *h*, *b*, and *s* are defined in Fig. 8.1. The height h should be taken as the height at the eave.

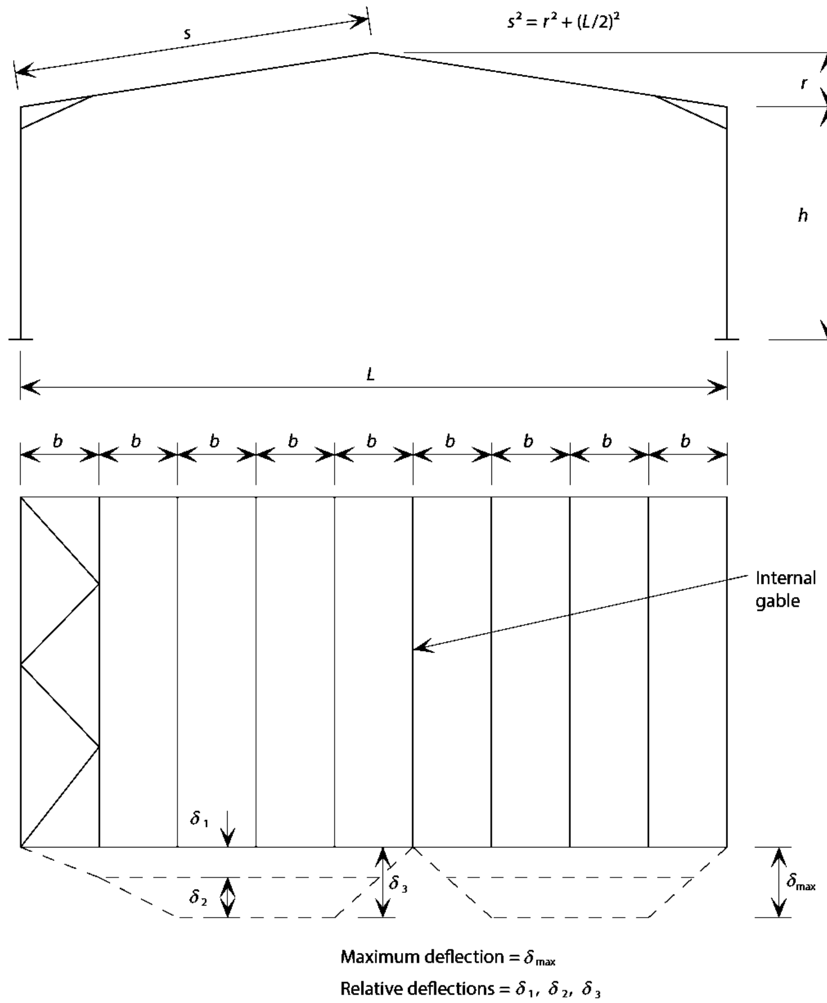


Fig. 8.1 Dimensions to be used in determining deflection limits [118]

8.6.2 Connection slip

In bolted connections with normal clearance holes, such as standard round holes and slotted holes loaded transversely to the axis of the slot, the amount of slip is small and it is not likely to have serviceability implications. In structures such as roof trusses, where the effect of the connection slip in addition to other elastic deflections will produce large movements, the use of slip-critical joints (category B) should be considered.

8.7 Commentary on existing buildings – in-situ reuse

Structures designed and constructed based on existing/old codes/standards may be considered appropriate for future use provided that the following conditions are met simultaneously:

- The structure has demonstrated satisfactory performance during building lifetime;

- There were no changes for a sufficiently long time, which could significantly increase the actions on the structure or affect its durability; and no such changes are anticipated;
- Careful inspection does not reveal any evidence of damage, distress, deterioration or excessive deformations that may indicate overloading;
- The structural system is reviewed, including investigation of critical details, member sizes and building global stability;
- There will be no changes to the structure and in its use that can significantly affect the applied loads;
- Predicted deterioration considering the present structure condition and planned maintenance ensures sufficient durability.

For the cases where changes on loads and/or use are expected, a re-assessment of the building structural performance is required. Documentation may not be available for such buildings, which would classify the existing steelwork as Class C according to Fig. 6.1 and Fig. 6.2 if the testing procedure according to Appendix A is not undertaken.

It is envisaged that achieving CE marking for Class C reclaimed steel may not be possible nor accepted by relevant authorities. This is the case of reclaimed steelwork that would be reused on a different application or location by means of individual elements or the whole structure. If a “new” building is being erected, it is likely that Class A or B steel according to Fig. 6.1 would will be required, allowing for CE marking.

However, this is clearly not the case for an existing building with a possible in-situ reuse scenario (not deconstructed). Such scenario will not require CE marking nor has most of the recognized uncertainties while using reclaimed steel. The so-called Class C reclaimed steel is an approach that is nowadays used while assessing existing structures both for buildings and bridges [119], which can be therefore explore for existing single-storey buildings.

For in-situ reuse scenarios, the proposed limitations for type of global analysis (see section 8.3) and revised material partial factor for member buckling resistance (see section 8.2.3) need not to be considered, which means that design procedures and types of global analysis allowed for by EN 1993-1-1 or other relevant standards can be used.

However, the condition of existing steel elements must be assessed for existing buildings, evaluating material properties (if necessary) and geometric tolerances as specified in 7.5.3 and Appendix A. Bow imperfections may need to be evaluated for existing columns. The bow tolerance according to EN 1090-2 and second order effects due to the strut action must be considered for the in-service load assess the value of actual bow imperfection.

Guidance from sections 8.2.1, 8.2.2 and 10.1.6 may still be applied for in-situ reuse scenarios.

8.8 Cranes

The reuse of structural elements subjected to fatigue is outside of the scope of the current publication. If the single storey building has cranes, a careful assessment of the fatigue remaining life time needs to be undertaken. This may be a necessity for in-situ reuse applications. The critical fatigue details are often related to the crane runway beams and to the brackets that provide support to such elements, which means that there are no major concerns in reusing the remaining steelwork on a reuse scenario where no cranes are to be installed. Further guidance about fatigue assessment of existing structures can be found on reference [120].

8.9 Remedial works for structural design

Nominally pinned base connections may be stiffened to increase the structural performance of the reclaimed steelwork. Stiffer connections (say semi-rigid) can offer an intermediate design outcome and suit the new application. However, such scenarios are not easy to apply for in-situ reuse, as foundations would not be prepared to accommodate such behaviour.

Portal frames reclaimed from an existing building may be used for higher load levels by reducing the frame spacing, which will need interventions on the secondary steelwork and bracing systems. The success rate of secondary steelwork is low, which means that this practice may be suitable for most of the reuse scenarios. Bracing systems often comprise 10-15% of the overall structure weight, which mean that modifications on such members are a still a cost-effective option which can be considered.

For member buckling design, extra discrete restraints and/or inclined bracings may be used to increase the buckling resistance of members by reducing their buckling length.

For excessive deformations, pre-cambering may be used to achieve project requirements for a building constructed in a new location.

The load bearing capacity of floor systems may be increased by introducing new secondary elements between existing secondary elements. For primary beams, welding plates to increase the cross-section resistance is an option. To control serviceability criteria, pre-cambering of reclaimed elements to be used in new applications may be considered.

Part 2: Recommendations for future single-storey buildings

9 RECOMMENDATIONS FOR FUTURE SINGLE-STOREY BUILDINGS

9.1 General

Part 2 of this document addresses the ways in which new single storey structures may be designed to facilitate the greater reuse of steel structures. The focus is on steel members and secondary components that are used in single-storey buildings such in industrial buildings, large retail units, warehouses, and how they may be designed in the first cycle of use to be demounted easily so that their components can be reused in future buildings. This also covers the connections between structural and non-structural elements, as presented in Section 10.

9.2 General principles for design for disassembly and reuse

The amount of steelwork that can be reclaimed and reused from buildings at the end of their life is dependent on how they were initially designed and constructed. This section discusses how decisions made at the design stage can enable disassembly and therefore can increase the quantity of the materials that can be salvaged and reused for subsequent life cycles in general building uses.

In design for disassembly and potential reuse, the following principles should be adopted [121]:

- The building should be built in layers that can be easily replaced as necessary throughout the life of the building. The components with the shortest lifetimes should be in the most easily accessible layers.
- The building complexity should be reduced as much as possible. Design using simple structural grids with clear support lines lead to use of regular-sized components which maximise their potential reuse with minimum variation. The amount of different materials and their different specifications should also be kept to a minimum to facilitate reuse.
- Work safety and space for machinery should be considered during construction and demounting. The design should also take account of future deconstruction logistics.
- Prefabricated components, or modules, that are installed on site are more easily disassembled for reuse in other locations or even on the same site.
- Connection details should be relatively simple and accessible. This also applies to the connections to the foundations and other components. Welding should be avoided except if the welded components can be reused in their entirety e.g. portal frame rafters.
- Fittings, fasteners, adhesives and sealants should be selected so as not to damage the secondary components, such as cladding and windows, during their removal as potentially reusable components.
- Design using reusable materials, and avoid complex composite materials, plaster, reinforced concrete etc that are difficult to separate and recycle. Hazardous materials should be avoided. Also, the effect of coatings and fire protection for steelwork should be considered in a reuse application.
- A building log/ data base should be prepared also in the form of a Building Information Model (BIM) that includes information on the design of the original building, the

specifications for materials and construction details of any refurbishment work, and also information relevant to dismantling.

9.3 Standardisation

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

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

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

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

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

1. The length between member splice points is limited by transportation generally by road. The usual stock lengths are 10, 12, 14, 15, 16, 18 and 20 m. Lengths of 12 m are generally transportable by lorry and lengths up to 18 m are possible depending on the local roads to the site. For containership transport, a total length less than 12 m is usually required;
2. The roof slope also depends on local snow and rain conditions and building practice in the region. A slope of at least 1:10 (6°) is normally specified for pitch roof portal frames; a slope of 1:20 (3°) is often used for pitch roof trusses as they are stiffer and the reduced slope due to their deflected shape is less than in portal frames;
3. Frame spacing is typically 5 to 8 m, depending on the span. The common dimensions for standardisation may be taken as 7.5 m for low snow regions and 5 or 6 m in high snow regions. In reuse scenarios, it is possible to vary the frame spacing depending on the new loading, which will require changes in the bracing and secondary structural system;
4. Typical spans and span-to-depth ratios for the primary roof members in single-storey buildings are given in Table 9.1 adapted from [122];

Table 9.1 **Typical spans and span to rafter depth ratios for single-storey buildings**

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

For buildings with eaves height between 6 m to 12 m, frame span to column depth ratios between 40 and 50 can be used for scheme design.

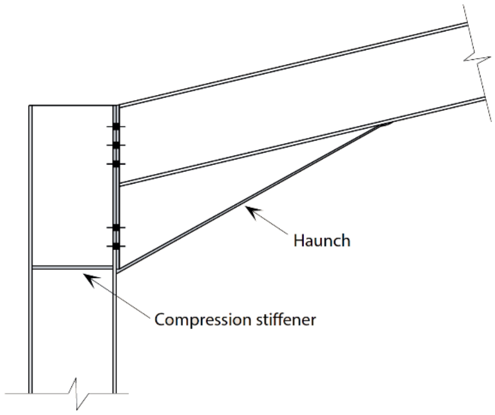
The following information on floor grids is provided that is based on national practices:

- Germany (according to DIN): planning grid of 100mm, typically multiples of 1200 mm;
- Netherlands: planning grid of 100mm, multiples of 1.8 m for schools, retail, hotels and office buildings;
- UK: planning grid of 300mm; typically, multiples of 0.6, 1.2 or 1.5 m. Schools and medical buildings are designed for multiples of 1,2 m and offices for a floor grid of 1.5 m;
- In general, multiples of 1.25 and 1.35 m are accepted in Europe (1.35 m may be preferred for standardisation of dimensions);
- Low rise office buildings (2-4 storeys): 6 to 9 m grids are often used to suit the use of precast concrete floor slabs on down-stand beams or possibly slim floor type beams.
- For high-rise office buildings in the UK, rectangular floor grids are preferred in which the longer span extends 13.5 m to 18 m across the building in steps of 1.5 m. A column grid of 16.5 m x 7.5 m is compatible with basement car parking;
- These long span beams generally have openings for services integration, such as cellular beams made from rolled sections, which have multiple circular openings;
- For high-rise buildings, a planning grid of 1.35 m leads to column spacing of 16.2 m x 8.1 m, which is a 2:1 grid that may be useful in planning of buildings for adaptability.

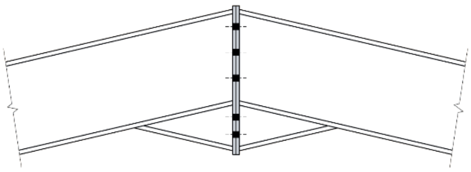
9.4 Best practice for analysis and design

9.4.1 Typical detailing for portal frames

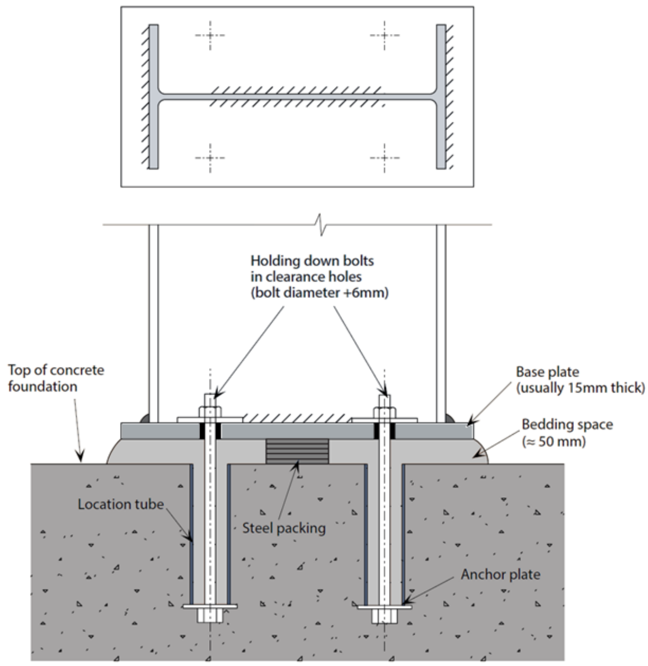
The typical detailing for a portal frame with nominally pinned bases is shown in Fig. 9.1.



Typical eaves connection (UK)



Typical apex connection (UK)



Typical nominally pinned base connection (UK)

Fig. 9.1 Typical detailing in a single bay portal frame with nominally pinned bases [118]

9.4.2 Bracing systems

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

- To transmit horizontal forces to the foundations;
- To provide stability during erection.

The bracing may be located at one or both ends of the building (Fig. 9.2), within the length of the building or in each portion between possible joints (where these are present). Braced bays (more common) or framed bays can be used for this purpose. Their position may also be influence by the layout of the building. Eave struts make sure that all portal frames are braced in the out of plane direction by the vertical bracing system.

Roof bracing is located in the plane of the roof, typically located on both ends of the building between the two first adjacent frames (Fig. 9.3). The primary functions of the roof bracing are:

- To transmit wind forces from the gable posts to the vertical bracing in the walls;
- To transmit any frictional forces from wind on the roof to the vertical bracing;

- To provide a stiff anchorage for the purlins that are used to restrain the rafters;
- To provide stability during erection;
- To restrain the tops of internal columns by bracing back to perimeter wall bracing.

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

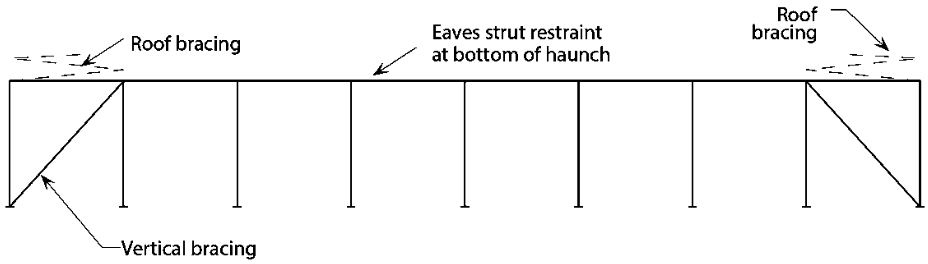


Fig. 9.2 Typical vertical bracing arrangement [41]

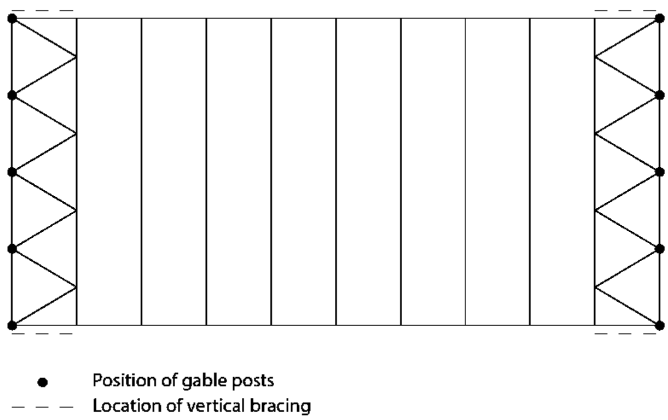


Fig. 9.3 Typical roof bracing arrangement [41]

9.4.3 Gable frames

The end gables are often designed using intermediate supports to the rafters, which are therefore lighter than the main structural frames. Another alternative is to use braced end gable frames, which may lead to some challenges for cladding performance due to for the differential displacements between subsequent frames. It is recommended that the end gables are the same size as the internal frames to facilitate their reuse and also to allow for future building extensions. This practice will also offer an improved performance of the cladding system as in theory, no significant differential displacement is expected between two adjacent frames.

9.4.4 Global analysis

Most portal frames in the UK are designed using plastic hinge analysis at the ultimate limit state but with additional checks on deflections using elastic design. To facilitate reuse of these structures, it is recommended to use elastic design for the first use and therefore for subsequent uses of the entire structure.

Member sizes in plastic design will be lighter than in global elastic design because of redistribution of moments from one part of the frame to another, but the additional cost increase is likely to be small given that the materials cost is less than half of the total erected cost.

9.4.5 Connection behaviour

Connections can be classified as nominally pinned, semi-rigid or rigid according to EN 1993-1-8. For scheme design purposes, it may be assumed that a nominally pinned connection following the typical detail presented in Fig. 9.1 offer 10% of the bending stiffness of the frame columns for a global stability analysis and 20% for serviceability checks. For nominally pinned connections, a nominal base stiffness of up to 20% of the stiffness of the column may be assumed [123].

For the final design, if a column is rigidly connected to a suitable foundation, the stiffness of the base connection should be taken as equal to the stiffness of the column for all ultimate limit state calculations. For SLS checks, the base may be treated as rigid [123]. For semi-rigidi connections, is recommended that the rotation spring stiffness of the connection is assessed according to the EN 1993-1-8 or by appropriate software.

Common software packages allow for the direct consideration of a rotational spring stiffness, which facilitate the implementation of the recommendations above. Particularly for base connections, if the software cannot accommodate such rotational spring, the base fixity may be modelled by a dummy member of equivalent stiffness as shown in Fig. 9.4.

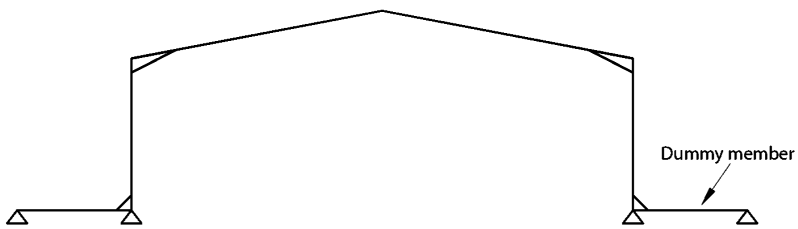


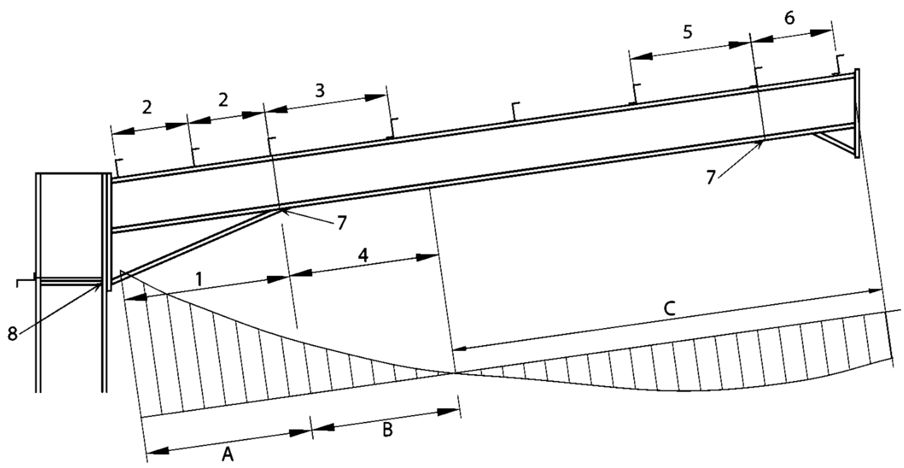
Fig. 9.4 Modelling base fixity by a dummy member [118]

The length of the dummy member is set as L = 0.75 L_{column}, and it is modelled with a pinned support at the extreme end. The second moment of area of the dummy elements can be set as a percentage of the second moment of area of columns to consider the desired stiffness on the base connections.

9.4.6 Member buckling design

Member buckling design should follow the procedures in EN 1993-1-1 section 6.3. It is recommended that structures designed for multiple assembling and disassembling processes are designed with partial factor $\gamma_{M1,mod} = K_{\gamma M1}\,\gamma_{M1}$ with $K_{\gamma M1} = 1.15$, which is in agreement with the recommendation for reclaimed steelwork proposed in section 8.2.3.

Secondary steelwork has an important role for an economic design of portal frames, to provide restraint to flexural, torsional and lateral torsional buckling of members. A typical portal frame design will rely on the minimum torsional restraints proposed in Fig. 9.5. For the wind uplift condition, additional torsional restraints may be necessary to the internal compressed flange of the rafter (Fig. 9.6) or for columns on a façade subjected to wind suction.



1. Tapered length, between torsional restraints
2. Tapered length, between lateral restraints
3. Length between lateral restraints
4. Length between torsional restraints
5. Length between restraints
6. Length between restraints
7. Torsional restraint to the rafter
8. Torsional restraint to the column

Fig. 9.5 Typical restraints arrangements on a portal frame: gravity load [118]

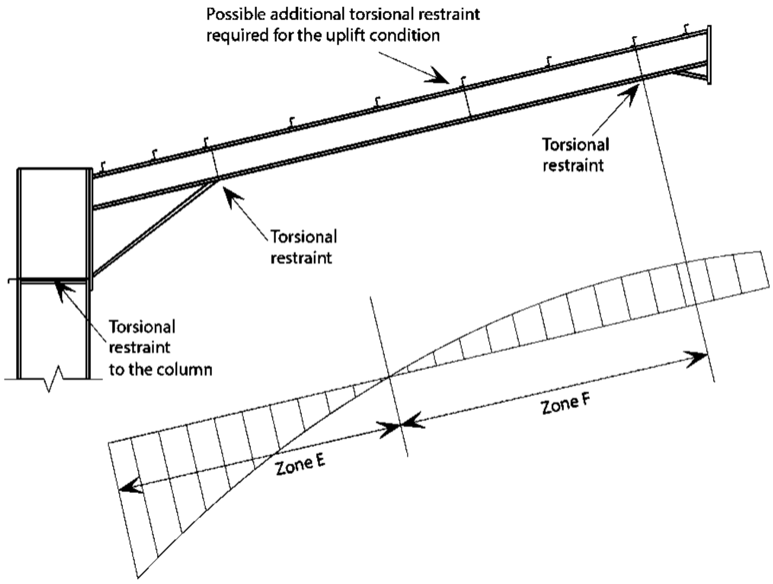


Fig. 9.6 Typical restraints arrangements on a portal frame: uplift [118]

9.4.7 Stress checks

There is no requirement to check service stresses according to EN 1993-1-1. However, since the deflection calculations are based on elastic analysis, plasticity should not occur at the SLS. It is recommended that a stress check is performed for the characteristic serviceability load combinations according to EN 1090.

9.4.8 Deflection checks

The deflections check criteria may be establish for a specific project or local practice. The recommendations from section 8.6.1 may be followed.

9.4.9 Trussed solutions

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

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

For long span claddings (such as built-up solutions of deep sandwich panels), the top chord of the truss may be assumed as restrained for the final stage. For uplift conditions, the bottom chord need to be restrained by a longitudinal roof bracing. For continuous roof trusses, bracings may also be necessary to restraint be compressed bottom chord close to the columns.

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

9.5 Durability

In common single storey buildings, metallic coating (hot-dip galvanized solutions) are less common in comparison with conventional paint coatings as they tend to be more expensive. When specifying paint coating systems for reusable buildings designers may wish to consider a high or very high durability class for the paint system according to ISO 12944-1 [100]. However, as a paint coating system tends to be weaker than a galvanized solution, the latter is preferable for structures with possible multiple assembling and disassembling cycles. Hot-dip galvanized solutions should follow ISO 1461 [124] and ISO 14713 [125] to [127].

9.6 Documentation, identification and traceability for reuse

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

9.6.1 Building memo

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

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

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

- identification of non-conformities, e.g. dimensional variations;
- reports of regular maintenance, changes and renovation work;
- pre-deconstruction audits if existing steelwork is being used;
- photo documentation.

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

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

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

9.6.2 Building information modelling

For achieving mainstream reuse, digital information has a key role in the process, as all relevant building data can be stored in a 3D digital model with Building Information Modelling (BIM) approach. The level of information that a 3D BIM model needs to accommodate is a responsibility of all project actors. The ISO standards EN ISO 19650‑1 [128] and EN ISO 19650‑2 [129] introduce the concept of level or information need (LOIN), for which is suggested that each project actor must define the relevant information to be stored for the purpose of the element on a specific project. The key concepts from a structural engineering point of view are proposed in Table 9.2 and Table 9.3.

Table 9.2 **Proposed information categories for the definition of the LOIN: general definition**

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

Table 9.3 **Proposed information categories for the definition of the LOIN: possible relevant data**

Category	Description
Context	For each life cycle: project description, site details, construction date, etc.
Project actors	For each life cycle: architects, engineers, contractors, etc.
Purpose	Features such as load bearing or non-structural, structural function (beam, column bracing), condition (permanent, temporary) etc.
Identification	For each life cycle: member identification number (ID), location (say floor number, bloc number), other relevant visual property; section serial size etc.;
Structural design	If the element belongs to a primary or secondary structural system (say according to EN 1998-1), ductility class according to EN 1998-1, fire rating, critical temperature, utilization factor and/or resistances, studs detailing, floor/element frequency/response factor/OS-RMS90, in-service deflections, project loading (loads on floors, wind action, snow load, etc.), type of connections (pinned, fixed or elastic – specify stiffness), maximum bending moment and shear forces on member and for connection design, tying forces etc.
Fabrication & erection	For each life cycle: fabrication company, fabrication date, standard for execution (say EN1090-2), execution class, fabrication records (project number), erection company, erection date, coating/galvanizing details (class, durability, thickens/mass)
Provenance & characteristics	New steel: producer, mill certificate number/ID, material product standard, delivery condition (EN 10204), steel grade, sub grade and Z quality, heat treatment delivery condition, geometry product standards, etc.; Reclaimed steel: stockholder, reference standard (say EN10025-2 or EN10219-1), grade and subgrade, relevant properties according to EN 1090-2 section 5.1 (measured/determined values and design values), stockholder internal documentation reference, product standards (say EN 10365 and EN 10034) etc.

The information proposed in Table 9.3 may be used as a reference to define the level of information stored for steel members part of a BIM model. References [130] to [132] may be used to help establishing the level of information need of the BIM model. Guidance form CWA 17316 [133] may be used to facilitate the exchange information.

9.6.3 Traceability of steel products for future reuse

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

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

Examples of permanent labels are laser-engraved plates with visible information or radio frequency identification (RFID) tags with information readable by RF scanner. Both methods should have unique identifier that can be linked to the digitally stored information; for example, the laser-engraved identifier can be QR code, barcode or just a simple identification code. Both methods can optionally contain the most essential information (such as CE marking) directly on the label. This can take a form of engraved table of essential characteristics or data stored on the memory chip connected to the RFID antenna. An example of a QR code for a reclaimed element it presented in Table 9.4.

Table 9.4 Proposed information to be stored on a permanent physical label

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

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

10 LOADING AND COMBINATION OF ACTIONS FOR NEW BUILDINGS

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

Snow load and wind loads are site specific (location, altitude) and are influenced by the geometry of the structure. These loads influence the reuse of a single-storey building with the same layout and frame spacing.

10.1 Characteristic values of actions

10.1.1 Loads on roofs

Permanent loads on roofs include, the self-weight of the cladding, and service loads, in addition to the self-weight of steelwork. Typical self-weights of roofing components are shown in Table 10.1. A minimum of 0.10 kN/m^2 allowance for the secondary elements is recommended. For service loads, a nominal value of 0.30 kN/m^2 should be allowed for lighting units, pipes for sprinkler systems, air-conditioning ducts and units, etc., but also to allow for solar panels.

The imposed loads on roofs are specified in Clause 6.3.4.2 (1) of EN 1991-1-1 and countries NAs. These loads are required for access for cleaning or maintenance only (category H). They should not be added to snow or wind loads. Values generally adopted in European countries are summarised in Table 10.3 for roof slopes not greater than 6°.

Table 10.1 **Self-weights of the roofing components**

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

Table 10.2 **Recommended design self-weights for different types of roof cladding**

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

Table 10.3 **Imposed loads for maintenance of roofs (category H)**

Country	q_k (kN/m^2)	Q_k (kN)
Czech Republic	0.75	1
Finland	0.40	1
France	0.80 (10 m^2)	1.5
Germany	—	1
Ireland	0.6	1
Italy	0.40	1
The Netherlands	1.00	1.5
Norway	0.75	1.5
Portugal	0.40	1
Romania	0.50	1
Slovakia	0.75	1
Spain	0.40	1
Sweden	0.40	1
United Kingdom	0.60	0.9

10.1.2 Loads on mezzanine floors

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

Imposed loads on floors are given in Clause 6.3.1.2 (1) of EN 1991-1-1 and countries NAs for office areas, and are summarised in Table 10.4. A value of 3 kN/m^2 is recommended as a standard value.

Table 10.4 **Imposed loads for offices and for mezzanine floors or attached office space**

Country	q_k (kN/m^2)	Q_k (kN)
Finland	2.50	2.00
France	2.50	4.00
Germany	2.00 (general office space)	2.00
Ireland	3.00	4.50
Italy	3.00 (also for public buildings)	2.00
The Netherlands	3.00	3.00
Norway	3.00	2.00
Portugal	3.00	4.00
Romania	2.50	4.50
Spain	3.00	4.00
Sweden	2.50	3.00
United Kingdom	2.50	2.70

10.1.3 Snow loads

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

- For persistent design situations:

$$s = \mu_i C_e C_t s_k \tag{10.1}$$

- For accidental design situations of exceptional snow load:

$$s = \mu_i C_e C_t C_{esl} s_k \tag{10.2}$$

- For accidental design situations of exceptional snow drift:

$$s = \mu_i s_k \tag{10.3}$$

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

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

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

Variations in snow load are mainly due to s_k. EN 1991 Part 1-3 divides Europe into nine different climatic regions, and defines zones to compute s_k as a function of the altitude. Four different snow classes are proposed. For each class, a recommended value of s_k is proposed to allow for reuse in the same and lower snow regions (Table 10.5 and Fig. 10.1).

10.1.4 Wind loads

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

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

$$v_b = c_{dir} c_{season} v_{b,0} \tag{10.4}$$

The value for $v_{b,0}$ is a national choice for a return period of 50 years. Table 10.7 presents the limits for this parameter and the average in various European countries [134]. Based on these values, a minimum European value for $v_{b,0}$ is proposed and, four different wind classes are defined (Table 10.6 and Fig. 10.2). Therefore, the basic velocity pressure for each European class $v_{b,class}$ can be obtained from Eq. (4.10) in EN 1991-1-4, as follows:

Table 10.5 **Proposed snow classes S1 to S4 for design of roofs**

Country	**s_k (kN/m^2)**			**Snow Class**
	Min. [a]	**Country average** [b]	**Min. European value**	
Finland	2.00	2.75	2.00	S1
Romania	1.50	2.00		
Norway	1.50	3.50		
Sweden	1.50	2.50		
Germany	0.45	0.85	1.00	S2
Italy	0.60	1.00		
United Kingdom	0.45	0.65	0.70	S3
France	0.45	0.65		
Ireland	0.40	0.55		
The Netherlands	0.70	0.70		
Portugal	0.10	0.30	0.40	S4
Spain	0.30	0.40		

[a] Assuming the average altitude for the less critical zone of the country
[b] Assuming the average altitude for the zone representing most area of the country

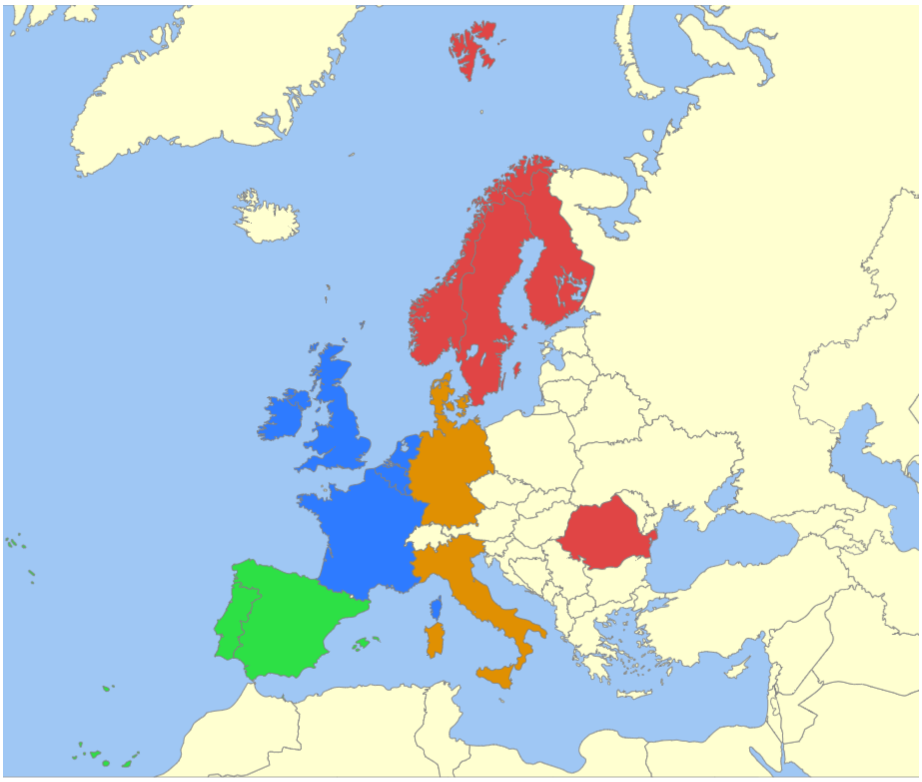


Fig. 10.1 Proposed design classes for the snow load map based on Table 10.5

$$q_b = \frac{1}{2} \rho v_b^2 \qquad (10.5)$$

where ρ is the air density and the proposed value is 1.25 kg/m³.

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

Table 10.6 **Basic wind velocity according to European countries: wind classes**

Country	**v_{b0} (m/s)**			**$q_{b0,mean}$ (kN/m²)**	**$v_{b0,class}$ (m/s)**	**$q_{b,class}$ (kN/m²)**	**Wind Class**
	Min.	**Max.**	**Average**				
Croatia	20	48	29	1.05	>28	1.05	W1
Cyprus	24	40	29	1.05	>28		
Greece	27	33	29	1.05	>28		
Romania	27	35	31 [a)]	1.20	>28		
Bulgaria	24	36	27	0.91	28	0.98	W2
Czech Republic	23	36	27	0.91	28		
Italy	25	31	27 [a)]	0.91	28		
The Netherlands	25	30	27 [a)]	0.91	28		
Portugal	27	30	27 [a)]	0.91	28		
Russia	20	44	27	0.91	28		
Spain	26	29	27 [a)]	0.91	28		
Belgium	23	26	24	0.72	26	0.85	W3
Denmark	24	27	25	0.78	26		
France	22	28	24 [a)]	0.72	26		
Germany	23	30	25 [a)]	0.78	26		
Ireland	25	28	26	0.85	26		
Lithuania	24	32	26	0.85	26		
Luxemburg	24	24	24	0.72	26		
Norway	22	31	25	0.78	26		
Slovakia	24	26	24	0.72	26		
United Kingdom	22	32	25 [a)]	0.78	26		
Ukraine	24	31	26	0.85	26		
Austria	18	28	21	0.55	23	0.66	W4
Belarus	22	24	22	0.61	23		
Estonia	21	21	21	0.55	23		
Finland	21	26	22 [a)]	0.61	23		
Hungary	24	24	23	0.66	23		
Latvia	21	27	23	0.66	23		
Poland	22	26	23	0.66	23		
Slovenia	20	30	23	0.66	23		
Sweden	21	26	22	0.61	23		
Switzerland	20	24	21	0.55	23		

[a)] Usual value from the NA/local standard

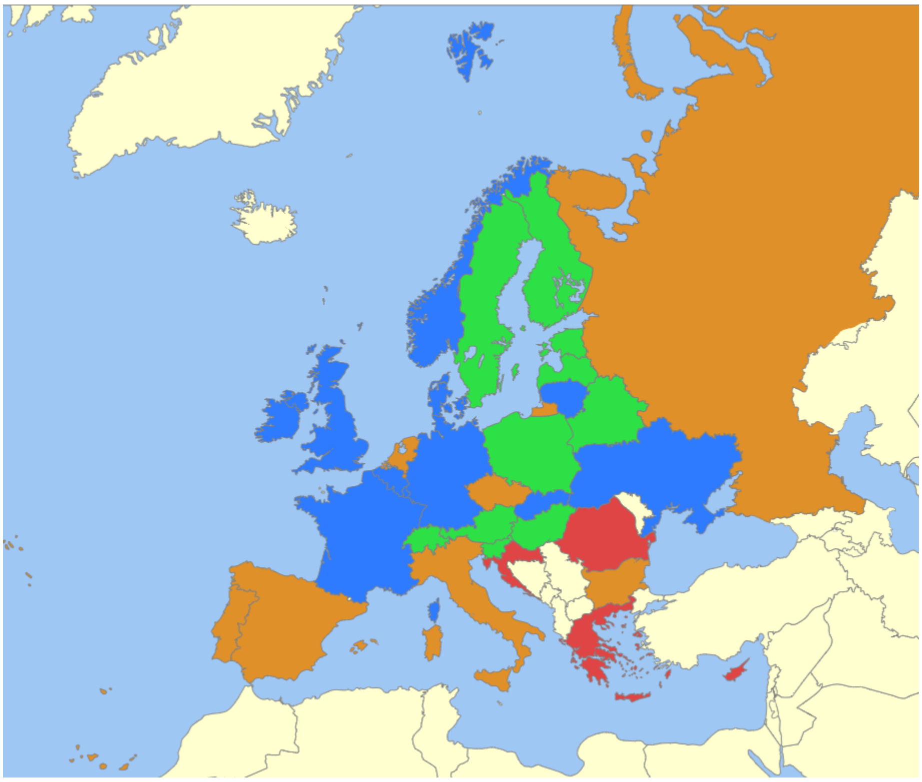


Fig. 10.2 Proposed design classes for the wind load map based on Table 10.6

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

$$q_{\mathrm{p}}(z) = [1 + 7 \cdot I_{\mathrm{v}}(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_{\mathrm{m}}(z)^2 = c_{\mathrm{e}}(z) \cdot q_{\mathrm{b}} \tag{10.6}$$

where:

$I_{\mathrm{v}}(z)$ – Turbulence intensity at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity,

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

$$v_m(z) = c_{\mathrm{r}}(z) \cdot c_0(z) \cdot v_{\mathrm{b}}, \tag{10.7}$$

$c_{\mathrm{r}}(z)$ – Is the roughness factor,

$c_0(z)$ – Is the orography factor, taken as 1,0 for the cases where orography (eg hills or cliffs) increases wind velocities less than 5% or when the average slope of the upwind terrain is less than 3°. The upwind terrain may be considered up to a distance of 10 times the height of the isolated. orographic feature. the load effects. The orography effects can be calculated according to EN1991-1-4 Annex A.3. National annexes may impose modification to this procedure.

Orography and roughness factors are too dependent of the building location to allow for specific recommendations for reuse. However, as a rule of thumb, it is recommended to keep the ratio between basic velocity pressure peak velocity pressure at 15 m of: $q_{\mathrm{p}}(15)/q_{\mathrm{b}} \geq 1.15$ while using the wind load classes proposed in Table 10.6 and Fig. 10.2.

10.1.5 Guidance for the use of wind and snow classes

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

- Define snow and wind loads according to the building location, which will need to consider nationally defined parameters,
- Compare the snow and wind loads according to the building location with the defined values for the different design class proposed in Table 10.5 and Table 10.6. Designers may wish to respect the proposed country average values or the European load class values,
- Engineering judgement is required to assess the costs of increasing the design loads to the ones proposed in Table 10.5 and Table 10.6.

The engineering judgment must consider the final outcome of the design process, and not a simple comparison between the load according to the building location and the proposed values in Table 10.5 and Table 10.6. On practical design scenarios, the available section sizes may lead to a unoptimized utilization factor for the structural members, which can allow for an increase of the design loads without increasing the solution costs. Designers may wish to document the ULS capacity of the structure, i.e., which characteristic snow and wind loads are admissible for the structure.

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

10.1.6 Adapting characteristic load values based on return period

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

Snow Load

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

$$S_{\mathrm{n}} = S_{\mathrm{k}} \left\{ \frac{1 - V \frac{\sqrt{6}}{\pi} ln\left(-ln(1 - P_n)\right) + 0.57722}{(1 + 2.5923\, V_{\mathrm{x}})} \right\} \quad (10.8)$$

where:

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

S_n – Is the ground snow load with a return period of n years;

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

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

Values for $S_{\mathrm{n}}/S_{\mathrm{k}}$ for different coefficient of variation are given in Table 10.7.

Table 10.7 **Adjustment of snow load according to the return period (EN1991-1-3)**

Return period	$S_{\mathrm{n}}/S_{\mathrm{k}}$					
	***V*x=0.20**	***V*x =0.30**	***V*x =0.40**	***V*x =0.50**	***V*x =0.60**	***V*x =0.70**
15	0.87	0.84	0.81	0.79	0.78	0.76
30	0.95	0.93	0.92	0.91	0.91	0.90
50	1.00	1.00	1.00	1.00	1.00	1.00
75	1.04	1.05	1.06	1.07	1.07	1.08
100	1.07	1.09	1.11	1.12	1.13	1.14
125	1.09	1.12	1.14	1.16	1.17	1.18

The coefficient of variation for the snow load may vary between 0.30 and 1.15. Values between 0.30 and 0.70 are suggested as lower boundary and mean value, respectively [135]. The value may be defined for specific countries by national annexes or other valid references.

Wind Load

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

$$c_{\mathrm{prob}} = \left(\frac{1 - K \cdot \ln\left(-\ln(1-p)\right)}{1 - K\, ln(-\ln\,(0.98))}\right)^{n} \tag{10.9}$$

where:

K – is the shape parameter depending on the coefficient of variation of the extreme-value distribution;

n – is the exponent.

According to the core Eurocode, the recommended values are 0,2 for *K* and 0,5 for *n*. The probability p can be obtained based on the return period, i.e., for a return period of 50 years, *p* = 1/50=0.02, leading to $c_{prob} \approx 1.00$. The values of c_{prob} for different return periods can be found in Table 10.8.

Table 10.8 **Adjustment of wind load according to the return period (EN1991-1-4)**

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

Imposed loads

As shown in the previous sections, the adaptation for snow, wind and thermal actions is discussed in the relevant parts of EN 1991 by means of a probability factor (c_{prob}). For imposed actions on floors, the characteristic values of the variable action may be adapted according to the procedure proposed by the Dutch National Annex to EN 1990, cl. A1.1 (2) [136] as follows:

$$q_{\mathrm{t}} = q_{\mathrm{t0}}\left(1 + \frac{1-\psi_0}{9}\ln\frac{t}{t_0}\right) \tag{10.10}$$

Where:

q_t – Is the adapted characteristic value of the variable action for the design working life;
q_{t0} – Is the characteristic value of the variable action for a design working life of 50 years;
t – Is the target design working life;
t_0 – Is the standard design working life of 50 years.

Table 10.9 **Adjustment of imposed load on floors according to the return period**

q_t/q_{t0}						
ψ_0	**Return Period**					
	15	**30**	**50**	**75**	**100**	**125**
0.70	0.96	0.98	1.00	1.01	1.02	1.03
1.00	1.00	1.00	1.00	1.00	1.00	1.00

10.1.7 Thermal action

A change in the temperature of a steel structure causes a thermal strain in the steel elements. The magnitude of the thermal strain is equal to the coefficient of thermal expansion which is stated in BS EN 1993-1-1 as α = 12 × 10-6 per °C for temperatures less than or equal to 100°C multiplied by the temperature rise. This corresponds to 1.2 mm expansion per degree temperature rise per 100 m of building. The result of the thermal strain can be free expansion of the element if there is no restraint, or if the expansion is fully restrained, an axial stress is induced.

Portal frame buildings are usually provided with vertical bracing in the side walls to achieve lateral stability in the longitudinal direction. If bracing is provided at each end of the building, axial forces will be developed on the thermal expansion of structural elements which are continuous between the braced bays. The magnitude of the axial force depends on the difference in temperature between that at completion of the structure and the temperature at the time in question and the stiffness of the restraint system.

In practice, axial stresses may not be realised because of slip at bolted connections or elastic buckling of secondary elements to relieve the axial load. Continuous longitudinal elements such as crane runway beams, crane rails, valley girders and eaves beams should be considered carefully and designed for axial loads due to thermal actions. Guidance provided by EN 1991-1-5 should be followed to define the thermal action. The magnitude of the axial loads depends on the stiffness of the restraint. Substantial elements such as crane runway beams may potentially deliver large forces due to thermal expansion.

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

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

10.2 Combinations of actions

Combinations of actions for a given limit state are presented in Clause 6.4.3.2 of EN 1990 for different design situations: persistent, i.e. final use of the structure, transient, accidental, and seismic design cases. This section focuses on the first design use and therefore only the fundamental combinations are considered. For the strength (STR) limit state, the combinations of actions can be obtained from Eq. 6.10, or Eqs. 6.10a and 6.10b in EN 1090, see also Table 8.1 in this document.

Design for reuse my specify a above normal design life for the structure. For a working life of say 100 years, it is recommended to increase the reliability requirement for the design (design working life category 5 in EN 1090 – *higher than normal* degree of reliability). Typically, single-storey buildings are designed as low occupancy buildings with a medium consequence of failure (CC2), and for a working life of 50 years. The reliability level corresponds to a target reliability index β of 4.7 for 1-year reference period and 3.8 for 50 years (RC2).

Assuming a working life of 100 years, the design verification can be based on a higher index β, say that corresponding to a CC3 structure, and therefore RC3. The reliability index for RC3 is 5.2 for 1-year reference period, and 4.3 for a period of 50 years. By using the reliability differentiation rules adopted in EN 1990, the partial factors for unfavourable actions in the fundamental combinations can multiplied by K_{FI} = 1.1 (see section 8.2.1). Another option to achieve the required target reliability is to increases the supervision (including maintenance) and execution inspection requirements, which is commonly applied for steel structures (say with a higher execution class – See Annex C of EN 1993-1-1 and Annex B of EN 1990).

For a proposed working life of say 100 years, if K_{FI} = 1.1 is used, it is recommended that the fundamental combinations of actions are assessed based on Eq. (6.10) from Clause 6.4.2.2(3) of EN 1990. While applying $K_{FI,}$ the reliability levels are achieved with the implementation normal supervision and execution inspection requirements for a typical RC2/CC2/EXC2 structure. Eqs. (6.10a) and (6.10b) give lower reliability levels than those obtained from Eq. (6.10), but still in most cases are above those desired. The fundamental load combinations are then obtained from the following general expression, in which "+" implies "to be combined with":

$$\underbrace{\sum_{j\geq 1} 1.5G_{k,j}}_{\substack{\text{Unfavourable}\\ \text{permanent actions}}} \text{"+"} \underbrace{\sum_{h\geq 1} G_{k,h}}_{\substack{\text{Favourable}\\ \text{permanent actions}}} \text{"+"} 1.65Q_{k,1} \text{"+"} \sum_{i>1} 1.65\psi_{0,i}Q_{k,i} \qquad (10.11)$$

EN 1993-1-1 Annex C (5) refers that a specification of a higher execution class for the execution of a structure should not be used to justify the use of lower partial factors for resistance in the design of that structure (see section 6.4.3). For a possible future reuse scenario, the use of K_{FI} may be beneficial as the higher combination factors assumed for the initial design may allow to accommodate differences between national defined design parameters or design action for a specific location, allowing to consider the standard reliability requirements for a CC2 structure with a design working life of 50 years.

11 REUSE THROUGH DESIGN AND BETTER CONSTRUCTION DETAILS

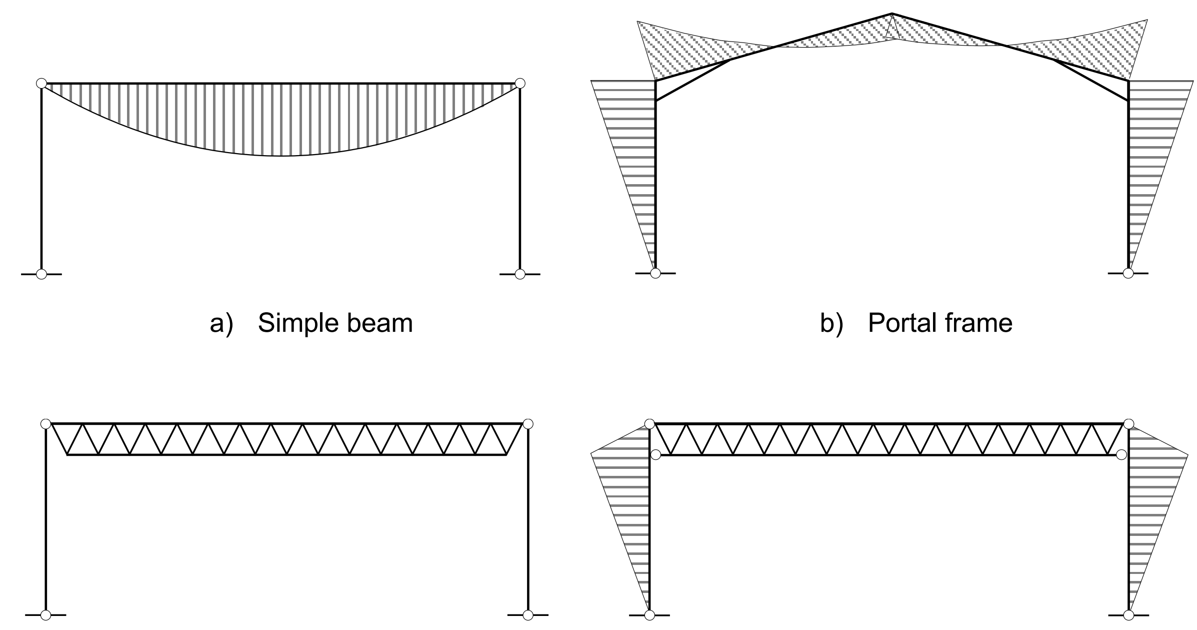
Three single-storey building forms are identified for the study of the opportunities to facilitate reuse through design. They comprise rafters (simple or continuous hot-rolled beams, or a lattice structures) and columns. The beams, columns, joints and column bases are the structural elements of the building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

11.1 Structural design

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

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

The basic structural systems are presented in simple terms in Fig 11.1.



a) Simple beam

b) Portal frame

c) Truss

d) Portal truss

Fig. 11.1 Structural concepts for primary steelwork and bending moment diagrams from global elastic analysis (after [66])

11.2 Standardisation in single storey structures

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

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

11.2.1 Opportunities for reuse in portal frame structures

A conventional portal frame system offers the possibility of reuse of its individual components, as most of the primary members without modification are long with have typical span to depth ratio of 40 to 50 for columns and 50 to 65 for rafters (identified in green in Fig. 11.2). The lengths in green may be separated from the more specialist critical zones (identified in red in this figure) by cutting to obtaining beam and column segments.

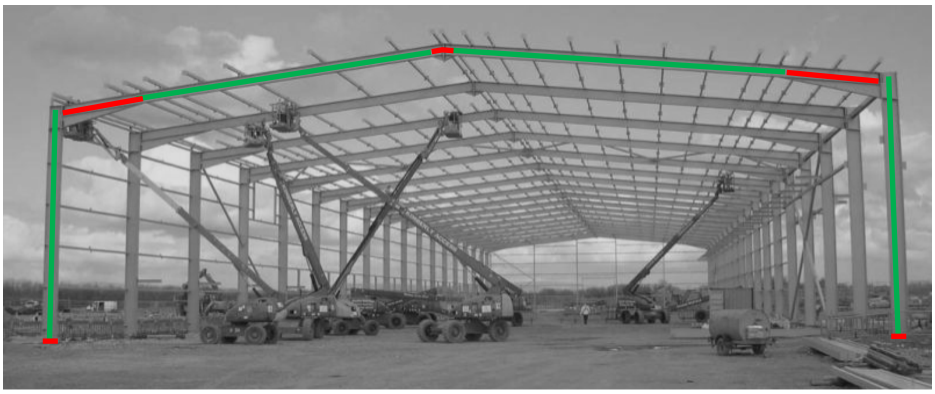


Fig. 11.2 Opportunities for reuse in portal frame: segments with minor modifications

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

- Span increments of 3m. Typical spans are 30 m, 36 m and 42 m using rolled sections;
- Roof slope of 6^o to the horizontal;
- Frame spacing of 6 or 7.5 m with 7.5 m being preferred for purlin and side rail systems;
- Columns with a height to the top of the column of 7.5 m as a standard (6 m may be used for portal frame spans less than 30 m and 7.5 m for longer spans). The height to the underside of the haunch may be up to 1 m less than the column height;
- Design the columns for the additional load from a mezzanine floor on a 7.5 m square grid with the floor level at 4 m above the ground floor slab, which would require 7.5 m long columns; the square grid approach ensures that the columns can be used in a possible multi-bay future application;
- Haunch length of 10 to 12% of the span and of a depth equal to twice that of the rafters, being 10% recommended as the standard dimension;
- End gables should be the same as the internal frames to facilitate building expansion,
- Column bases with 4 bolts that may be treated as nominally pinned at the ultimate limit state but which may offer some rotational stiffness for sway deflection calculations,
- Bracing in the form of circular or square hollow sections with a typical range of cross section sizes (diameter/width) between 130 mm and 200 mm diameter with lengths between 3 m up to (but excluding) 12 m length between the frames (at 7.5m spacing); avoid using “x” bracing arrangements; it’s preferable to use few robust members that can be reclaimed without modifications;

11.2.2 Standard portal frame with welded eaves and apex segments

Using the guidance presented in the previous section, a portal frame may be composed of standardised components to facilitate the re-use of the beams and columns either in a similar portal frame or in general building construction. These components are shown in Fig. 11.3 and are:

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

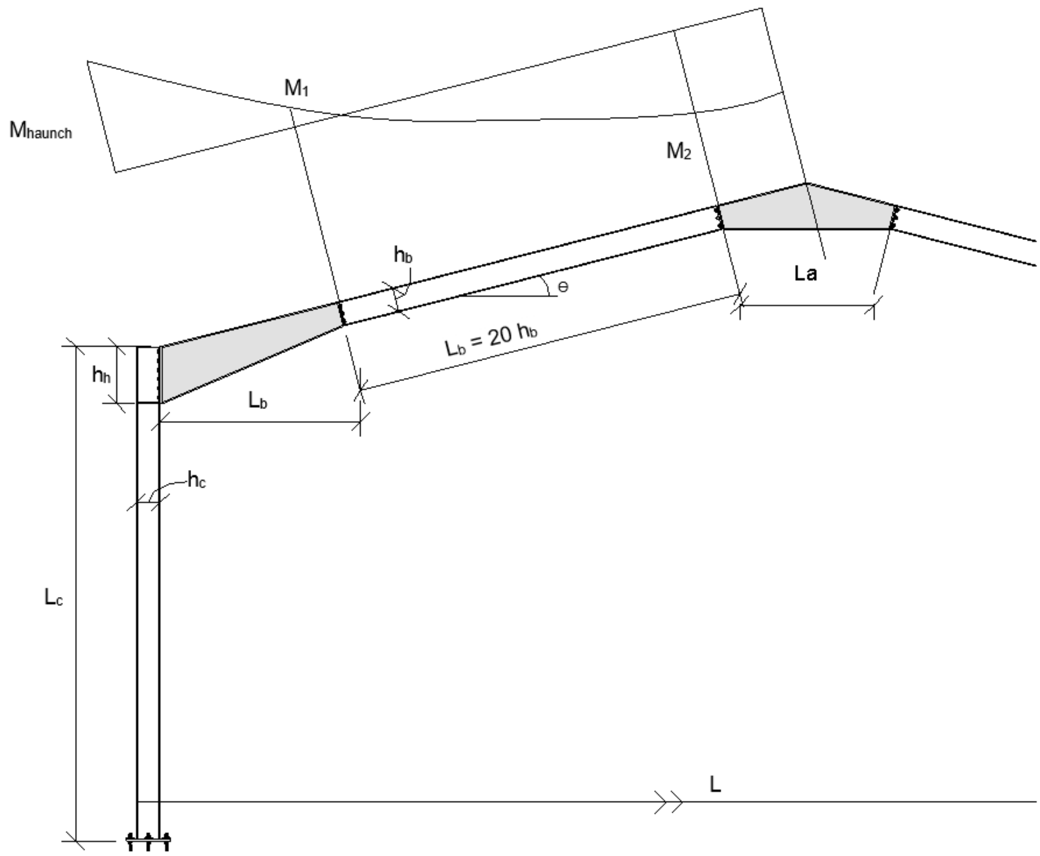


Fig. 11.3 Re-usable components in a portal frame [41]

The overall span of the portal frame is given by:

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

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

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

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

11.2.3 Portal frame rafter with bolted haunch

Other opportunities to increase the reuse components of a portal frame are shown in Fig. 11.4, which shows a bolted haunch detail. In this case, friction grip bolts are used between the separate haunch section and the inclined rafter. A similar example of this haunch segment is shown in Fig. 11.5 with a fabricated profile with tapered section. The same system may be used for the apex connection in Fig. 11.6.

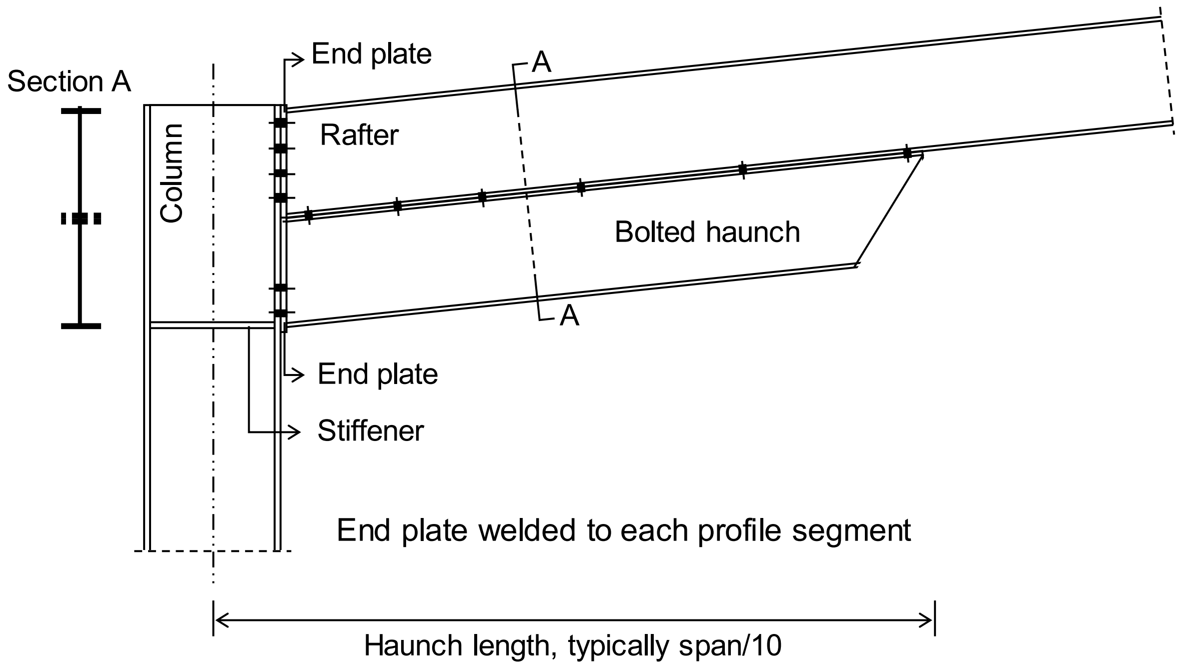


Fig. 11.4 Bolted haunch components in a portal frame [41]

Both bolted haunch and rafter have an individual welded end plate, which allow the connection behaviour to be similar to common eaves portal frame connection.

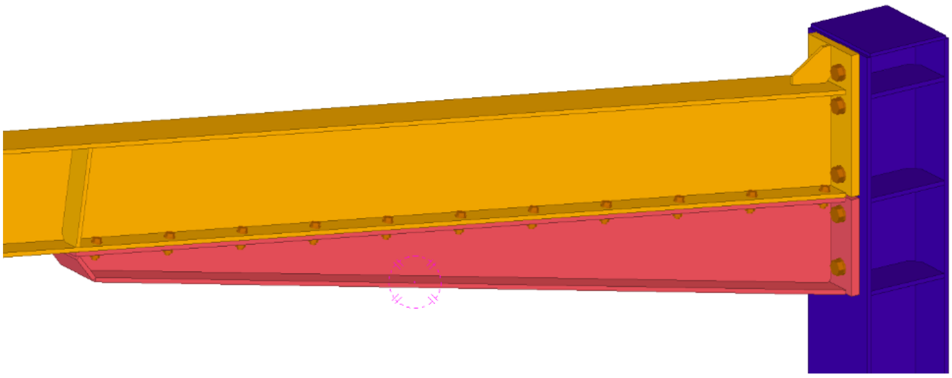


Fig. 11.5 Bolted tapered haunch segment in a portal frame

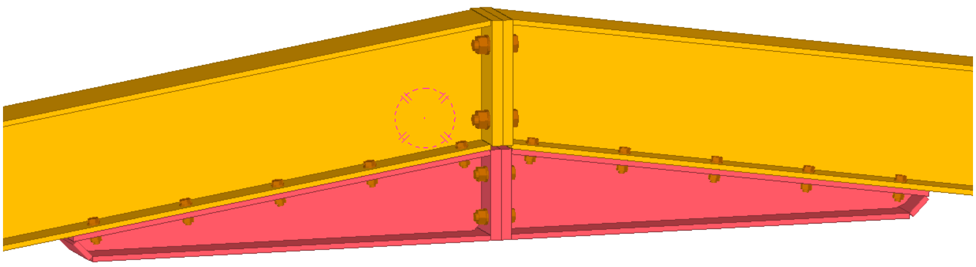


Fig. 11.6 Bolted apex components in a portal frame

11.2.4 Portal frame with corner strut

For short and medium span portal frames, the haunch detail can be replaced by a strut using a Square Hollow Section with 4 bolts connected to the rafter and column flange (Fig. 11.7). These bolts act in shear and tension depending on the moment direction. The inclined strut is at an angle of 45+3 = 48° to the horizontal so that the ends of the strut have the detail at the same angle to the member axis. The strut will generally be a 150x150 SHS member. The strut will be located approximately 1.5m below the axis of the member connection so it may interfere with the use of the space. Conversely, it will not be as long as a conventional haunch so is less effective in reducing the moment in the rafter. The web of the column and rafter would have to be stiffened locally by a half web stiffener.

As a guide, the axial force in the inclined strut will be up to 200 kN based on the resistance of 4 bolts and the end plate to the strut, which will lead to a bending moment of about 300 kNm at the haunch. To be able to reuse the full length of the rafter, a small segment of the same size as the rafter may be used to which the rafter is bolted by an end plate. This means that the end details of the rafter are compatible with their use in general applications and the fabricated segments can be discarded in the second use if the complete portal frame is not reused.

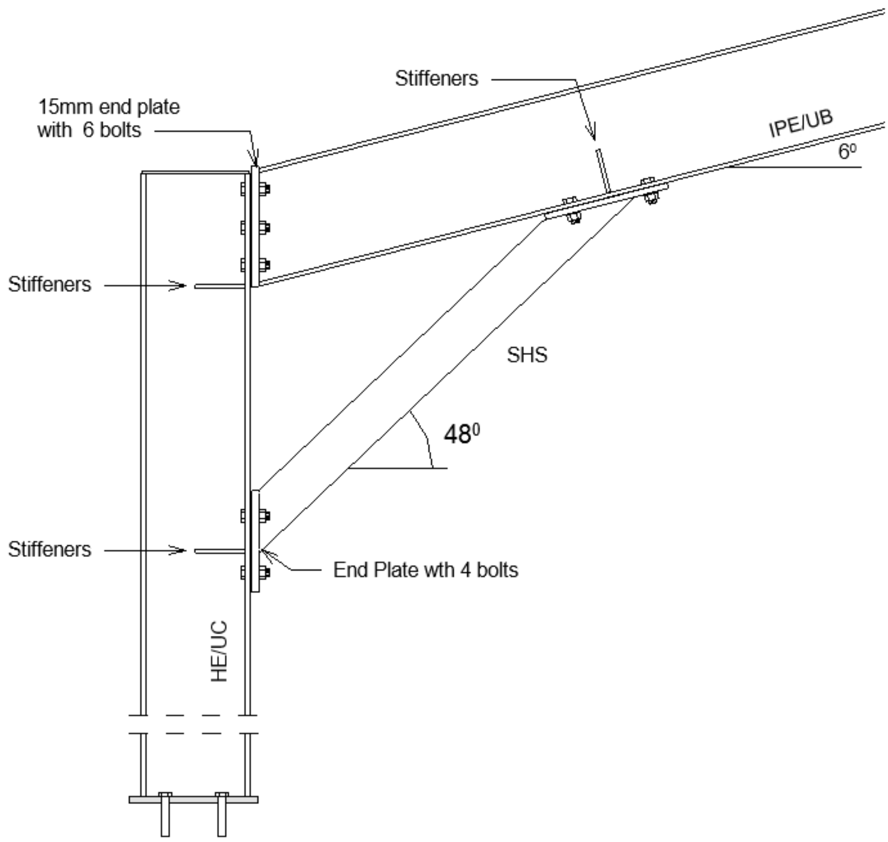


Fig. 11.7 Strut-type haunch in a portal frame [41]

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

11.3 Column connection to foundations

There are three generic forms of connections to the foundations that may be considered depending on the scale of the structure;

- Pinned connections using bolted angles for relatively simple and short span portal frames. An example is shown in Fig. 11.8 (a); to facilitate grouting operations, the proposed solution can be bolted to a based plate which can provide a constant soffit;
- End plate connections that have some bending stiffness but are normally treated as pinned unless the end plate extends outside the column. The use of additional welded stiffeners is not recommended for reuse of the columns (Fig. 9.1).
- A column 'shoe', which is bolted to the column by friction bolts, as shown in Fig.11.8 (b) and may transfer a high moment to the foundation. The column shoe is prefabricated for a particular column size. It may also be combined with use of a bolted connection for inclined bracing as also shown.

a) Pinned b) Fixed

Fig. 11.8 Examples of demountable base connections to columns

It is also recognised that the column bases should be accessible to enable them to be demounted easily without damage and without the need for major demolition works. An example of a detail that achieves this accessibility is shown in Fig. 11.9.

Fig. 11.9 Example of accessible base connection [137]

11.4 Trussed solutions for reuse

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

A typical truss configuration is shown in Fig 11.10. It is usually manufactured in two pieces that are connected with sleeve joints in the ridge of the truss and in the middle of the bottom chord. These connections acting tension or compression depending on the moment divided by the truss height. The connection to the column can be either pinned or fixed.

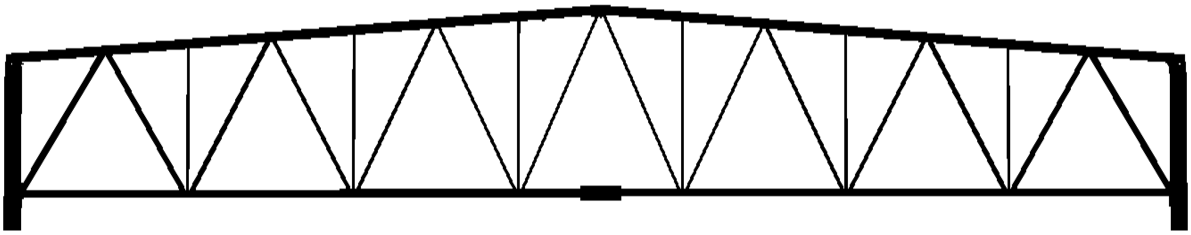


Fig. 11.10 Typical truss configuration with roof slope of 1:20 (~3°) [138]

The principles that guide the reuse of truss systems are:

- Trusses should be considered for spans exceeding 30m and are preferred where additional loads are applied to the truss including suspended services or walkways.
- Trusses should have a span: depth ratio of about 20 and so a truss may be 2m deep at its apex for 40m span. Therefore, a span: depth ratio of 20 may be adopted as a standard for reuse and this may be expected to be very stiff in bending.
- Because of their higher fabrication costs, trusses should ideally be placed at 7.5m or even 9m apart depending on the form of the secondary elements.
- A truss should be fabricated from Structural Hollow Sections (SHS) in which the chords and bracing members are of the same width so that high axial forces can be transferred without bending of the face of the SHS.
- For trusses more than 50m span, it may not be possible to deliver the trusses in two equal segments and so intermediate splices may be required. These splices should be at the quarter span positions.
- The width of the truss chords is normally chosen so that they are stable in compression under their own weight when being lifted at two points at an assumed angle of 45°. For a 20m long truss segment with lifting points at 12m apart, the slenderness in the transverse direction should ideally not exceed 200, in which case the minimum width would be 150mm (i.e. 150x150 SHS or 150x100 RHS).
- The top chord is relatively stable in compression when restrained by the roof purlins but the bottom chord would generally have to be stabilised at mid-span in the case of wind uplift on the roof. This can be achieved by an inclined strut to a beam at ridge level.
- Trusses can be designed to transfer high moments at the columns by 'push-pull' action and so structures using trusses are efficient in terms of their resistance to horizontal

forces, provided that that bottom chord is stabilised by in-plane bracing in the transverse direction.

One option to improve re-usability of welded steel trusses is to change the typical truss configuration so that the truss would be always delivered in two pieces having uniform height with parallel top and bottom chords as shown in Fig 11.11. The system should enable installation of roof truss into different slopes and thus enabling small range of variation in truss spans. The truss halves are connected in the middle with project specific connector parts to accommodate different roof slopes in the range of 3 to 10^{o}. The connectors are designed with friction grip bolts acting in shear and they are shown in Fig 11.12.

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

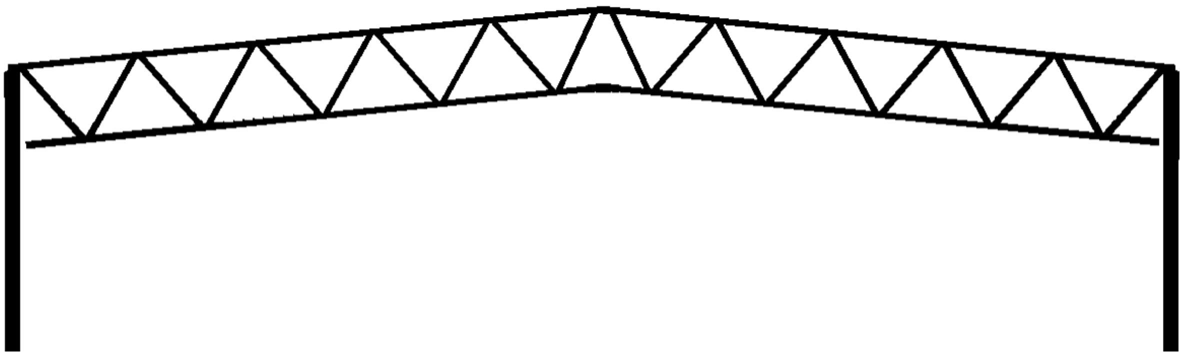


Fig. 11.11 Steel truss system for better re-usability [138]

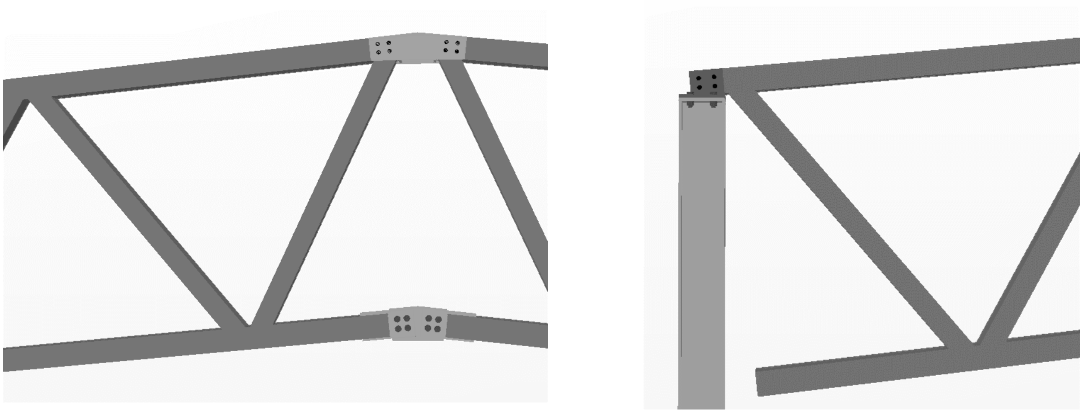


Fig. 11.12 Connectors enabling truss assembly to different slopes.[138]

11.5 Braced box type structures

Braced box type of structures offer good opportunities for reuse as the structural components can be easily standardised. All connections between members can be pinned, which require simple detailing and a minimum number of bolts. The solution requires a bracing system in the longitudinal building direction and a roof truss bracing supported by the braced gable frames which provide in plane stability of the frames (Fig 11.13). This concept is widely used in temporary structures and may be considered for single storey buildings with short spans.

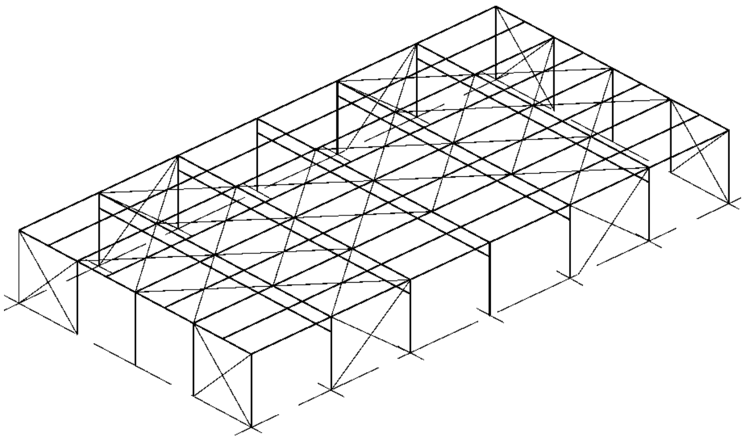
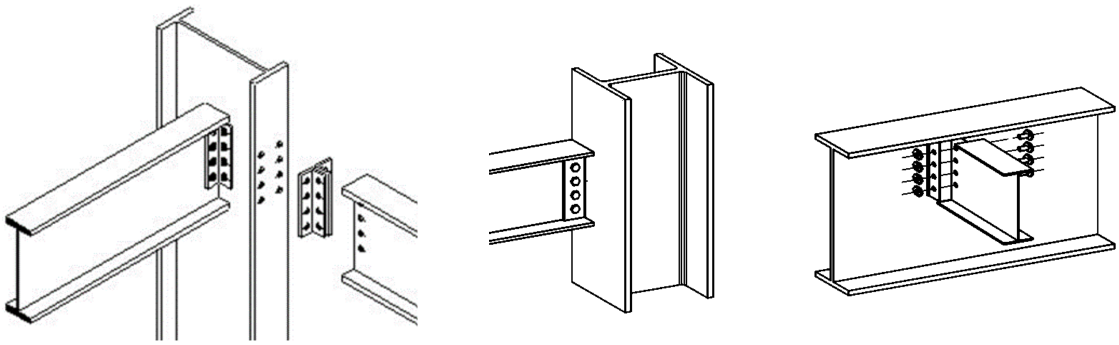


Fig. 11.13 Examples of braced box type structures [41]

11.6 Mezzanines

It is recommended that mezzanine floors rely on a column grid with multiples of 1.5 m, and typically 6 m. It is proposed to use a value of 4m as a standard height above the ground floor for a portal frame with 7.5 m eaves height. Mezzanines should use a grid of pin-ended beams with cleated connection (Fig. 11.14a). Members with equal cross section should have the same detailing. While connecting beams to the minor axis of a column, cleated connections may not be possible. For such cases, a fin plate detail can be used (Fig. 11.14b). The general recommendation is that welding on floor beams shall not be used. If hollow section columns are used, Hollo Bolt of Blind Bolt solutions are recommended (Fig. 11.15).



a) Cleated connection with angles

b) Fin plate connections

Fig. 11.14 Recommended connections for mezzanine floor beams

a) Hollo-Bolt

b) Blind-bolt

Fig. 11.15 Hollo-Bolt and Bild Bolt solutions [112]

An alternative to the cleated and fin plate connection is the system presented in Fig. 11.16, which can speed up the erection and deconstruction processes. Guidance on this concept can be found in reference [139].

Fig. 11.16 Quicon connections [140]

One of the critical details that hinders the reusability of mezzanine floor is the use of a permanent attachment between the floor plate and floor beams, such as the traditional solution with composite floor with welded studs. To increase the reusability of floor beams, it is encouraged the use of detachable floor solutions as those presented in Fig. 11.17 to Fig. 11.20. The demountable composite floor solution proposed in Fig. 11.20 offers the benefit of demountability together with higher stiffness and resistance provided by the composite action between the steel beam and the concrete topping over the steel sheet. Guidance about the analysis and design of such systems can be found in reference [141].

Fig. 11.17 Demountable floor system using precast units and floor bracing [137]


Fig. 11.18 Demountable floor system using cross laminated timber (CLT) [142]

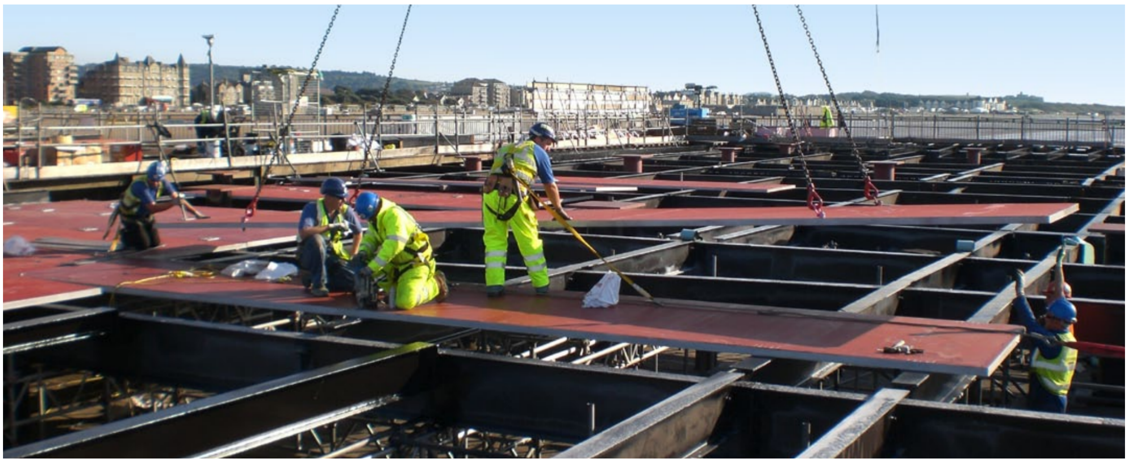


Fig. 11.19 Demountable floor system SPS panels – two steel plates with polymer core [143]

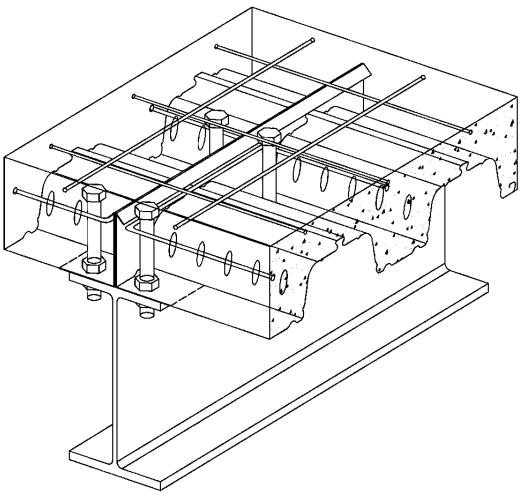
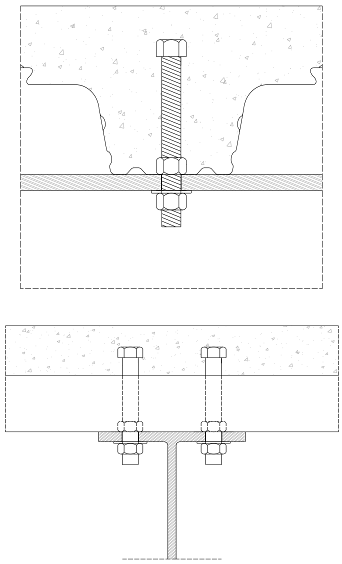



Fig. 11.20 Demountable composite floor system with a cast-in edge trim to form a cut line to be able to reuse slab segments [141]

11.7 Secondary steelwork and cladding

11.7.1 General

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

11.7.2 Secondary steelwork

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

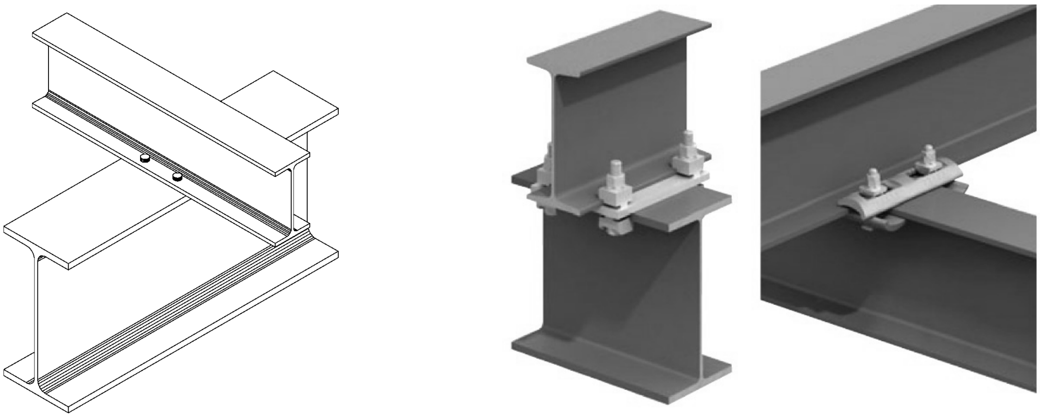
In many countries, cassette systems are used as an alternative to C section side rails, as they support the external cladding and insulation in the cassette tray. Cassettes are more efficient for column spacings up to 6m.

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

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

11.7.3 Alternative systems for secondary steelwork

The common solution for secondary steelwork relies on cold formed purlins and side rails that typically provide an economic solution for a spacing of 1.8 m to 2 m between members. A possible measure to reduce the number of connections between the building layers would be to rely on purlin/side rails spacing between say 3.5 m to 4 m. This will require stronger and stiffer purlins, for which hollow section (typically rectangular) or hot rolled section may be used. The benefit of continuity for the secondary steelwork may be achieved by using the well-known *Gerber* system, which used simple connection in continuous elements located at the points of expected counter flexure (bending moment equal to zero). For open section, the system would not require welding of auxiliary steelwork to the purlins/side rails. For open section, cleats and other fittings are not required as the secondary steelwork can be directly bolted to the primary structure (Fig. 11.21a). As an alternative, clamped solution may be explored, as they avoid drilling holes in the steel elements (Fig. 11.21b)



a) Bolted connection between hot rolled purlin and primary structure

b) Clamped connection between hot rolled purlin and primary structure [144]

Fig. 11.21 Connections between hot-rolled purlins and primary steelwork

11.7.4 Cladding

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

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

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

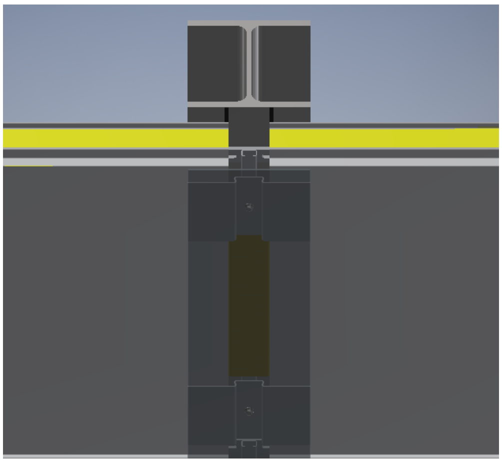
As noted above, the use long-span cladding is encouraged, as no purlins/side rails are needed. With the reduction of a building layer, a considerable reduction in the number of connections would be achieved, which may allow for an increase reusability of the cladding system.

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

A clear area of interest is to investigate new long span roof and wall panelised solutions (spans of 6-10m). If an appropriate connection system is developed (bolted or clamped with clear disassembly points), these elements could be easily disassembled and reused (see section 11.7.5).

The most common practice for long span cladding panels (of 6 to 8 m span) rely on deep trapezoidal sheeting (Fig. 11.23) or deep sandwich panels in roofs (Fig. 11.24, Fig. 11.25) and horizontally installed sandwich panels on walls spanning between frames (Fig. 11.22). Such solutions are a common practice in Finland and Sweden, where the thermal insulation requirements usually demand thick panels that are consequently structurally strong, enabling horizontal installation between primary structural elements without the need of purlins.

A possible solution for wall sandwich panels with clamped connections is shown in Fig. 11.22.



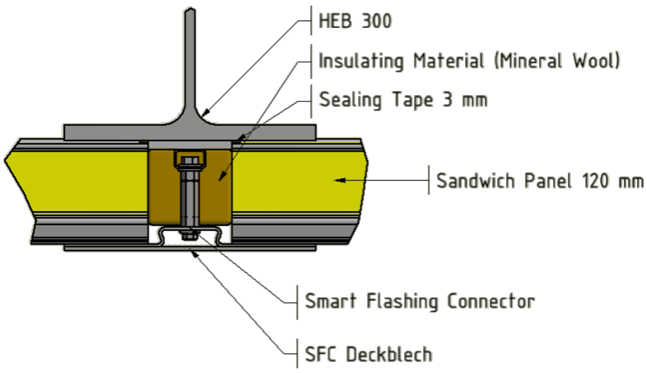


Fig. 11.22 Clamped fixing system for long span wall cladding

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

Fig. 11.23 Example of roof system with deep decking [138]

Fig. 11.24 Example of roof system with long span composite panels [138]

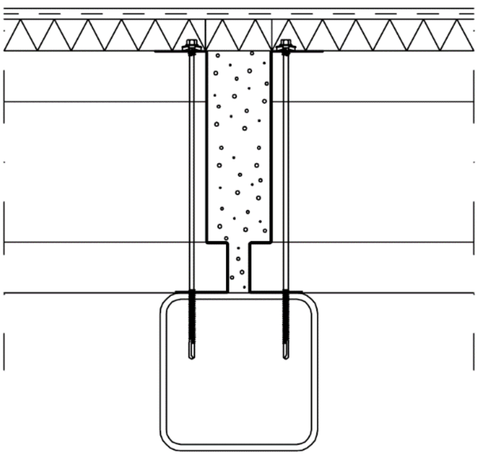


Fig. 11.25 Connections between long span composite panel and primary structure [138]

11.7.5 Use of pre-fabricated light steel cassettes in portal frames

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

The same form of cassette may also be used for roofs (accessible only for maintenance) and walls in portal frames for spans of 6 to 7.5 m. The cassette may be suspended from the top flange of a rafter beam by an edge Z–section, as shown in Fig. 11.26.

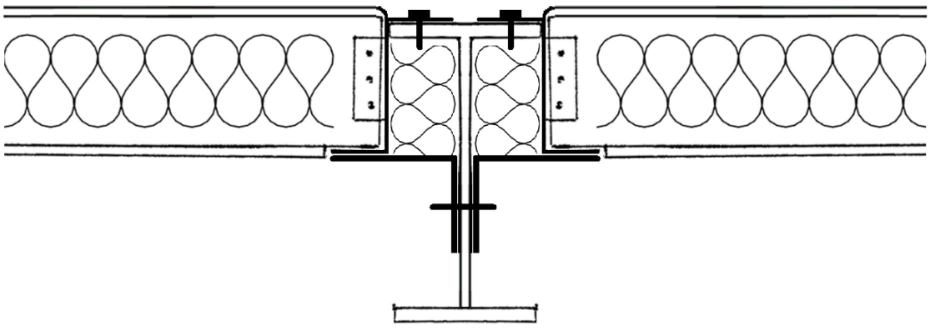


Fig. 11.26 Use of pre-fabricated insulated suspended roof or floor cassette [41]

An alternative would be to place the cassette on the top of the flange, in which case, U-sections are used at its ends (Fig. 11.28 11.27). A gap between cassettes may be provided to facilitate erection, which can later be filled with a thin layer of insulation. A minimum top flange width of 150 mm is recommended. Self-drilling screws with sealing washers can be used to fix the cassette to the primary steelwork. The solution is also suitable for a trussed solution with hollow sections.

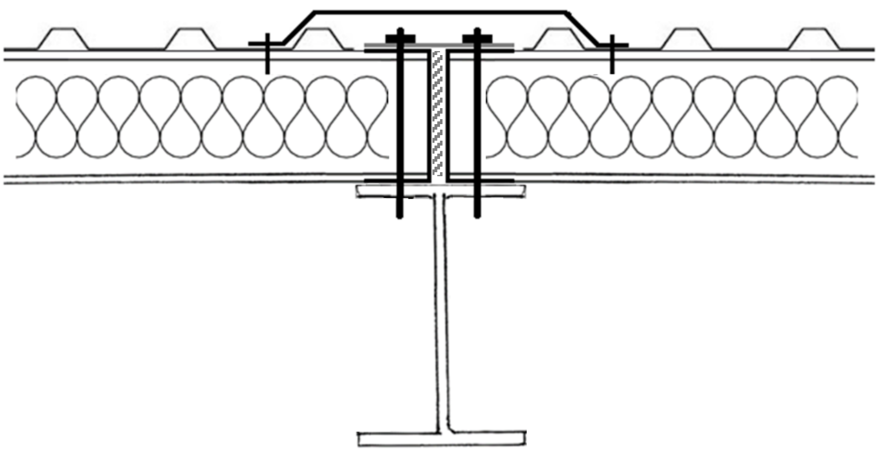


Fig. 11.27 Use of pre-fabricated insulated roof cassettes supported on the top flange [41]

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

The details of a typical roof cassette are shown in Fig. 11.28 and Fig. 11.29. To allow for tolerances, the Z-section should be 75 mm wide and 2 to 3 mm thick, positioned to allow for a flat soffit levelled with the cassettes surface. The C sections are 1.2 to 2 mm thick and are placed at 600 to 900 mm spacing depending on roof/floor loads and cassette span. The C-sections may have perforated webs to improve the thermal performance of the system (potential heat loss via cold bridging of the element by 70 to 80% - Fig. 11.29) [138].

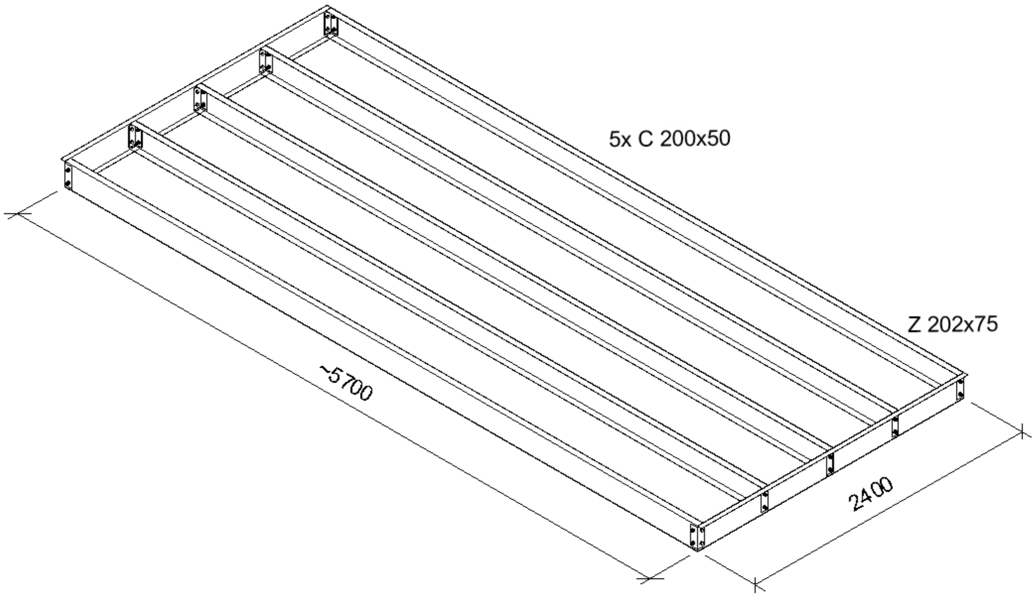


Fig. 11.28 Light steel cassette used in pre-fabricated roof construction for 6m beam spacing (less the beam width of nominally 300mm) [41]

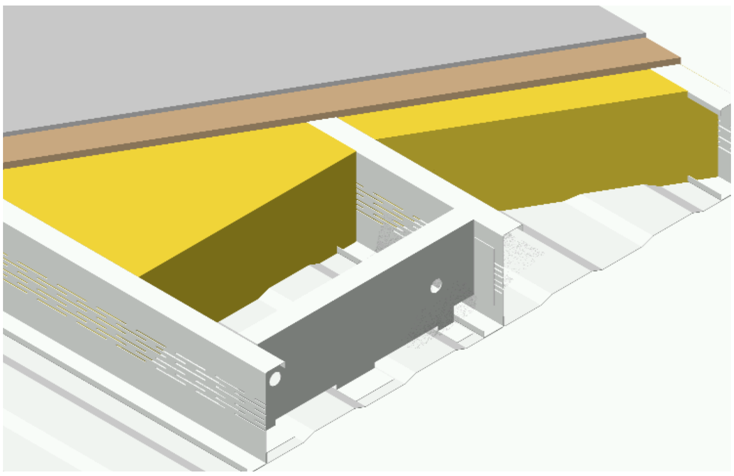


Fig. 11.29 Light steel cassette with perforated webs [138]

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

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

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

Diaphragm stiffness is also provided by the boarding/sheeting applied to the floor cassettes and the screws to the beams. Tie cap plates can be used to connect the two roof cassettes placed over a rafter (as shown on Fig. 11.27), allowing for improved diaphragm action.

In the system shown in Fig. 11.26, the upper part of the beam may be insulated between the roof cassettes to reduce thermal bridging. This can be done from the inside of the building. Additional continuous angles may be bolted to the web of the beam, so that their tension-compression action provides torsional restraint to prevent lateral torsional buckling of the beams under negative bending.

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

The final over cladding layer of the building may be incorporated in the cassette itself (as done on a typical sandwich panel) or installed on site over top ‘hat’ spacer sections that were previously attached off-site over the board. The benefit of the latter is that the roof sheeting can easily be replaced in the future and the cassettes are retained without affecting the use of the building nor damaging the cassette while replacing the sheeting.

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

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

Part 3: Case studies

12 CASE STUDIES

12.1 General

The end-of-life of construction products play an important role, considering the circular economy, as buildings have a long life-span and require a significant amount of material resources. In order to maintain circularity, preserve resources and minimise environmental impact, these resources should be kept in-use through re-use, reclaim or recycling.

This section demonstrates the technical performance and cost-efficiency of the developed methods and strategies for reuse of existing single-storey steel building components.

The following two theoretical studies are presented for two different situations:

- a building is design from reclaimed steel elements and environmental and economic benefits of reused elements are highlighted, and;
- an existing building that offers accurate as-built investment and operating cost information will be re-designed to maximize deconstruction and reuse of its components.

In the first case, the main aim is to go through the processes needed if reused components are used in a building. The second case focuses on the design where future reuse of the building elements has been considered. One of the main aims is to demonstrate the competitiveness of steel buildings due to their high reuse possibilities.

12.2 Building design from reclaimed elements

12.2.1 Design constraints

This section presents theoretical studies of buildings designed using reclaimed steel elements to declare environmental and economic benefits of the reused elements.

It will cover the following six different theoretical case studies, presenting a comparative environmental and economic impact of the same steel building when the structure is built reusing an existing steel structure, either using reclaimed elements or using new construction materials. The purpose was to compare environmental and economic indicators (in terms of impacts such as GHG emissions and costs) of steel structures made from elements using new materials along with structures made from reused steel components, by quantifying the savings achievable by reusing construction materials. The study is based on an environmental impact assessment and on a life cycle cost assessment of a single-storey industrial hall constructed in Romania. The assessments were carried out and compared for the following scenarios:

1. Baseline scenario (case 0) where the structure was designed as a new structure made with elements using new materials (optimal design).
2. Second scenario (case 0+) where the structure was designed with elements from new materials, taking account of the deconstruction methods at the end-of-life of the

structure– covering also part of Section 12.3 regarding Design for Deconstruction (see Fig. 11.5 & Fig. 11.6 as an example of design for deconstruction).

3. Third scenario (case 1) regarding a relocated steel structure; the scenario considered the reuse of an existing steel structure originating in Germany and reassembled in Romania (see Fig. 12.1).
4. In the fourth scenario (case 2), it was the steel structure is made with reclaimed elements. Existing profiles for beams and columns were identified in a storage yard in Germany obtained from other deconstructed buildings. All other components were made with new steel.
5. The fifth scenario (case 3) closeness Case 2, considering reclaimed elements such as columns and beams and also their end plates. All other components use new steel.
6. The last scenario (case 4) considered the reuse of an entire structure relocated from Germany to Romania. Therefore, the percent of the reused steel in the superstructure in this scenario is 100%.

The assessment was carried out by defining the following system boundaries:

- The heated floor area of the industrial hall is 525 m^2;
- Other materials and components considered are:
- Concrete foundations and concrete floor: 185 m^3;
- Triple glazed windows: 22.5 m^2;
- Sectional sliding gates: 48 m^2;
- The U-value for the external walls is 0.56 W/m^2K, for roof elements is 0.34 W/m^2K, for ground floor slab is 0.76 W/m^2K while for windows and entrance-door is 1.3 W/m^2K;
- The operational lifetime of the new building is 25 years;
- The assessment considers the main following building components: foundation and ground floor slab (concrete and steel reinforcement), load bearing structure (hot-rolled and cold-formed steel elements), sandwich panels (panels with mineral wool insulation), triple glazed windows and sectional sliding gates;
- The steel reinforcement was counted as new material, with an input of 20% steel scrap in the process of manufacturing and an end-of-life scenario with 85% recycling potential and 15% landfilling or loss material after sorting.

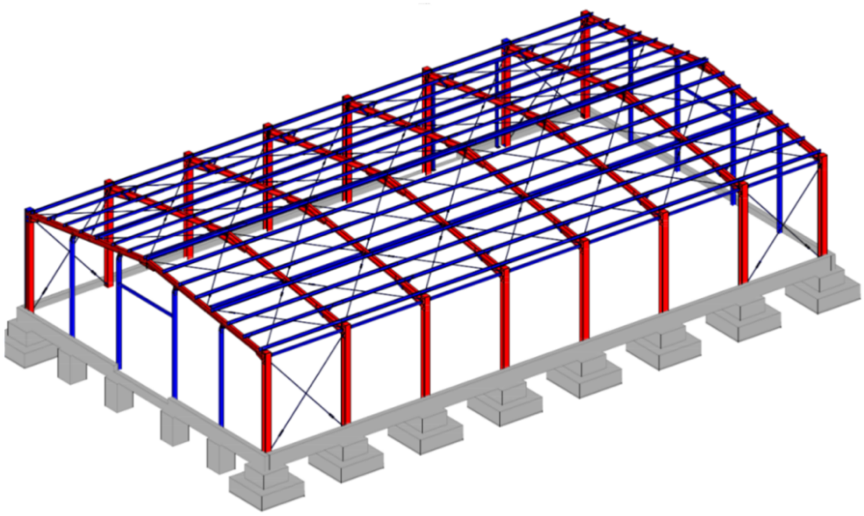


Fig. 12.1 The steel structure rebuilt in Romania (reused steel in red, and new steel in blue)

12.2.2 Environmental assessment

The life cycle assessment method (LCA) chosen for the evaluation of the environmental impact follows the rules of ISO 14044 [145], EN 15804 [35] and EN 15978 [36]. The approach for accounting of recycling in LCA is based on the modular building life cycle approach, where the benefits beyond the system boundary are reported separately.

The Global warming potential (GWP) was considered the indicator for the environmental performance. Each case from the four cases studied covered an assessment for “demolition and recycling” and one for “deconstruction and reuse” in the end-of-life module (except for the Case 0+ which was considered for reuse only, as it included ‘design for deconstruction’), see Table 12.1 and Table 12.2.

Table 12.1 **Assessed scenarios for steel in superstructure (incl. purlins) – input: recycle scenario**

End-of-life for steel in superstructure: recycle scenario	Input to the assessment			Other materials aspects of use		
	New material	Reused material	Recycled material (scrap)	Waste	Material for reuse	Material for recycling (scrap)
Case 0	80%	0%	20%	10%	0%	90%
Case 1	27.9%	65.1%	6.7%	10%	0%	90%
Case 2	35.6%	55.5%	8.9%	10%	0%	90%
Case 3	29.7%	62.9%	7.4%	10%	0%	90%
Case 4	0%	100%	0%	0%	0%	100%

Fig. 12.2 presents the total LCA results for the assessed scenarios. The LCA savings are reflected as negative values while positive values define burdens of material utilisation. It can be seen that benefits and loads beyond the system boundary are not aggregated with the life cycle impacts (Modules A to C), as provided by the CEN/TC 350 methodology.

According to the results, highest environmental impact is shown by the cases when structures erected with new elements are deconstructed for the next reuse case (1.24 t CO_2 e/m^2 – case 0 and 0+) while the lowest rate of emissions are registered when structures are built with reused portal frames and at the end-of-life of the structure steel is recovered for recycling (0.97 t CO_2 e/m^2 – case 1).

Table 12.2 Assessed scenarios for steel in superstructure (incl. purlins) – input: reuse scenario

End-of-life for steel in superstructure: reuse scenario	Input to the assessment			Other materials aspects of use		
	New material	Reused material	Recycled material (scrap)	Waste	Reused material	Material for recycling (scrap)
Case 0	80%	0%	20%	1%	90%	9%
Case 0+	80%	0%	20%	0%	100%	0%
Case 1	27.9%	65.1%	7.0%	3%	72%	25%
Case 2	35.6%	55.5%	8.9%	4.5%	55.5%	40.1%
Case 3	29.7%	62.9%	7.4%	3.7%	62.9%	33.4%
Case 4	0%	100%	0%	0%	100%	0%

The results show that beyond the system boundary, in the scenarios where the structures built with reused elements and at the end-of-life steel is recovered for recycling, a burden is recorded in the assessment. The highest potential savings (0.244 - 0.269 t CO_2 e/m^2) appear in the scenario when the industrial hall was constructed with new elements which are deconstructed at the end of this use for the next reuse case.

12.2.3 Economic assessment

The calculation of economic indicators is based on the same scenarios and modules as in the LCA analysis, referring to new steel and reused steel structures, to which time and financial costs are associated. The assessment of the economic performance of studied cases follows the methods described in EN 16627 [37]. Potential cost savings include revenues for recycled steel (earnings from the sold steel scrap), revenues from the sold steel elements or sold structure and revenues from the sold sandwich panels. Additional costs for expertise, redesign, testing, sandblasting, repainting were counted in cases where reused steel was involved.

According to the results, highest LCC (modules A-C) is shown by the cases when structures erected with reclaimed elements are deconstructed for the next reuse case (608 €/m^2) and lowest LCC by the case when construction uses new steel structures and after demolition recovered steel is sold for recycling (547 €/m^2). Fig. 12.3 presents the total LCC results of the assessed scenarios, where the savings are reflected as negative values. When considering the loads and benefits from Module D only, the highest savings (34,70 to 36,66 €/m^2) are obtained when the structure is deconstructed for a future reuse at the end-of-life. Based on the total life cycle, the costs of the structure built with reused materials (cases 1, 2, 3 and 4) exceed the costs of the structure designed with elements from new materials (case 0 and 0+).

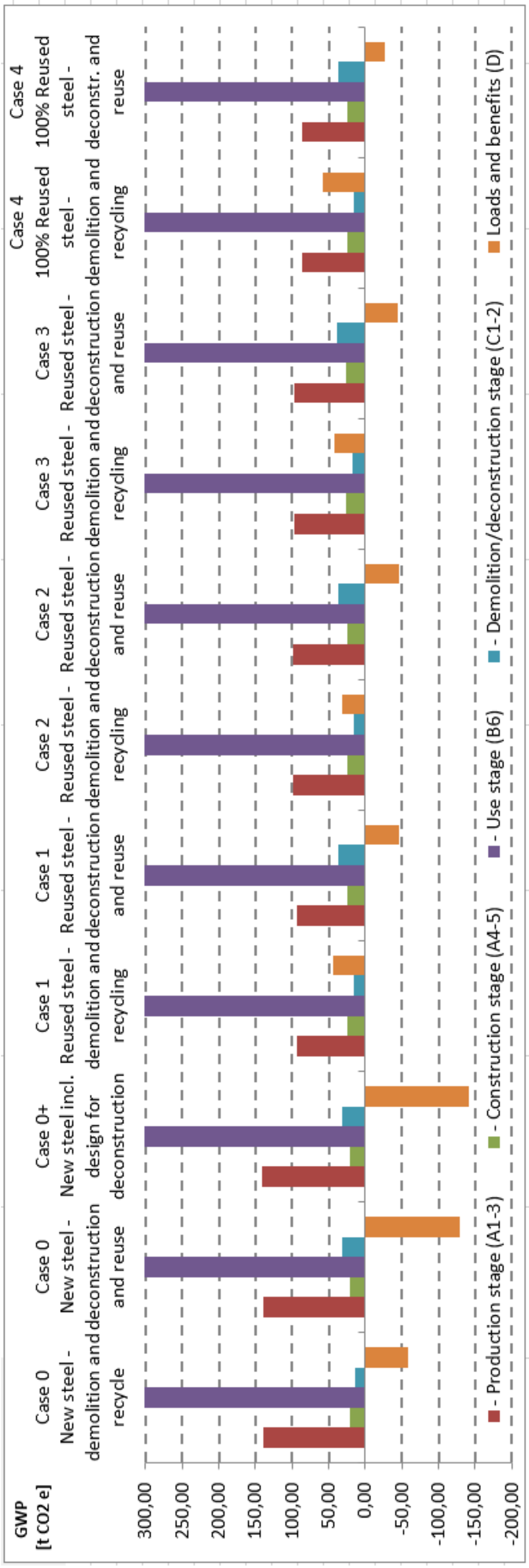


Fig. 12.2 Total LCA results of the scenarios including loads and benefits beyond the system boundary

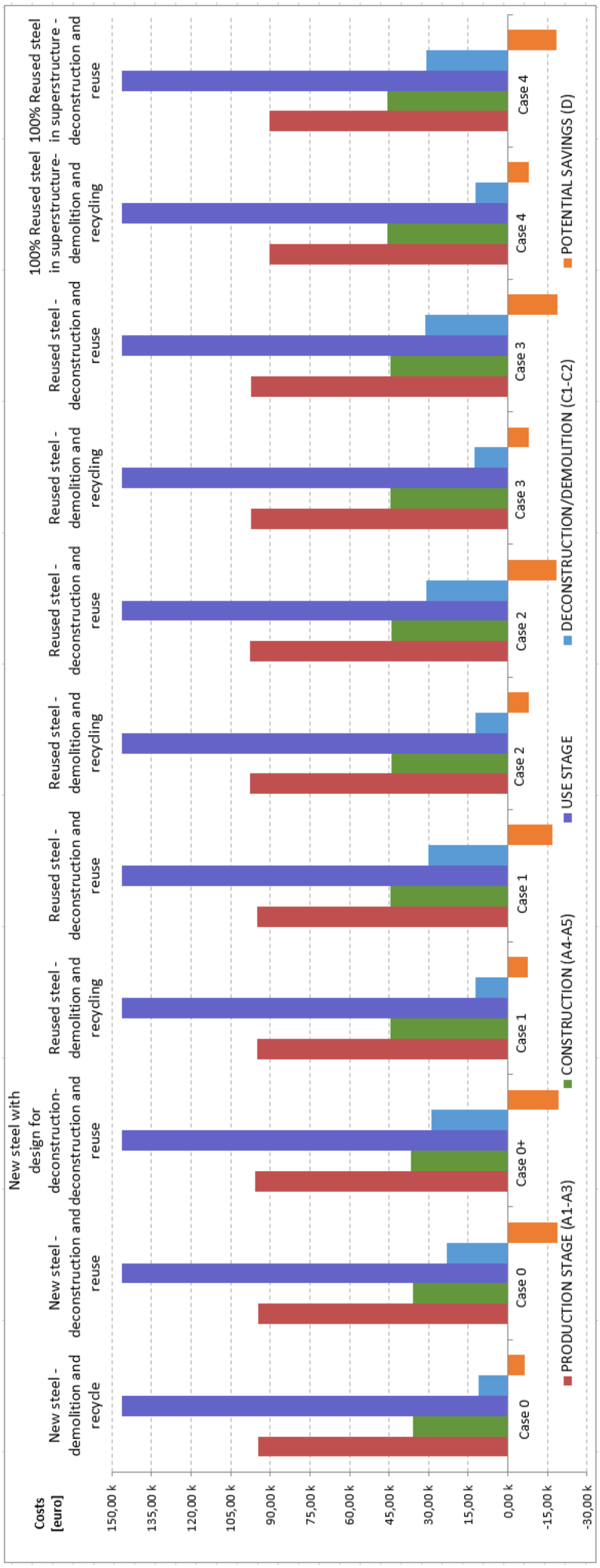


Fig. 12.3 LCC results of the scenarios, including potential savings

The LCA results show that the reuse approach is a strategy that has higher environmental benefits compared to a recycling approach (modules A-C). The greatest gain is achieved in the production stage (A1-3) where GHG emissions are 29 to 33% less when the structure is built with reused steel. The results give 188.5 kg CO_2 e/m^2 for structures built with reused steel in comparison to 266.3 kg CO_2 e/m^2 for structures built with new steel.

According to the results on the economic potential, the scenario using reused steel elements results in higher potential savings of 34.7 to 36.7 €/m^2 compared to recycling scenario (between 12.5 to 15.3 €/m^2).

12.3 Building design for improved reusability in the future

The example building for the design case is an existing warehouse building located in Tampere, Finland. The building is rectangular in shape with length of 41.750 m, width of 31.5 m and height of approximately 10m. The building has a steel frame, with hollow sections as columns and hollow section trusses as main structure and beams in the end of the building.

The frames are spaced at 5 to 7m, and there are six columns with 6.15 m distance between them in both ends of buildings. In lateral direction, building is stiffened by rigid frames and in longitudinal direction, braces are used in both wall and roof to transfer horizontal wind loads.

The building has profiled sheeting in the roof, which also act as a horizontal restraint for top chords of the trusses. Walls consists of sandwich panels that provide the required insulation and act as the façade. In addition to storage spaces, there are also business premises inside the building that include meeting rooms, offices, and social areas. Screen captures from existing building information model are shown in Fig. 12.4 and Fig. 12.5. Fig. 12.4 shows the entire building and Fig. 12.5 shows the structure.

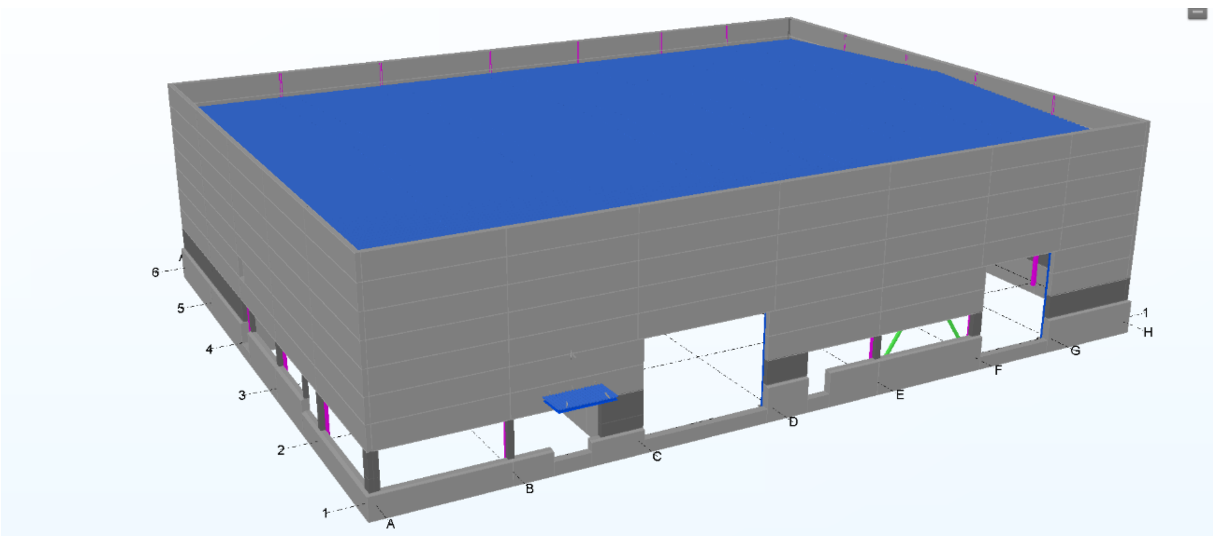


Fig. 12.4 Screenshot of existing buildings information model

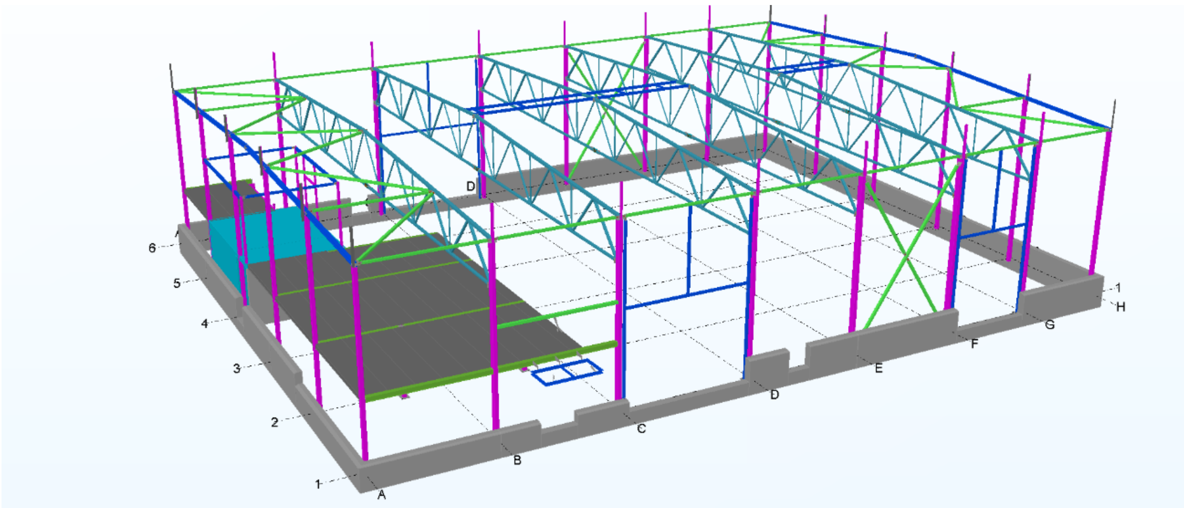


Fig. 12.5 Structure of the existing building

Re-design strategy

The case study targeted the ways to maximize reusable steel weight after the building is deconstructed. Solutions identified in the case study were based on observations made in literature review. There were originally 14 different solutions made to improve the reuse possibilities as follows:

- regular distance between columns,
- increasing environmental loads (mainly snow load),
- braces as lateral restraints (instead of sheeting),
- braces for stiffening,
- regular cross-sections for columns,
- regular material grade,
- regular load-bearing roof sheeting,
- friction clamp connection for sandwich element detailing,
- assembling columns from standard length pieces,
- base – and end plates connected with bolts to columns,
- same cross-section for diagonal bars,
- “expendable” parts for connection parts,
- regular distance between inside columns,
- using screws as fasteners for load-bearing sheets instead of shot fired nails.

One important idea in the proposed solution was the use of “expendable” parts. If complex gusset or connection plates are welded into main components, either remanufacturing is needed, or reuse possibilities are limited. Connection details should be bolted to the main components and these connection assemblies can be “sacrificed” and sent for recycling. The aim of this is to assure that the larger components with the highest weight would be easier to reuse while smaller components with more specific details and lower weight can be recycled. In the case study, this was realized for example by avoiding welded parts in the main components but introducing them only with predrilled boltholes where prefabricated expendable parts may be connected. Some examples of this kind of expendable parts are shown in Fig. 12.6.

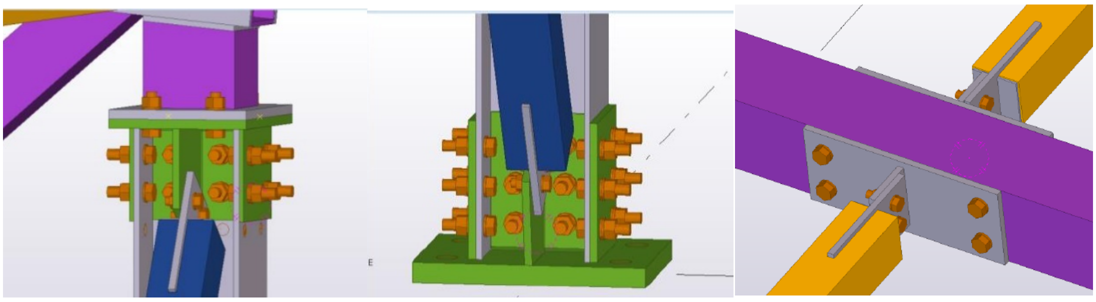


Fig. 12.6 Proposed details for columns base, truss-column and bracing connections

The proposed method included also concepts where the steel columns were all of regular length of 3, 6 or 9 m and were fitted with prefabricated holes in the ends for universal joints. If column lengths apart from standard lengths are needed, the extra short column piece would be used and could be sacrificed in dismantling.

Financial evaluation

In the first part, several re-use design options were presented to meet different levels of reusability of the building components. If all the above-mentioned solutions were included in the re-use design, the total steel mass would be 85.24 tonnes while the example building had 61 tonnes of steel. Two more re-use solutions were defined where some of the first proposed methods were rejected due to too high investment costs. For example, the standardized column system with universal connections proved to be too expensive.

Steel quantities and number of fasteners in different options is shown in Table 12.3. Descriptions of different options are shown in the Deliverable report. However, even the lightest version includes for example additional live loads, regular column spacings and material grades, as well as expendable parts in main components. Finally, the last option “Re-used design 3” was selected for more closely economical consideration. In this option, the total weight of the components was about 10% higher than in the original design.

Table 12.3 **Steel quantities and fastener amount in different re-use options**

Elements	Original design	Re-use design 1	Re-used design 2	Re-use design 3
Profiles	39026 kg	50846 kg	49 801 kg	42760 kg
Plates	4286 kg	7983 kg	7 983 kg	5992 kg
Sheets	17680 kg	26015 kg	17 680 kg	18033 kg
TOTAL	**60,992 kg**	**84,844 kg**	**75,464 kg**	**66,785 kg**
No. of fasteners	387	2749	2749	1315

It follows that a higher steel weight means generally higher capital investment. Therefore, it was essential to estimate profitability of higher investment and the evaluation was made with

Net Present Value method (NPV). The NPV method was used to estimate a service life limit for buildings with estimated extra investment and variable residual value differences and interest rates.

One way to study profitability of the concept is to define critical life span to the building structure when extra investments in the re-use concept still gives positive NPV for a given interest rate value and difference in residual value. An example of this is shown in Fig. 12.7 where the NPV limit has been calculated for different life spans with fixed interest rate of 5% and fixed residual value difference of 40%. This is the residual value expressed as the % of re-used case minus residual value % of normal case. As an example, according to Fig. 12.7, the critical time for disassembly in terms of NPV would be after 20 years, if the extra investment is 15%.

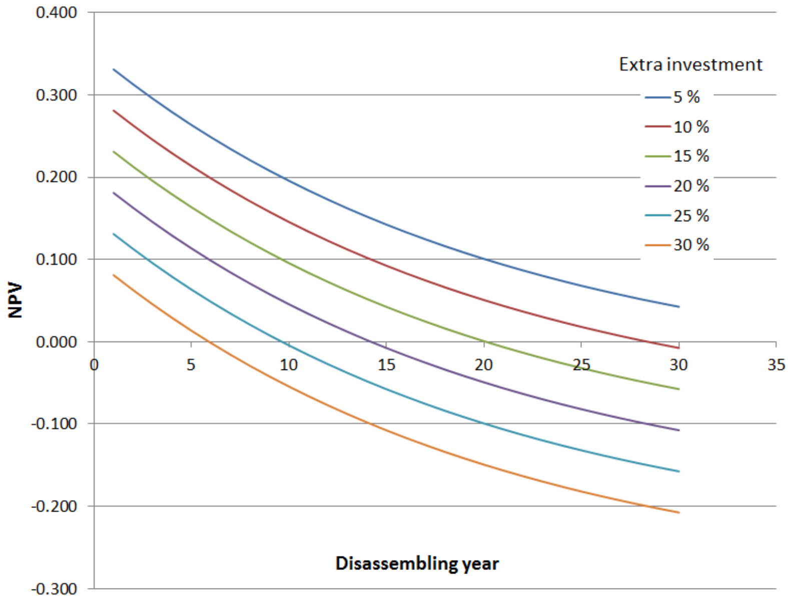


Fig. 12.7 Profitable disassembling year limits with different extra investment levels. Difference in residual value percent is 40% and interest rate is 5%

Investment profitability can be estimated from different viewpoint. If interest rate and service life are constants, the extra investment limit with different residual value differences can be calculated with NPV. These further NPV calculations were made to evaluate the limit for the extra investment. These calculations were made for 20-year service life and 5 % interest rate. For example, it was found that if extra investment due to re-use design is 12%, the residual value of the re-use design should be about 32% higher than in the normal case in order to be profitable (i.e. positive NPV).

These results from the NPV calculations are helpful in estimating investment profitability, but there are some problems in the estimations. The residual value is taken as constant over time and it does not reflect reality. The residual value of components for design for reuse is also dependent on the price of a new component over time and these calculations do not take this into account. It leads to some inaccuracies in the estimations that are in favour of reuse economics.

A reuse project would also be a good marketing asset for companies involved in the project. NPV calculations may give conservative estimations of investment profitability in the case of design for reuse.

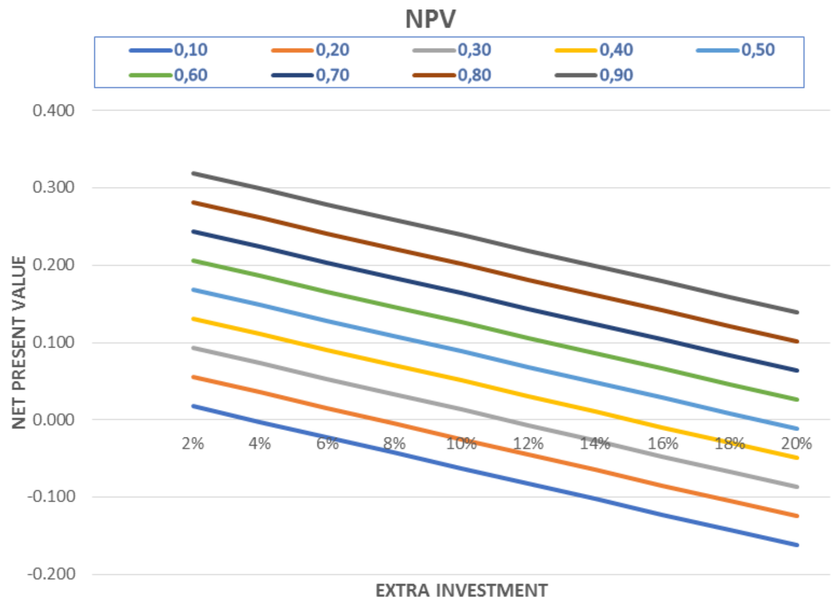


Fig. 12.8 Extra investment limits with different levels of residual value; 20 years' service life and 5% interest rate

In the second part, as economic study was performed to resolve the feasibility of design for deconstruction (DfD) and reuse in a case of a single-storey steel building.

In the first step, the economic feasibility of DfD was explored by comparing the net present values (NPV) in 10-year discounted cash flow between two buildings: a building not designed for deconstruction (Scenario 1) and a building designed for deconstruction (Scenario 2). The only difference between buildings was that the DfD building was assumed to be constructed by applying the DfD principles that was reflected in slightly higher construction costs of 61k€ or +5,6% in the base cost. Then by changing some parameters where the DfD was thought to have an impact, the aim was to find favourable options for the DfD building when comparing the NPVs.

An assumption was made that the buildings will remain in their current use after a 10-year period, so the building would not be deconstructed in near future. The calculated NPVs were positive for both buildings and were €148k for the traditional building and €88k for the DfD building. The initial difference of about €61k favouring the traditional building compared to the DfD building resulted from the higher initial construction costs for the DfD building.

After this basic calculation a sensitivity review and break-even analysis were performed. By changing the some of the parameters (such as yield, market rent, operating costs, construction costs), where the DfD principles can be thought to have an effect, the aim was to discover the ways in which the DfD building (Scenario 2) would be a better option than a traditional building (Scenario 1).

Break-even points for the DfD building were yield of 6,7%, market rent of 8,28 €/m^2/month and operating costs of 1,73 €/m^2/month. The base level for these parameters were yield of 7%, market rent of 8 €/m^2/month and operating costs of 2 €/m^2/month. When changing two parameters simultaneously, to a yield of 6,9% and market rent of 8,2 €/m^2/month, the DfD building becomes more profitable.

Studies have shown that environmental certificates have a positive effect on the value of the building when compared to a building without a certificate. However, as the DfD principle in procurement of buildings does not yet have the same established "image premium" as the environmental certificates, significantly lower yield requirements may be difficult to justify for the investor. As the most tangible asset of the DfD occurs in the deconstruction phase of the building, this asset is hard to quantify, if the building will stay in its use more than 10 years.

In the second step, it was assumed that after certain time period (of 10, 20, 30 years) the building will either be demolished in case of the traditional building (Scenario A) or deconstructed in case of DfD building (Scenario B). The calculation compared the discounted residual values of demolition (where most of the steel is recycled) and deconstruction (where most of the steel is reused). In other words, this second calculation does not include normal real estate business incomes and costs from the use phase, but the comparison is made only taking into account end of life costs. In the deconstruction option, the initial construction cost premium of about €61k was also taken into account when making the comparison. This can be thought as an extra investment for the DfD building which enables the future reuse.

The discounted residual values calculated were negative for both buildings in the three time periods because calculation covers only the end of life phase without use phase. In the 10-year assessment using the base parameters, the discounted residual value when including the extra investment was about -€108k for the DfD building and about -€98k for the traditional building. Therefore, the difference favouring the traditional building was €10K in the 10-year assessment but there was some uncertainty related to the largest cost items (deconstruction/demolition costs, construction costs, resale price of the components).

In order for the DfD building with reuse (Scenario B) to break-even against the traditional building with recycling (Scenario A) when comparing the discounted residual values in 10-year assessment, for example the present value of the saleable components should be about 13% higher than in the base case. This would mean that their value should increase from €81k to €92k. The construction cost premium should decrease from €61k to €50k for the Scenario B to become economically a better option.

If the other deconstruction costs for the Scenario B decrease more 13% from the base figure of 119€/sqm to 103€/sqm, this would make the Scenario B a better option. As there was some uncertainty related to these cost items, these kind of changes favouring the DfD building with reuse could be possible.

When the time period is increased, this favours the traditional building with recycling, as the benefits of reuse are reduced over time and the effect of the initial construction cost premium becomes more important. In the 20-year assessment, the difference favouring the traditional building with recycle option when comparing the discounted residual values was about €42k. For example, for the DfD building with future reuse to break-even in the 20-year period when comparing the discounted residual values, the present value of the components should be about €77k, whereas in the base case their value was calculated to be about €35K. It follows that all the residual values are relatively low comparing to the total investment cost.

The calculation was also performed for the situation where the building is not designed for deconstruction but its parts (the steel structure, roof structure and sandwich panels) are reused. The comparison was made between the reuse and recycle option for the traditional building. It can be speculated that the future tightening of legislation may impose requirements that the major parts of the building must be deconstructed in a way that enables their reuse.

When compared to the DfD with reuse option, it was estimated that the deconstruction costs for the reusable parts were 25% higher and the costs of preparing for reuse were estimated to be about. €12K higher. Other parameters were assumed to be the same. These assumptions were quite moderate, and it is possible that costs of deconstruction and preparing for reuse could be higher. However, with these assumptions, it would make sense to deconstruct rather than demolish the building at the end of its life cycle even though it was not specifically designed for deconstruction.

Overall conclusions

With current practices the principles of DfD and reuse would not be market driven, when the building remains in place for more than 10 years. The results from the DCF calculations support the view that when the possible benefits of DfD and reuse are in future, this is not attractive for investors. As the costs of DfD incur in the construction phase, there is an imbalance of the benefits and costs from the investor's point of view. This means that currently it is not profitable to choose to design a DfD building over the traditional building for a reuse cycle of more than 10 years.

Two good examples were found in Finland with using the DfD principles (Hakaniemi's temporary market hall and a shopping centre in Pikkulaiva). These cases show that DfD and reuse are utilised if the building is known to be moved and re-built in a period of 2 to 5 years after its initial construction. If the time span of the building is longer, using the DfD principles becomes more questionable from the economic point of view. The building parts that are reusable for after 30 years probably include only the frame of the building. Overall it can be thought that the potential user group of DfD and reuse is user-owners rather than investors.

It should be noted that there are uncertainties in the real investment costs if building is designed for deconstruction. The main structure of the building is reusable. Also, the study showed that the differences in net present values (NPV) in all studied cases are relatively low between traditional and DfD buildings. This encourages a change in design culture towards the design for deconstruction. This could be also achieved by changes in regulations.

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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 inevitably 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 in combination, 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 serious 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 was described on section 7.5.3. Flowcharts on Fig. 4.1 and Fig. 7.7 illustrate the overall framework to reclaim existing structural steel elements. It was also highlighted the necessity to assess the fabrication processes for existing steelwork (mainly welds), to make sure that such processes agree with the quality requirements according to EN1090. The following section provides further guidance to the assessment of the existing steelwork as well as for inspection of the 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 lather 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 in accordance with 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 [146].
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 [147] and EN ISO 13385-2 [148].

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 the defects.

A.2.4 Inspection protocol for welds

Welds between plates of fabricated members (including cellular beams) shall be inspected. As stated in section 7.6.5, the same amount of weld testing required by EN 1090-2 (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-destructives tests. A connection may have different weld segments. In a typical rafter-columns connection, 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. 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. BS EN 17637 [73].
Penetrant testing	Dye highlight 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 [149] gives the general principles for this technique. For welds, see EN ISO 23277 [155].
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 [150] and for welds see EN ISO 17643 [151].
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 [152]. For welds see EN ISO 17640 [153].
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 [154] 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 [156] can be pointed out as a reference.

Table A. 3 **Suggested minimum percentage of welds to be tested [157]**

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

If original construction documents, including drawings and specifications, are available, it will typically not be necessary to perform materials testing in a steel moment-frame building. 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 2-6 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 flange tips at ends of simply supported beams, flange edges in the mid-span region of members of moment-resisting frames, and external plate edges.

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 summarizes 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 [158] 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 [159] and Brinell ISO 6505 [160] test methods. See also ASTM 1038:2017 [161].
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 [162]. See ASTM E572 [163] and ASTM 1476 [164].
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 145775 [165].
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 [166] [167] 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 extracting small samples from the existing structure. Potential DT techniques are identified in the table below. 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 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 [168].
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 [169].
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 [170]. 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 [171].

A.5 Testing Protocols

A.5.1 Minimal testing

Minimal testing is intended for the cases where material documentation is available (Class A steel – section 6.2.1) or to perform a preliminary assessment of existing steelwork. Minimal testing can also be used as part of a preliminary assessment as described in section 7.5.3.

The optional minimal testing is intended to confirm that a certain existing material documentation is related to a certain group of steel elements. Only non-destructive tests are recommended. A summary of the minimal testing procedure is presented in Table A. 6.

Table A. 6 **Recommendations for minimal testing**

Characteristic to be determined	Type of testing	Percentage of elements to be tested
Tensile and yield strength	Non-destructive	10% – with a minimum of 3 tests per group/test unit
Chemical composition (CEV)	Non-destructive	

A.5.2 Comprehensive testing – hot rolled and hollow sections

The recommendations for comprehensive testing require 100% non-destructive testing in combination with non-statistical or statistical destructive testing. The non-destructive testing of all reclaimed members establishes that a group of members can be represented by destructive test results from one or more representative members from the group.

Non-statistical testing requires one destructive test, taken from a member of each group, to confirm the results obtained from the non-destructive tests. Non-statistical testing is only recommended for Consequence Class (CC) 1 or 2 structures. Non-statistical testing is equivalent to the requirements for ‘new’ steel specified in the product Standard (say EN 10025-1).

Statistical testing requires more destructive tests to assess material characteristics in accordance with EN 1990. Statistical testing is recommended for reclaimed steel to be used in CC3 buildings, or when the provenance or quality of the original source material is considered to be unreliable. Statistical testing exceeds the requirements for ‘new’ steel specified in the product Standard (say EN 10025-2).

Table A. 7 relates the recommended testing approach for yield strength, ultimate strength, elongation and chemical composition to Consequence class.

Table A. 7 **Testing procedure for hot rolled and hollow sections products [172]**

Consequence class	NDT to establish yield strength, ultimate strength and CEV	Minimum number of DT to establish yield strength, ultimate strength and CEV and elongation	Acceptance approach
CC1	All members to be subject to non-destructive tests	1	Non-statistical (maximum value of CEV)
CC2		1	
CC3		3	Statistical for yield strength, ultimate strength and elongation (maximum value of CEV)

A.5.3 Comprehensive testing – cold formed steelwork

Table A. 8 relates the recommended testing approach for yield strength, ultimate strength, elongation, and chemical composition (if needed) according to the building Consequence Class. Non-statistical testing is not recommended for reclaimed cold formed steel elements.

Table A. 8 **Testing procedure for cold formed light gauge products**

Consequence class	NDT to establish yield strength, ultimate strength and CEV[1]	Minimum number of DT to establish yield strength, ultimate strength and CEV[1] and elongation	Acceptance approach
CC1	All members to be subject to non-destructive tests to establish yield strength and ultimate strength (and CEV if required[1]).	3	Statistical for yield strength, ultimate strength and elongation (maximum value of CEV if required[1])
CC2		5	
CC3		7	
1 – Usually not required as welding procedures are not often used with cold formed elements			

Purlins, side rails, and other type of elements shall be treated separately for each type of profile cross-section (as individual groups) with a maximum of 4 tonnes. Statistical testing requires destructive testing to assess material characteristics in accordance with EN 1990. Statistical testing exceeds considerably the requirements for 'new' steel specified in the product Standard (EN 10346).

To undertake hardness measurements, coating system need to be removed first. The testing area can be repaired with a zinc-rich spray.

Samples for destructive tests shall be collected from the profile web, as far away as possible of any bent part. The coupons can be simple strips of 250x20mm collected from different elements.

A.6 Comprehensive testing implementation for strength and elongation

A.6.1 Introduction

Material strength and elongation are assessed by both destructive and non-destructive tests, as recommended by Table A. 7 and Table A. 8. 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. 9 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. 10.

Table A. 9 **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. 10 **Steel properties data according to reference [173]**

Steel grade	Yield Strength Mean (X characteristic value)	CoV	Tensile Strength 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. 12 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. 10.

As buckling curves do not depend of the yield strength for cold formed 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. 12.

Table A. 11 **Recommended minimum values for yield and tensile strength to undertake the reliability assessment of cold formed 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. 12 **Steel properties data according to reference**

Steel grades	Yield Strength Mean (X characteristic value)	CoV	Tensile Strength 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 the flanges of 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 ultimate strength should be calculated and compared with the minimum values from Table A. 9 and Table A. 11 to define the steel grade.

Assessment of hardness test results

The hardness of an individual member should be taken as the average of three 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 D1 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. 13.

Table A. 13 **Values of kn for the 5% characteristic value (EN 1990 Table D1)**

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. Vickers hardness test according to EN ISO 6507 [158] is one of the available options.

Correlation between hardness and material strength

If the Vickers hardness test is used, the following relationship between hardness and strength can be used to estimate the material properties based on reference [174]:

$$f_y = 2.70\,H_v - 71 \quad \text{(A.2)}$$

$$f_u = 2.50\,H_v + 100 \quad \text{(A.3)}$$

where:

H_v – Is the Vickers hardness value for the group;

f_y – Is the yield strength;

f_u – Is the ultimate strength.

The proposed empiric correlations show a good agreement according to EN ISO 18265 [175] and considering the data from Table A. 9 and Table A. 11.

Calculation example

In this example, 20 steel members have been identified as a group. Each member was subjected to a non-destructive hardness test. Three measurements were taken from each member and the mean result was calculated. The mean of the 20 individual results resulted as 169.5. The standard deviation resulted as 5.06.

As 20 members have been tested, n = 20 and kn = 1.76 (from Table A. 13)

For the group, H_v = 169.5 – 1.76 × 5.06 = 160.6

If H_v = 160.6, then:

f_y = 2.7 × 160.6 – 71 = 362 N/mm^2

and:

f_u = 2.5 × 160.6 + 100 = 502 N/mm^2

According to Table A. 9, the steel is identified as S275, as the yield strength is greater than 313 kN/mm^2 and the ultimate strength is greater than 452 kN/mm^2.

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

Introduction

The location of 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 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 [168]. As a reference, test sample locations may be defined according to ISO 377 [176]. Guidance from the relevant product standard may be also followed, for examples, EN10025 or EN10219.

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 EN1090-2 section 5.1. As an example, if the elongation at failure measured by a destructive test does not comply with the minimum values from EN10025-2 for a specific steel grade, but if the measured elongation is such that the minimum values from EN 1993-1-1 for elastic global analysis are fulfilled (Table 6.1), 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. 9 or Table A. 11. 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.

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:

$$X_\mathrm{d} = m - k_\mathrm{n} S_\mathrm{x} \tag{A.4}$$

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. 14.

Table A. 14 **Values of k_n for the 5% characteristic value (EN 1990 Table D1)**

Number of DT	1	2	3	4	5	6	8	10	20	30	∞
V_x known	–	–	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. 9 or Table A. 11.

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. In accordance with 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 [170].

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 EN 10025-1 section 7.2.3 or EN 10219-1 section 6.6.1.

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 need 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 BS ISO 19272 [162].

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 [169].

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 (say according to Table 6.3, EN 10143 or EN 1993-1-3), 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 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 from 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 from 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 EN10346 section 7 shall be applied. The film thickness of coil coated metals may be assessed according to EN 13523-1 [177].

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 [178] and [179].

A.11 Assessment for Class C reclaimed steel

A.11.1 Hot rolled and hollow sections products

For the cases where CE marking is not mandatory, conservative assumptions about the material properties may be used for the analysis and design. The conservative material properties provided in Table A. 15 may be assumed.

Table A. 15 **Recommended material properties for non-tested structural steel**

Material	Period of erection	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	after 1970	235	360	81000	210000	15+	0.30	7850	10
Steel – Welds	after 1970	–	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 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. 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 (Class C steel), 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 [60]). 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 the derivation of the 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 6.3.5 (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 [135]. The partial factor γ_m is obtained from Clause 6.3.3:

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

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 C7(6) 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 C7(3) suggests a value of 0.8;
β is the target reliability index, which is taken here as 4.3, corresponding to a class CC3 structure (only for instability verifications);
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

Based on the findings from the *SAFEBRICTILE* RFCS-funded project [173], which are likely to be included in the new Informative Annex E in revised part 1.1 of EN 1993 [180], and from Eqs. (B.2) and (B.3), the values for $\gamma_{m,ad}$ adjusted to a different target reliability index of 4.3, corresponding to a CC3 structure, are obtained. Data is tabulated in Table B. 1.

If the partial factor for model uncertainty and geometric deviation takes the maximum value of 1.15, as the expected geometry deviations for reused steel members may be a concern for stability verifications, then the modified partial factor $\gamma_{M1,mod}$ is calculated, see Table B. 2. This coefficient is defined as follows:

$$\gamma_{M1,mod} = K_{\gamma M1}\, \gamma_{M1} \tag{B.4}$$

where $K_{\gamma M1}$ is a correction factor

From these results, it seems reasonable to assume a value of 1.15 for all steel grades. To cater for the country-specific values of partial factor γ_{M1} the proposal is to adopt this correction in terms of the correction factor, rather than the partial factor itself:

$$K_{\gamma M1} = 1.15 \tag{B.5}$$

Table B. 1 **Partial factors $\gamma_{m,ad}$ for yield strength f_y for $\beta = 4.3$**

Steel grade	Mean value	Coefficient of variation	$f_{y,d}$	$\gamma_{m,ad}$
S235, S275	$1.25f_{y,nom}$	0.055	$1.01f_{y,nom}$	0.99
S355, S420	$1.20f_{y,nom}$	0.050	$0.99f_{y,nom}$	1.01
S460	$1.15f_{y,nom}$	0.045	$0.97f_{y,nom}$	1.03

Table B. 2 **Partial factors $\gamma_{M1,mod}$ for designs with reclaimed steel**

Steel grade	$\gamma_{m,ad}$	$\gamma_{M1,mod}$
S235, S275	0.99	1.14
S355, S420	1.01	1.16
S460	1.03	1.18

Using reclaimed structural steel on a project is an effective strategy to reduce the environmental impact of a building by eliminating the energy required to recycle scrapped steel into new structural sections. This practice cannot be generalised to all structural steel though, as not all components can be effectively reused. The RFCS-funded project PROGRESS (PROvisions for Greater REuse of Steel Structures) focused on single-storey buildings, and within this project, the reuse scenarios were broadly divided into three categories: (i) entire primary structure, (ii) components (elements of the primary structure), and (iii) individual members, and may involve or not relocation.

This project was funded by the Research Fund for Coal and Steel under Grant Agreement 747847

Research Fund for Coal & Steel