Conceptual design and design examples for multi-storey buildings

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Background and Applications

Overview

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- **Client Guide**

- **Case Studies**

- **Scheme Development**

- **Flow Charts**

- **NCCI**

- **Examples**

Client briefing: guides, case studies and concept designs demonstrate the value that steel offers for economic, fast, adaptable, safe, construction

1. Introduction

Commercial buildings, such as **offices, shops** and **mixed residential-commercial buildings**, account for **20% of construction output in the EU**, representing over **20 million square metres of floor space per year**. The commercial sector demands buildings that are **rapid to construct**, of **high quality, flexible and adaptable in application**, and **energy efficient in use**.
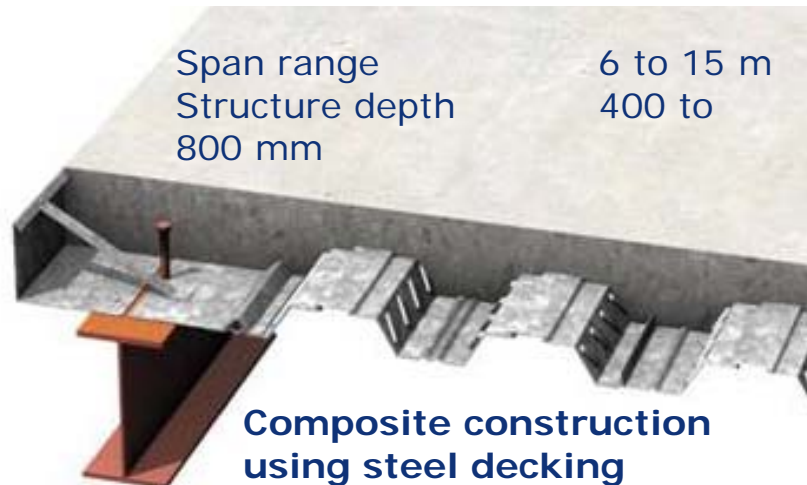
Steel and composite construction has achieved over 60% market share in this sector in some countries of Europe where the **benefits of long spans, speed of construction, improved quality** and **reduced environmental impact** have been recognised.

A wide range of steel technologies may be used in commercial buildings. Specific technologies are chosen to match client requirements. Many **long span steel solutions** provide **service integration** within their depth, which saves on building height. Cellular beams combine an efficient manufacturing process with opportunities for service integration. Integrated beams minimise the floor zone while allowing services uninterrupted access below the floor soffit.

2. Maximum Value from Steel Construction

- Speed of Construction
- Flexibility and adaptability
- Service integration
- Reduced disruption to the locality
- Quality
- Safer Construction
- Environmental Benefits

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3. Construction Systems and their Ranges of Application

Span range   6 to 15 m
Structure depth   400 to 800 mm

Composite construction using steel decking

Cellular beams in composite construction

Span range 9 to 18 m
Structure depth   600 to 1000 mm

Span range 9 to 20 m
Structure depth   600 to 1200 mm

Fabricated or rolled beams with large web openings

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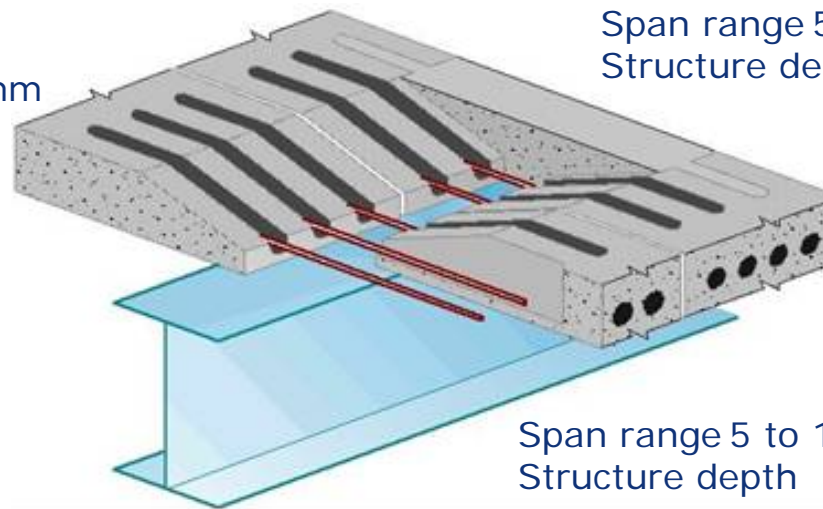
Integrated beams with deep decking

Span range 5 to 9 m
Structure depth   300 to 350 mm

Integrated beams supporting precast concrete slabs

Span range 5 to 9 m
Structure depth   300 to 400 mm

Span range 5 to 10 m
Structure depth   500 to 900 mm

Steel beams supporting precast concrete slabs

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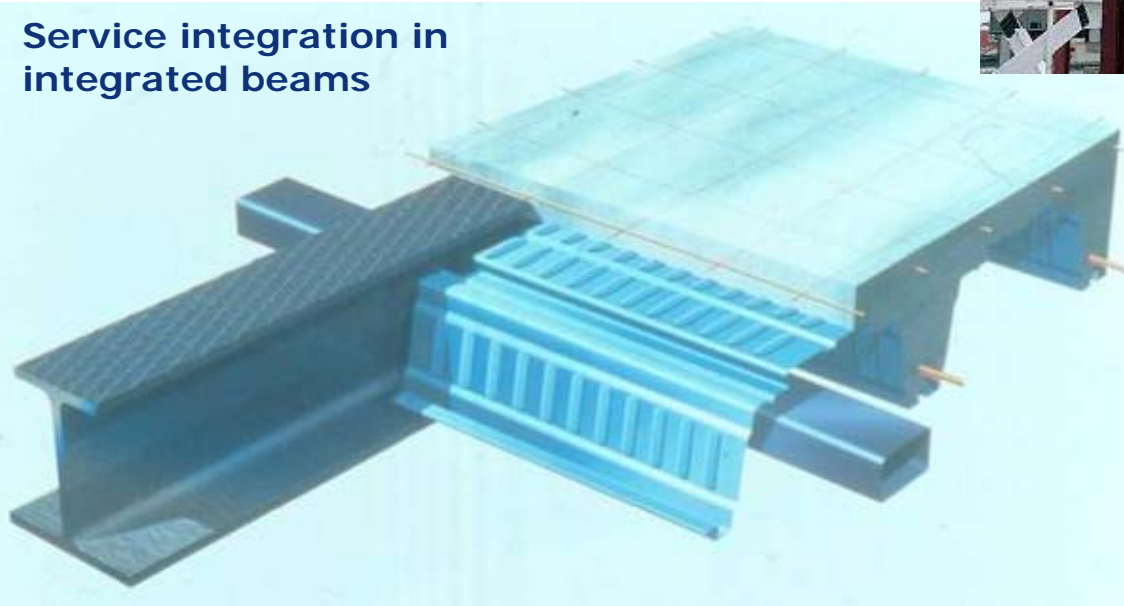# 4. Specific Technologies for Commercial Buildings

Commercial buildings require a range of specific technologies that have been developed to meet client needs:

Service Integration:

Cellular beams with regular openings for services

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Service integration in integrated beams

Fabricated beams with variety of shapes of openings

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Fire Resistance

Tubular columns exposed for fire resistance

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Energy Efficiency

Double façade in the Kone Building, Finland

Energy efficient design at GLA building, London

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Cladding and infill walls

- large variety of cladding systems: traditional brickwork, curtain walling and glazing
- Light steel infill walls as sub-structure to all types of facades
- Double skin glazed façades to reduce solar gain
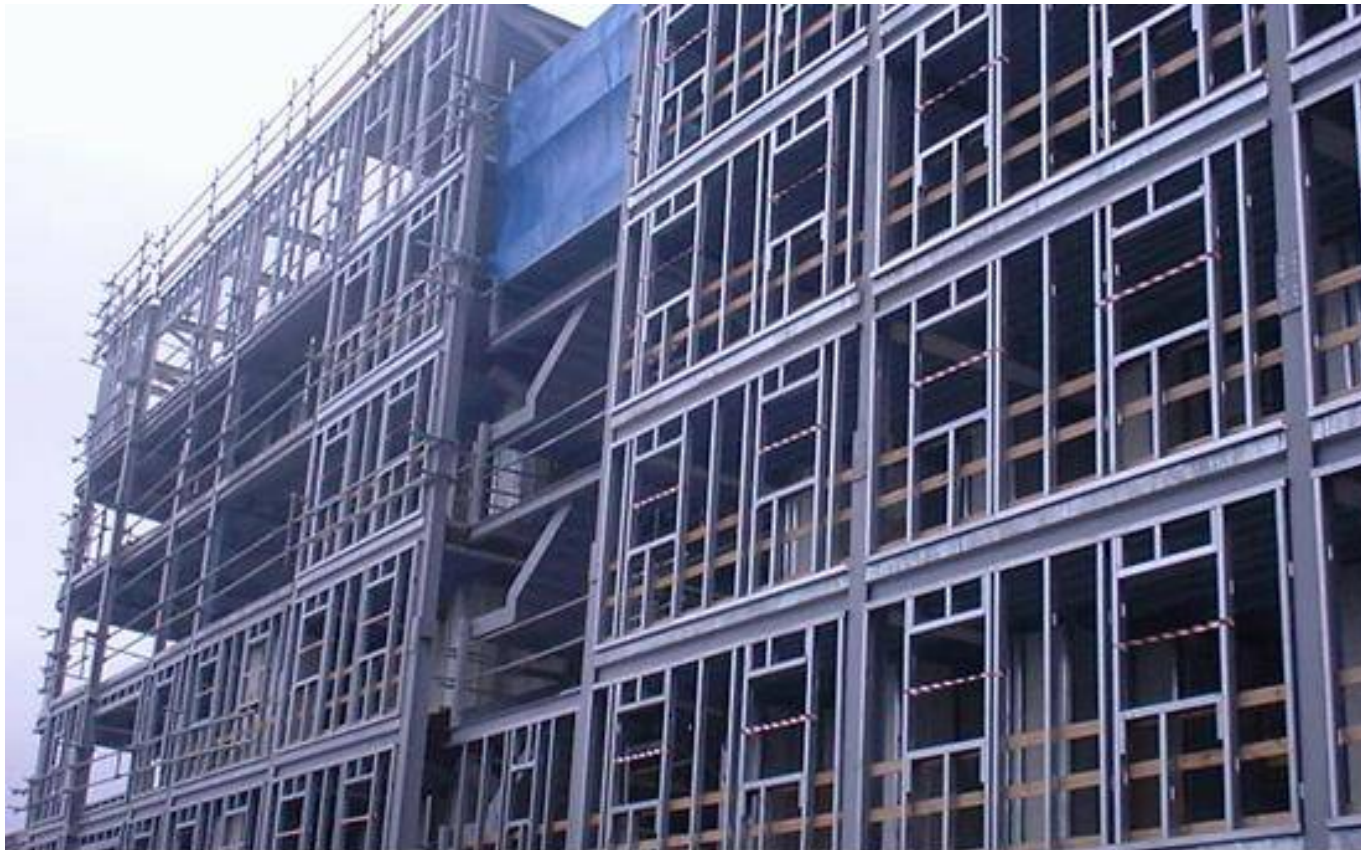
Light steel infill walls used in steel framed buildings

Access Steel highlights the benefits of steel as the primary construction material through detailed short case studies of successful buildings

- Le Sequana, Paris
- Kista Science Tower, Stockholm
- Place d'Iena, Paris
- Office Building, Palestra, London
- Luxembourg Chamber of Commerce's
- ING Headquarters, Amsterdam
- Sheraton Hotel, Bilbao

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Le Sequana – artist's impression

near completation

- 25,000 m² of exceptional quality office accommodation in the heart of the new commercial centre, Ville de Paris
- Column free spaces of 18 m x 36 m overlooking the Seine
- 33% pre-let from plan, prior to the commencement of construction
- Fully air conditioned with individual control for every 12m²
- Construction completed in 22 months to programme and budget, including the erection of 2000 tonnes of structural steelwork in 12 weeks

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Kista Science Tower, Stockholm

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Kista Science Tower:
42000 m² of high quality office space in six buildings, 158 meters tall with 32 storeys.
Long span steel trusses create open space overlooking northern Stockholm.

Completed building showing the new façade

Demolition stage of the original concrete frame

- Refurbishment of a building constructed in the 1960's to create large open spaces for a high quality office building in one of the most prestigious districts in Paris
- The project involved preservation of the existing façades of the building, which imposed a constraint on the method of building
- Creation of office open spaces of 1200 m² without intermediate columns
- Floor span of up to 14 m
- Service integration in cellular beams achieves minimum floor zone
- Construction completed in 22 months

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Office Building Palestra, London

During construction, showing steel framework and fully glazed facade

- 12 storey building of 31,5 to 36 m width and 90 m length with a 9 m projection of the upper 4 floors
- Pairs of cellular beams were placed either side of the 4 inclined tubular columns. The cellular beams were designed as continuous over a span of 15 m in order to reduce their depth
- The floor-floor zone is a minimum of 3,65 m and the structure and services zone is only 850 mm. The cellular beams permit passage of 400 mm diameter circular service ducts
- Fully glazed façade, which was directly attached to the composite slab
- The inclined tubular columns and shear connections were designed for an eccentric load transfer which carried bending in the columns. The columns used innovative double circular hollow sections filled with cement grout to achieve 120 minutes fire resistance
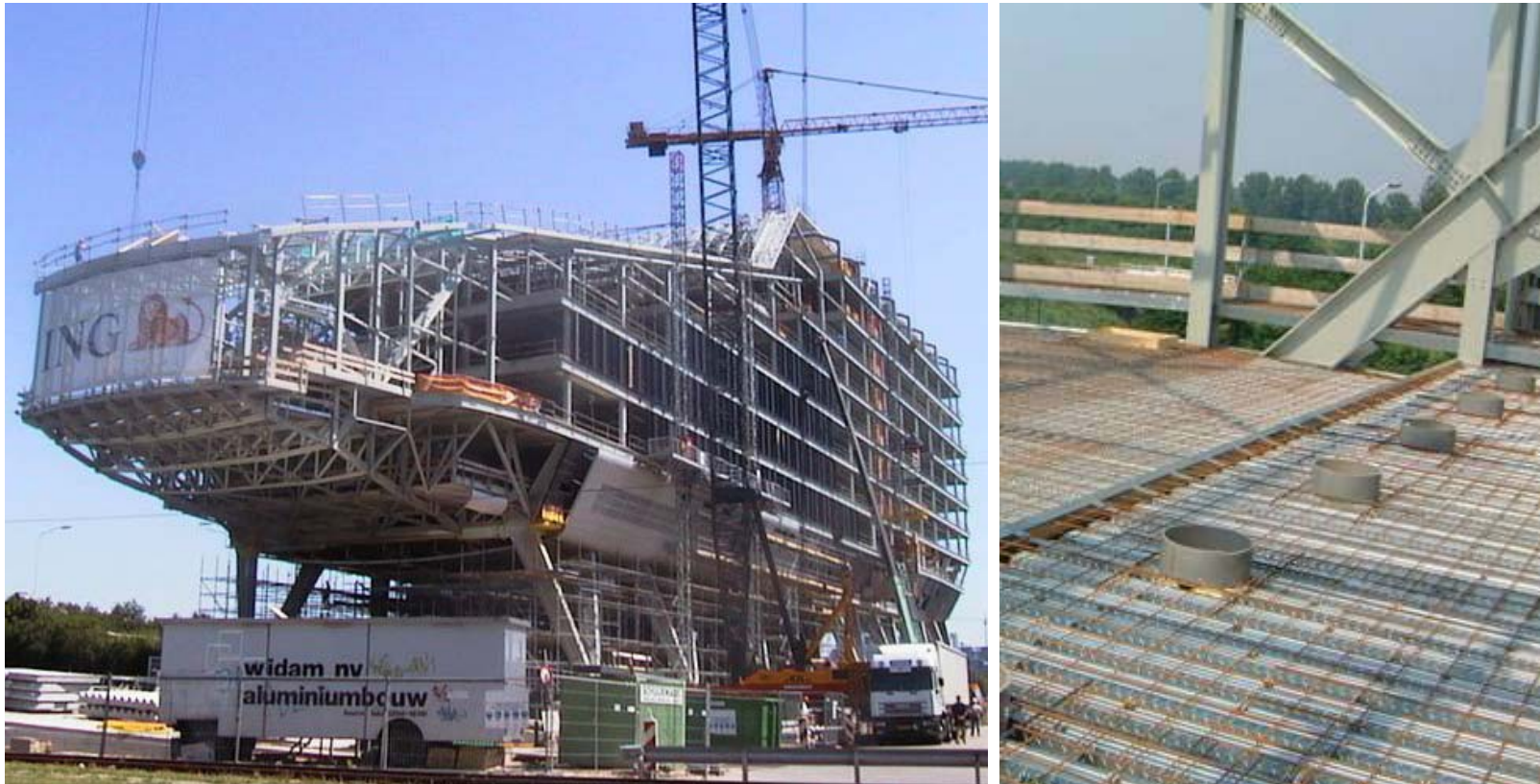- Erection of the 3500 tonnes of steelwork in only 10 months

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Luxembourg Chamber of Commerce´s

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Chamber of Commerce´s

- 20 000 m² of new 5 storey office space, a conference centre of 8 000 m² and 650 underground car parking spaces on 4 below ground levels.
- Full double-glazed ventilated façade with shading with serigraphed glass shields.
- Water-cooled composite slab using sinusoidal stainless steel façade, which acted as framework to the concrete slab of 300 mm depth. No temporary propping was required.
- Integrated IFB beams with under-tied hollow sections to create a span of 12,5 m, which is 30% larger than is possible with integrated beams. These beams are exposed usually.
- Fire engineering, using natural fire approach, led to a fully unprotected steel structure.
- These operating conditions for the water-cooled slab are:
  Summer-night-time; Summer-day-time and winter.
  Heating and cooling is provided
- Diaphragm action of composite slab and stability through the lift shaft.

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ING Headquaters, Amsterdam

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- 9 storey steel structure of 20 000 m² floor area supported on inclined columns.
- The 'nose' of the building provides an auditorium and cantilevers 26 m from the adjoining structure.
- A shallow construction depth of 310 mm was achieved using Slimdek for a structural grid of approximately 7 m. Slimdek consists of asymmetric steel beams (ASB) support a deep composite slab.
- The inclined columns support the entire weight of the building and provide overall stability. They are designed as composite sections.

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Composite floor slab in the auditorium area showing service penetrations

- The space between the inclined legs provides access to the building and to the below-ground car parking.
- A double skin glass façade was used to control 'solar gain' and regulate internal temperatures.
- Minimum ground works were required by concentrating loads at discrete points.
- A fire resistance of 90 minutes was satisfied by a fire engineering analysis.
- Slimdek minimised the visual impact of the structure, which was important for the architectural concept.
- Steelwork was supplied by a consortium of 3 fabricators. Erection of the 1 000 tonnes of steelwork took only 28 weeks.

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Sheraton Hotel, Bilbao

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- 20 000 m² multi-storey luxury hotel in Bilbao: 11 above-ground storeys of 13 000 m² floor area: and 4 below ground floors of 7 000 m² floor area.
- 211 rooms: one presidential suite, 20 suites and 190 double rooms.
- 1 100 tonnes of structural steelwork were used in the form of I and H sections, tubes and plates.
- Only 7 months for the erection of the steel structure.
- The Sheraton hotel has a high level of building services comprising:
  - Full air conditioning system in all rooms and in common areas, and radiant flooring in the atrium.
  - Architectural design for optimised energy requirements, such as natural cooling, solar gains or natural daylight.
  - Emergency electric power plant for essential facilities, such as the fire protection, safety and UPS-supported computer network installations.
  - Optical fibre voice and data networks to all rooms.
  - Fire detection and extinguishing network, backed up by a computerized management system that enables the fire to be located immediately.
  - Advanced Smoke Control systems and safe means of egress in fire.

16 documents

Concise information on proposal development guides the architect and engineer through all the decisions that have to be made to develop a best practice design

- Form and function
- Key information
- Location and its influence on the design of MSB
- Service strategies – Overview
- Service buildings
- Sustainability – Overview
- Sustainability factors
- Fire Safety
- Structural System
- Composite Slabs, Web Openings etc.
- Vertical Structure

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Coordination of structural and architectural design for multi-storey buildings with steel frames

- Floor Grids

- Influence of Building Height

- Horizontal Coordination

- Vertical Coordination: Floor-to-Floor Heights

- Vertical Coordination: Structural Zone

- Vertical Coordination: Service Zones

- Horizontal Structural Options

- Span Range of Structural Systems

- Benefits of Long Span Construction
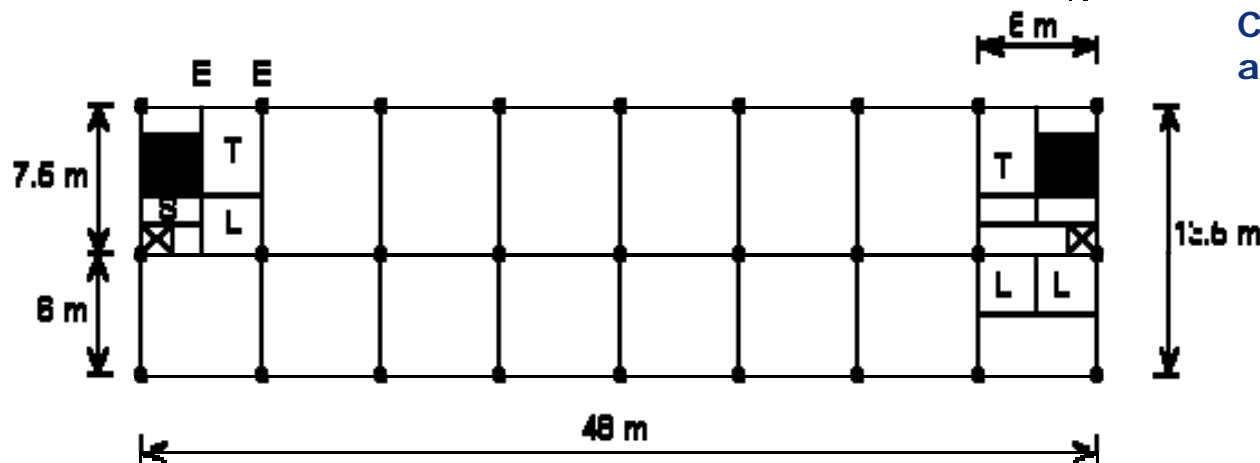
- Approximate Steel Quantities

Floor Grids

Influenced by:

- Planning Grid

- Column Spacing

- Use of internal space

- Requirements of Building Service Integration
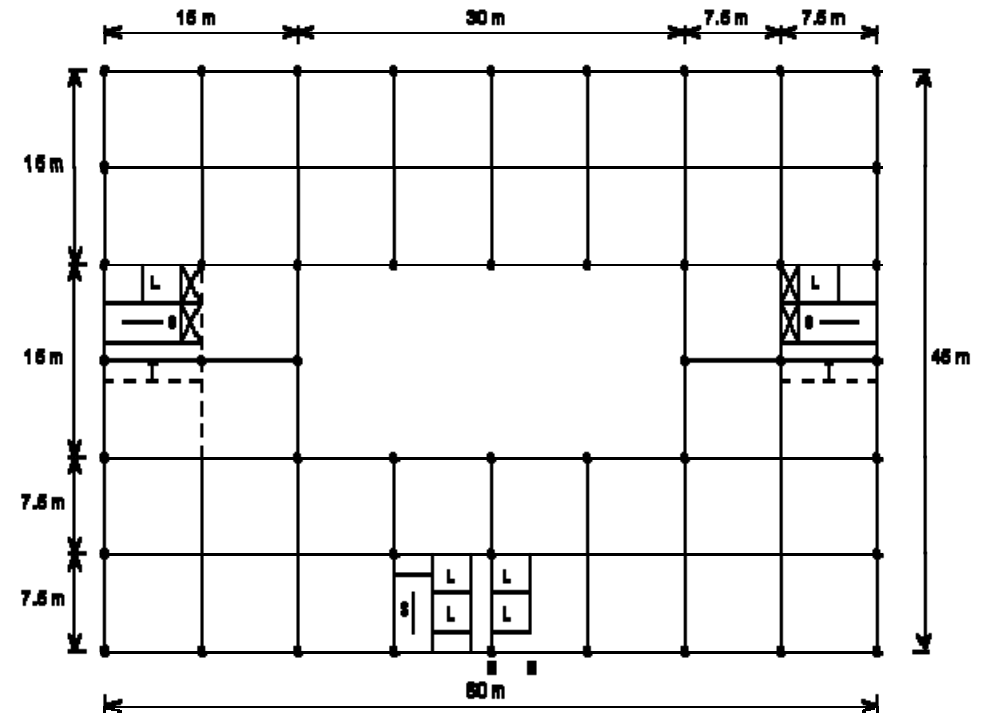
Column grid for long span floor in a prestige air-conditioned office

Column grid for a naturally ventilated office

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Building Height

The building height has a strong influence on the:

- Structural system that is adopted.
- Foundation system.
- Fire resistance requirements and means of escape.
- Access (by lifts) and circulation space.
- Cladding system.
- Speed of construction and site productivity.

The **structural system** is primarily influenced by the means of **stabilizing the building**. For buildings **up to 8 storeys** high, **vertical bracing** is preferred, but for **taller buildings**, strategically placed **concrete or braced steel cores** are usually adopted. For ultra-tall buildings, many types of external bracing systems have been used. These are outside the scope of this information.
Sizes of lifts and their speed of movement also become important considerations for tall buildings. Depending on the Regulations for fire safety in the particular country, the use of sprinklers may be required or buildings of more than 8 storeys.

Horizontal coordination / Position of cores

Positioning of cores is influenced by:

- Horizontal distribution systems for mechanical services.
- Fire rating requirements, which may lead to shortened evacuation routes, and to reduced compartment sizes.
- The need to distribute the structural stabilizing systems more-or-less symmetrically throughout the building plan.
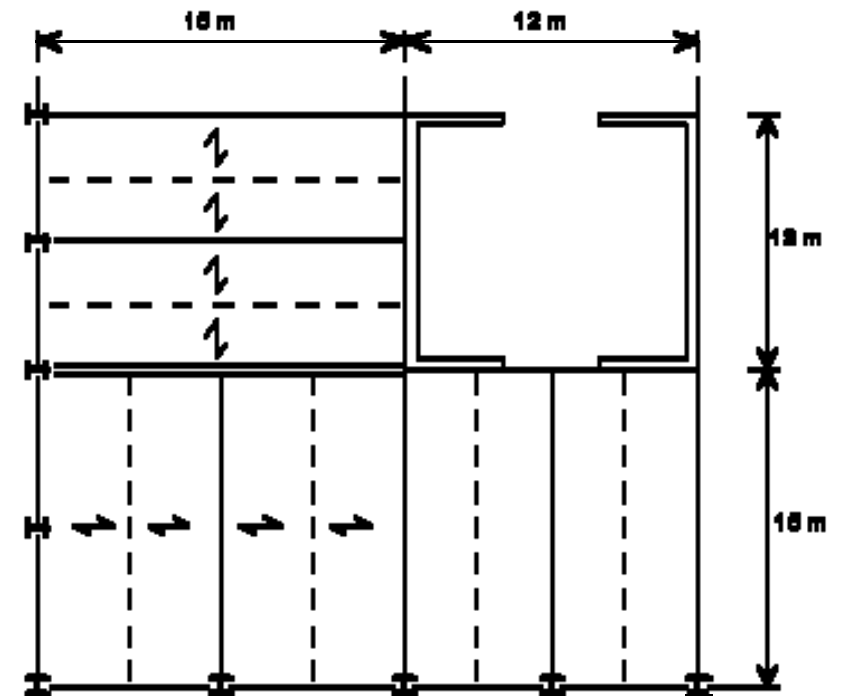
Beam layout around a concrete core

Vertical coordination / Floor-to-Floor Heights

Prestige office	3,8 – 4,2 m
Speculative office	3,5 – 4,0 m
Renovation project	3,5 – 3,9 m

Vertical coordination / Structural Zone

Composite beams	Span/depth ratio ≤ 25
Cellular beams	Span/depth ratio ≤ 25
Fabricated primary beams	Span/depth ratio ≤ 20
Slim floor or integrated beams	Span/depth ratio ≤ 25

Vertical coordination / Service Zones

Raised access floor	150 to 200 mm
Air-conditioning units	400 to 500 mm
Ceiling and lighting	120 to 250 mm

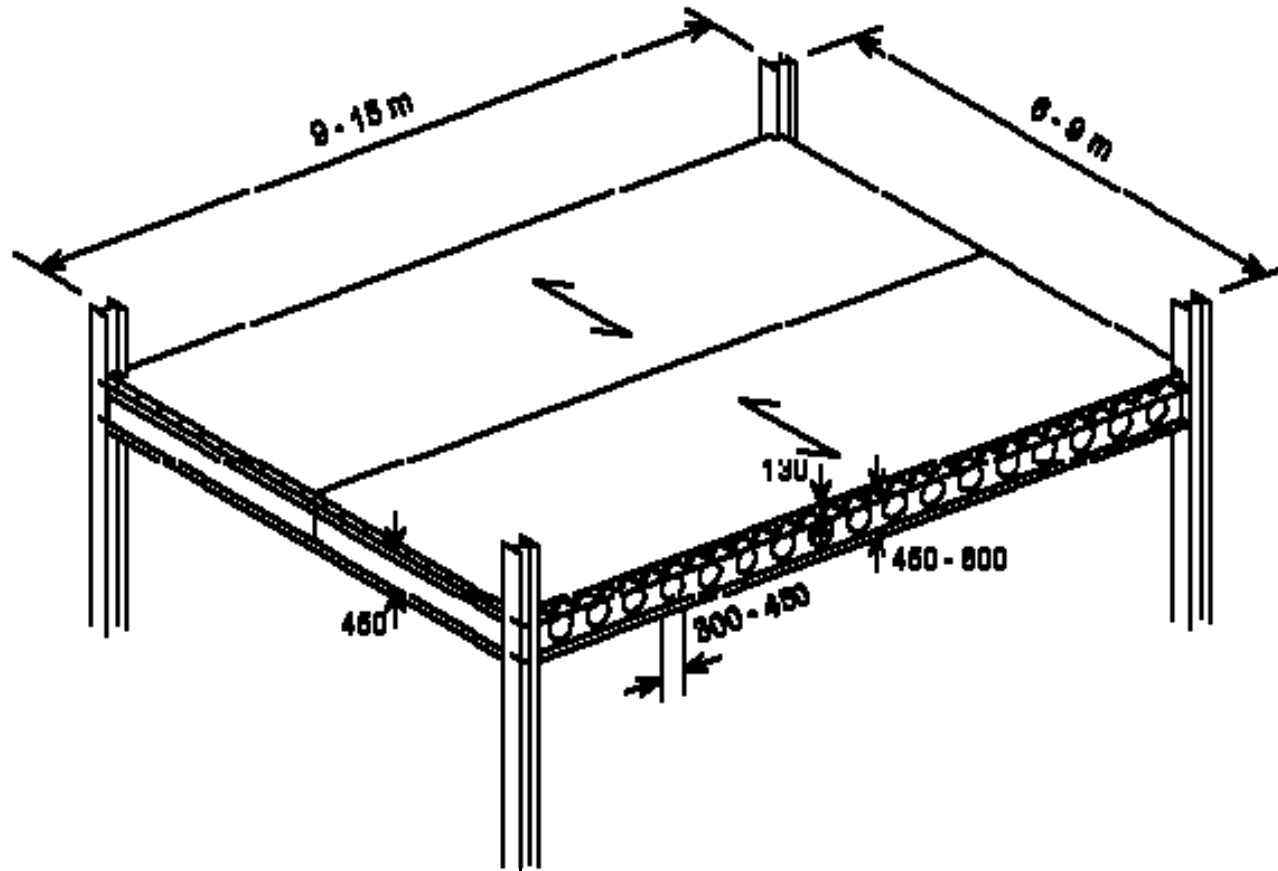# Horizontal Structural Options

Composite beams (with composite slabs)   **Spans of 6 to 13 m**

Non-composite beams (with precast planks)   **Spans of 6 to 9 m**

Partially encased composite beams   **Spans of 6 to 12 m**

Cellular or fabricated beams (with composite slabs)   **Spans of 8 to 18 m**

Slim floor or integrated beams   **Spans of 5 to 9 m**
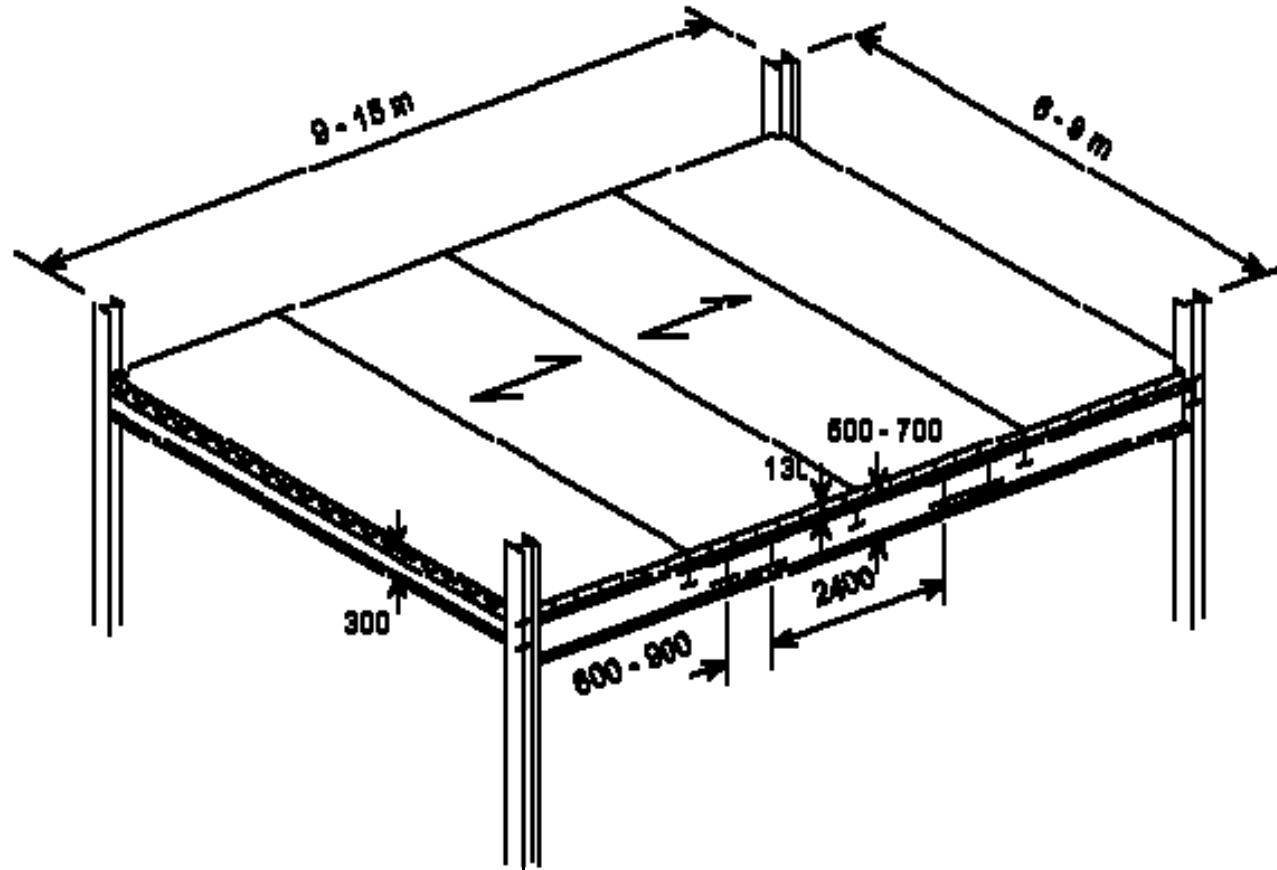
Forms of construction used in floors
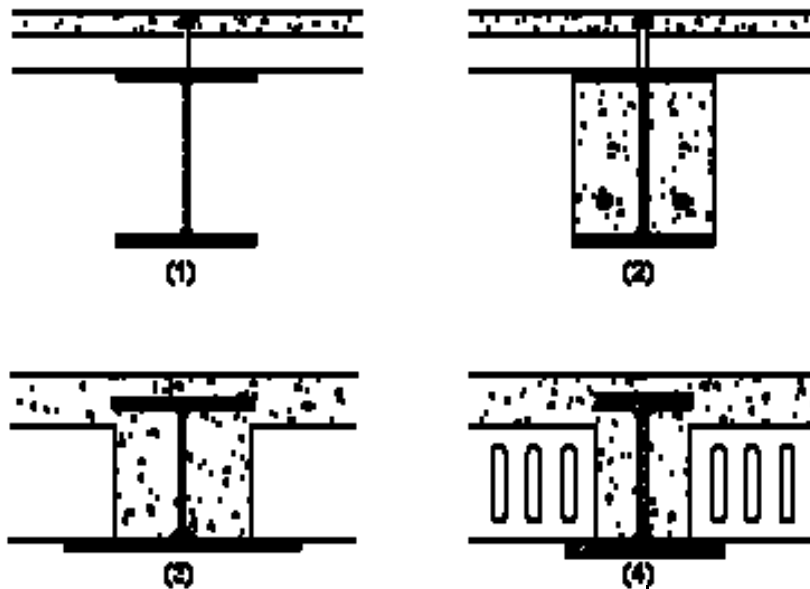
1. Composite beam
2. Partially encased beam
3. Integrated beam
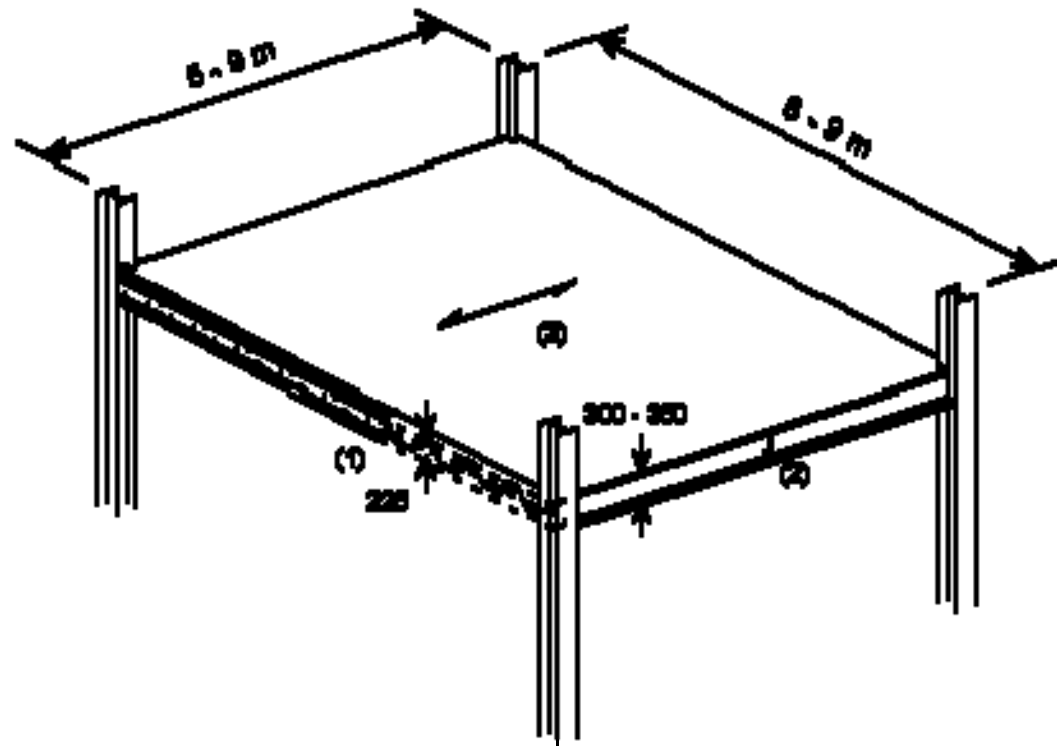4. Slim floor beam

Horizontal Structural Options

Typical long span secondary beams

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Horizontal Structural Options

9 - 15 m
6 - 9 m
500 - 700
130
300
600 - 900
2400

Typical long span primary beams and shorter span secondary beams

Horizontal Structural Options

Integrated beams or slim floor

Span Range of various structural systems

	Span (m)					
	6	8	10	13	16	20
Reinforced concrete flat slab	▬▬					
Slim floor beams and deep composite slab	▬▬▬					
Integrated beams with precast slabs	▬▬▬▬					
R.C. beams and slab		▬▬▬				
Post-tensioned concrete flat slab			▬▬▬			
Composite beams and slab		▬▬▬▬▬▬				
Fabricated beams with web openings			▬▬▬▬▬▬▬			
Cellular composite beams			▬▬▬▬▬▬			
Composite trusses					▬▬▬	

Benefits of Long Span Construction

Long span beams have gained in popularity in the commercial building sector because they offer the following benefits in design and construction:

- Internal columns are eliminated, leading to more flexible and efficient use of internal space.
- Services can be integrated within the depth of the structure, and so the floor-to-floor depth is not increased.
- Fewer components are required (typically 30% fewer beams) leading to reduced construction and installation time.
- Fire protection costs can be reduced due to the massivity of the longer span members.
- For cellular beams, multiple circular ducts for services are cheaper than rectangular ducts.
- Steelwork costs are not increased significantly, despite the longer spans.
- Overall building costs are increased by a negligible amount (less than 1%).

Approximate Steel Quantities

Form of Building	Approximate steel quantities (kg/m² floor area)			
	Beams	Columns	Bracing	Total
3 or 4 storey building of rectangular form	25–30	8–10	2–3	35–40
6–8 storey building of rectangular form	25–30	12–15	3–5	40–50
8–10 storey building with long spans	35–40	12–15	3–5	50–60
20 storey building with a concrete core	40–50	10–13	1–2	55–65
20 storey building with a braced steel core	40–50	20–25	8–10	70–85

21 documents

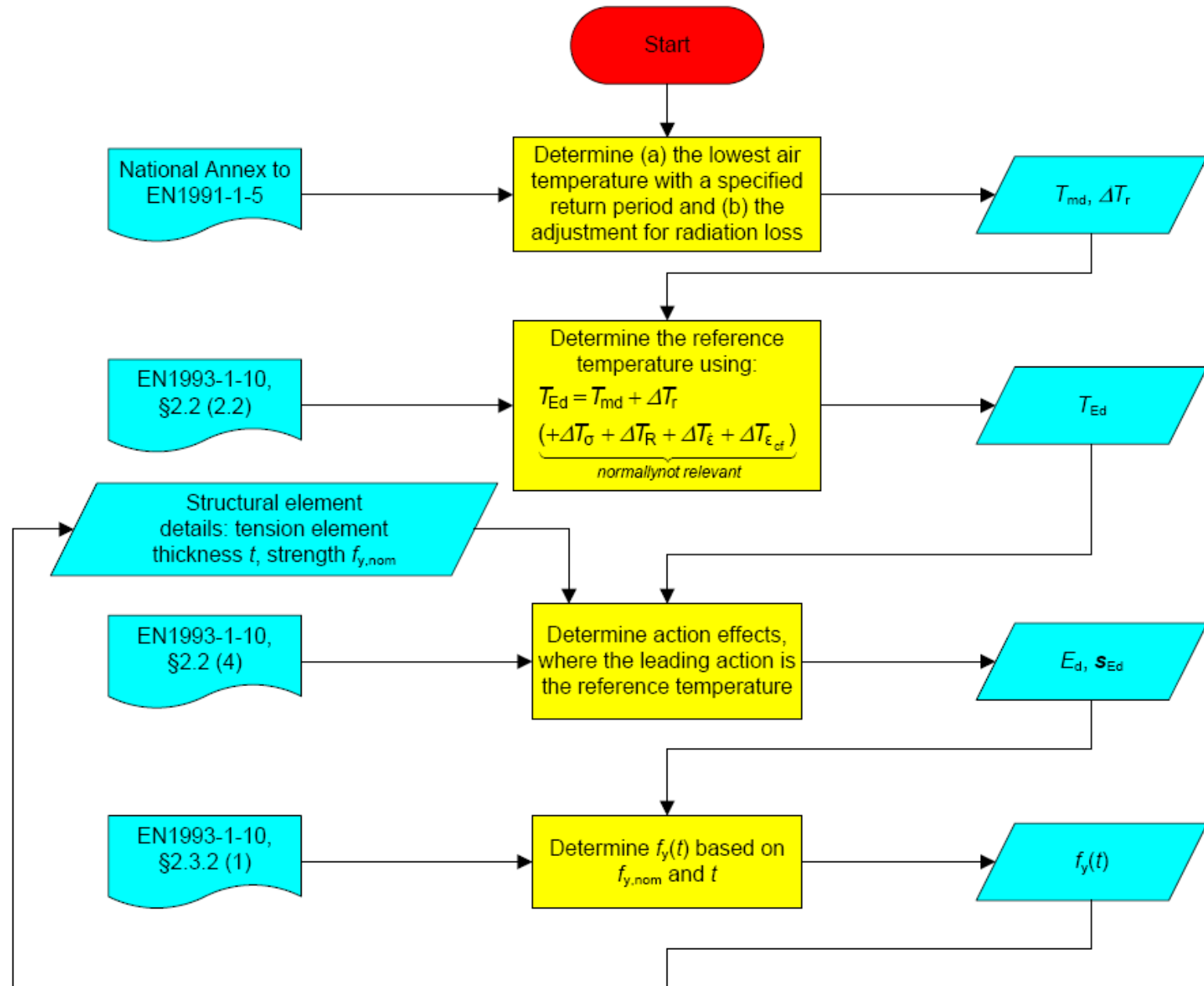**Each design activity is described separately by a flow chart:**

- Evaluation of wind loads
- Governing load combinations
- Frame Analysis
- Choosing a steel sub-grade
- Column design
- Non composite beams / Composite Beams
- Design of vertical bracings
- Simple Connections
- Secondary beam design

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Flow Charts

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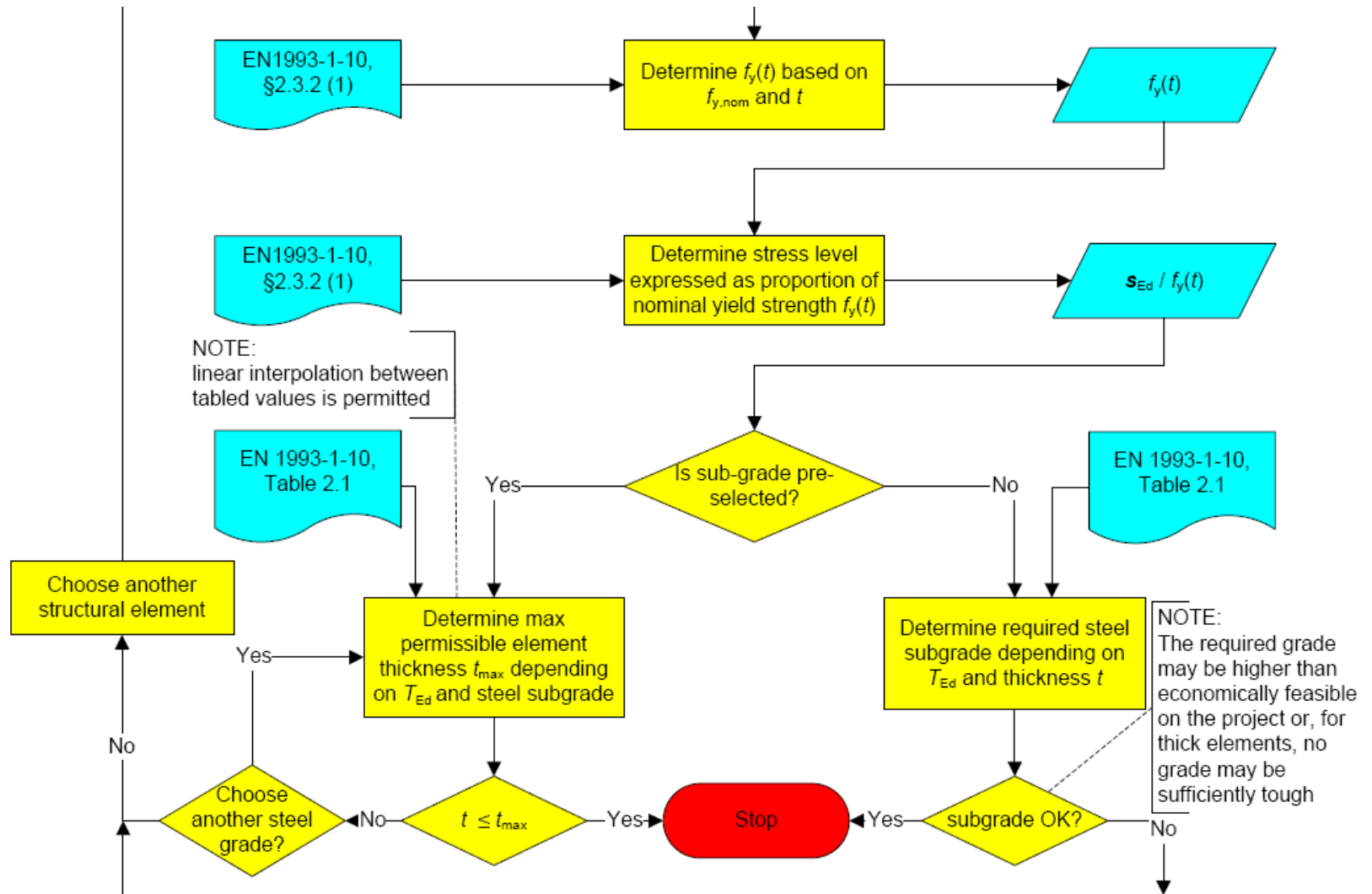Leichtmetallbau

21 documents

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- Evaluation of wind loads
- Governing load combinations
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- **Choosing a steel sub-grade**
- **Column design**
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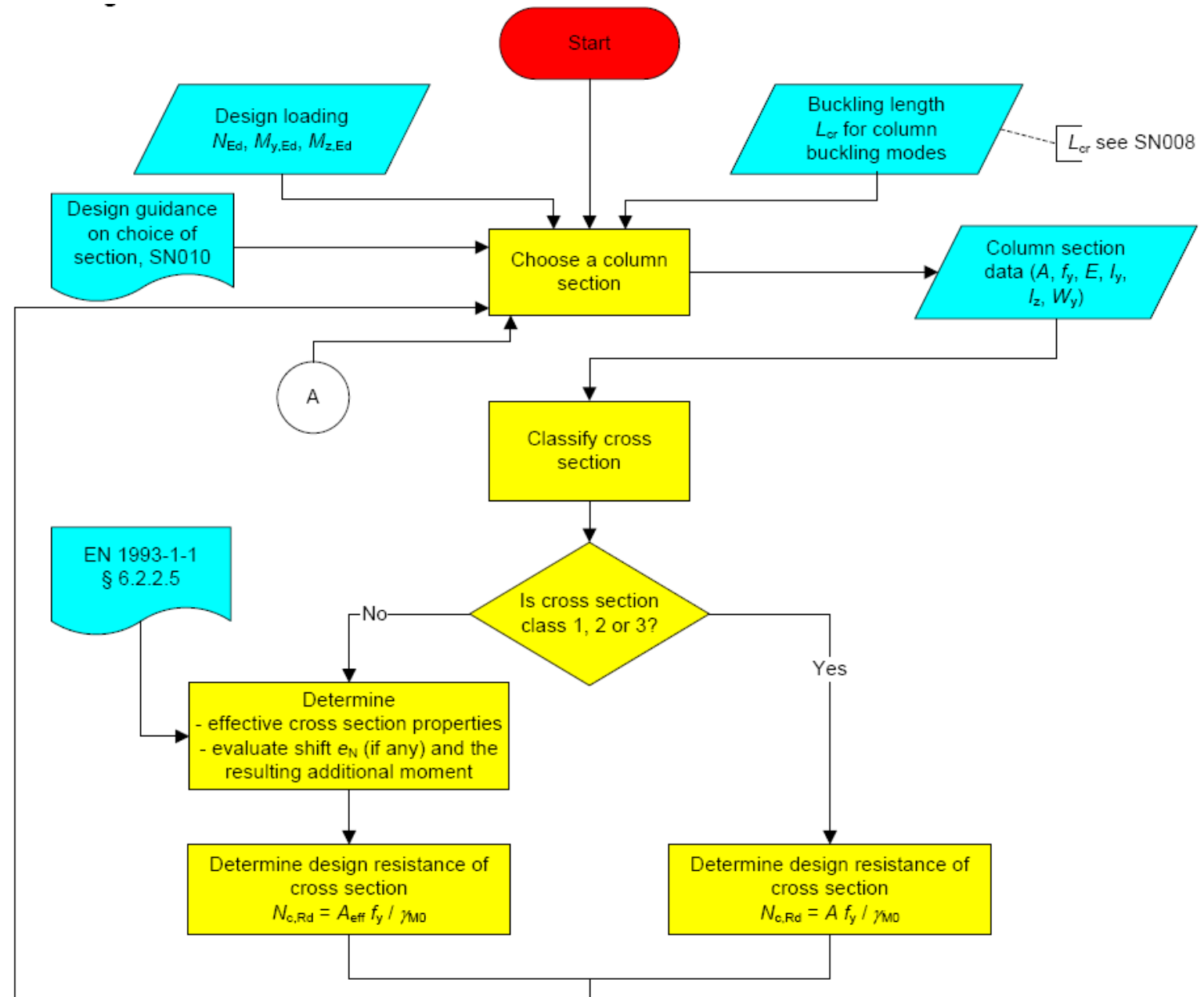- Simple Connections
- Secondary beam design

Choosing a steel subgrade

Start

National Annex to EN1991-1-5 → Determine (a) the lowest air temperature with a specified return period and (b) the adjustment for radiation loss → T_{md}, ΔT_r

EN1993-1-10, §2.2 (2.2) → Determine the reference temperature using:
$$T_{Ed} = T_{md} + \Delta T_r$$
$$\underbrace{(+\Delta T_\sigma + \Delta T_R + \Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon_{cf}})}_{normally\,not\,relevant}$$
→ T_{Ed}

Structural element details: tension element thickness t, strength $f_{y,nom}$

EN1993-1-10, §2.2 (4) → Determine action effects, where the leading action is the reference temperature → E_d, s_{Ed}

EN1993-1-10, §2.3.2 (1) → Determine $f_y(t)$ based on $f_{y,nom}$ and t → $f_y(t)$

Choosing a steel subgrade

Flow Charts

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Column design

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Column design


```
                                              ┌─────────────┐
                              ┌──── No ────────┤ N_Ed ≤ N_c,Rd │
                              │                └─────────────┘
                              │                      │ Yes
                              ▼                       ▼
┌──────────────┐    ┌──────────────────────┐    ┌──────────────────────┐
│ EN 1993-1-1  │───▶│ Determine non-       │───▶│ λ̄ for axis that gives │
│ § 6.3.1.2    │    │ dimensional          │    │ minimum value of χ    │
└──────────────┘    │ slenderness λ̄ for    │    └──────────────────────┘
                    │ flexural buckling    │
                    │ for both axes        │
                    └──────────────────────┘
```

$N_{Ed} \leq N_{c,Rd}$

EN 1993-1-1 § 6.3.1.2

Determine non-dimensional slenderness $\bar{\lambda}$ for flexural buckling for both axes

$\bar{\lambda}$ for axis that gives minimum value of χ

EN 1991-1-1 § 6.3.1.4 (2)

Determine non-dimensional slenderness λ_T for torsional and torsional-flexural buckling

$\bar{\lambda}_T$

NOTE: in most multi-storey building columns this slenderness is less than for flexural buckling and therefor does not govern

Page 2

Column design

Column design

Column design – Determination of interaction criteria for both axes

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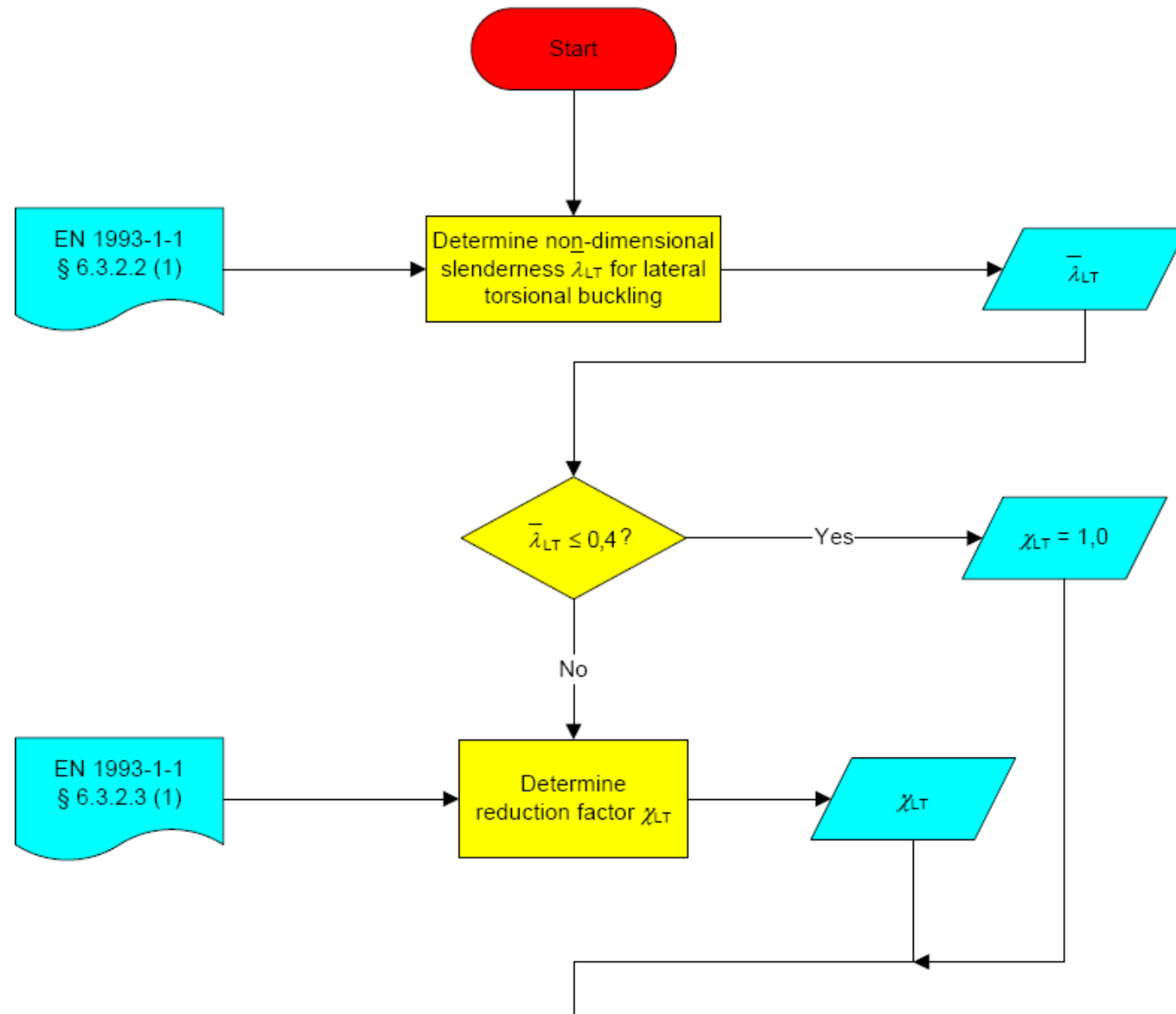
Column design – Determination of interaction criteria for both axes
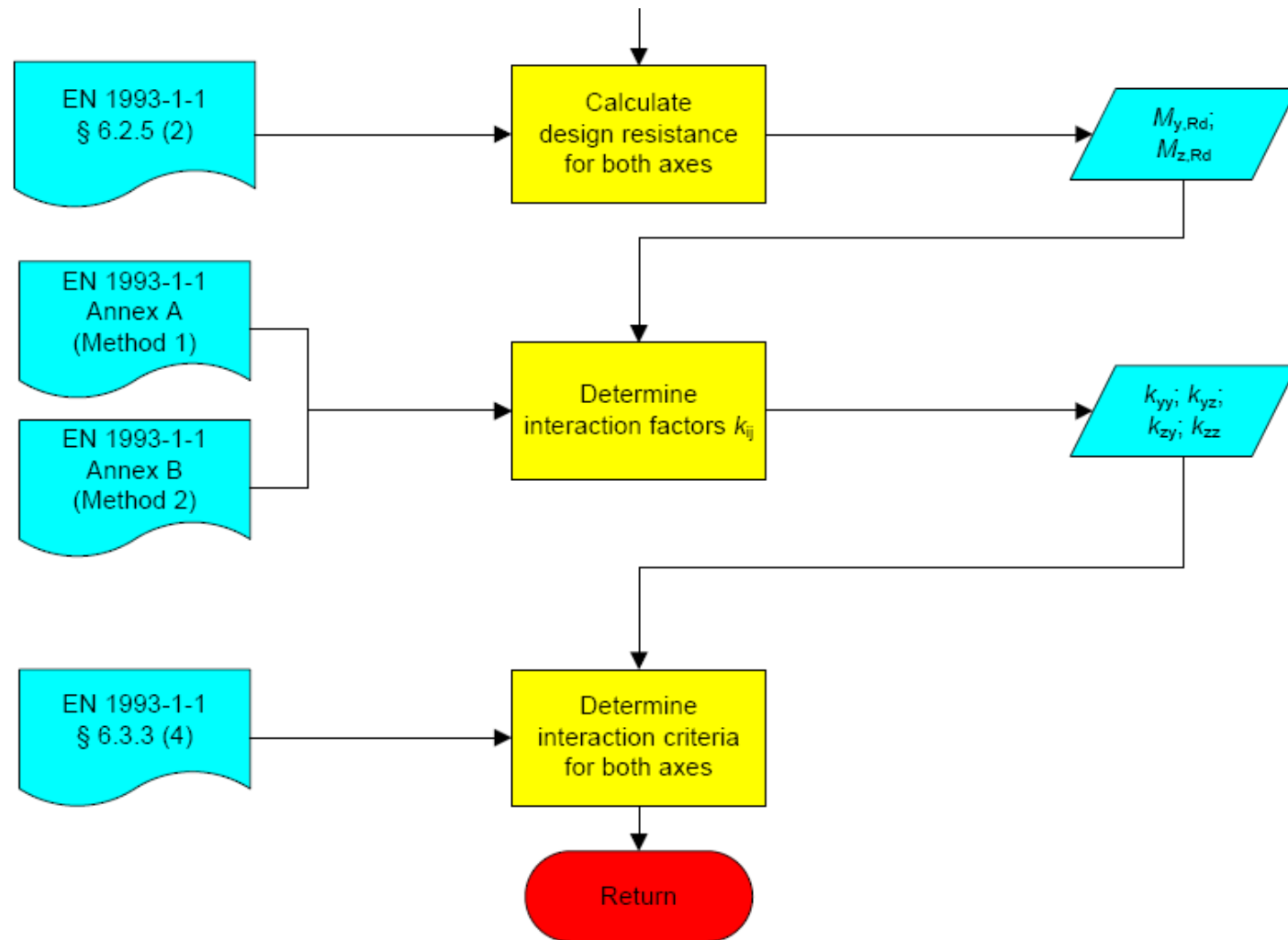
34 documents

Non-contradictory, complementary information (NCCI) is presented that addresses all the information that the Eurocodes do not cover that is essential for design, e.g.:

- Calculation of α_{crit}
- Vertical and horizontal deflection limits
- Effective lengths of columns
- Torsional Buckling (calculation of $N_{cr,T}$ and $M_{cr,T}$)
- Design Rules for Web Openings in Beams
- Torsion (Section Properties and stresses)
- Vibrations
- Sizing Guidance
- Horizontal and vertical Bracings
- Design Model for simple Column Splices
- Choice of Method for Frame Analysis
- Simplified Bracing Systems

34 documents

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Calculation of α_{cr}

This NCCI sets out the basis for the calculation of α_{cr}, the parameter that measures the stability of the frame.

1. Methods for determining α_{cr}

EN 1993-1-1 §5.2.1 concerns the checking of buildings for sway mode failures and defines the parameter α_{cr} as follows:

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}}$$

in which

F_{Ed} is the design load on the structure

F_{cr} is the elastic critical buckling load for the global instability mode.

For multi-storey buildings, the value of α_{cr} is calculated for each storey in turn and the criterion of expression (5.1) must be satisfied for each storey.

EN 1993-1-1 §5.2.1(4)B states: "α_{cr} may be calculated using the approximate formula (5.2)", which is given as:

$$\alpha_{cr} = \frac{H_{Ed}}{V_{Ed}} \frac{h}{\delta_{H,Ed}}$$

where

H_{Ed} is the (total) design value of the horizontal reaction at the bottom of the storey to the horizontal loads and fictitious horizontal loads

V_{Ed} is the total design vertical load on the structure at the bottom of the storey

$\delta_{H,Ed}$ is the horizontal displacement at the top of the storey, relative to the bottom of the storey (due to the horizontal loads)

h is the storey height

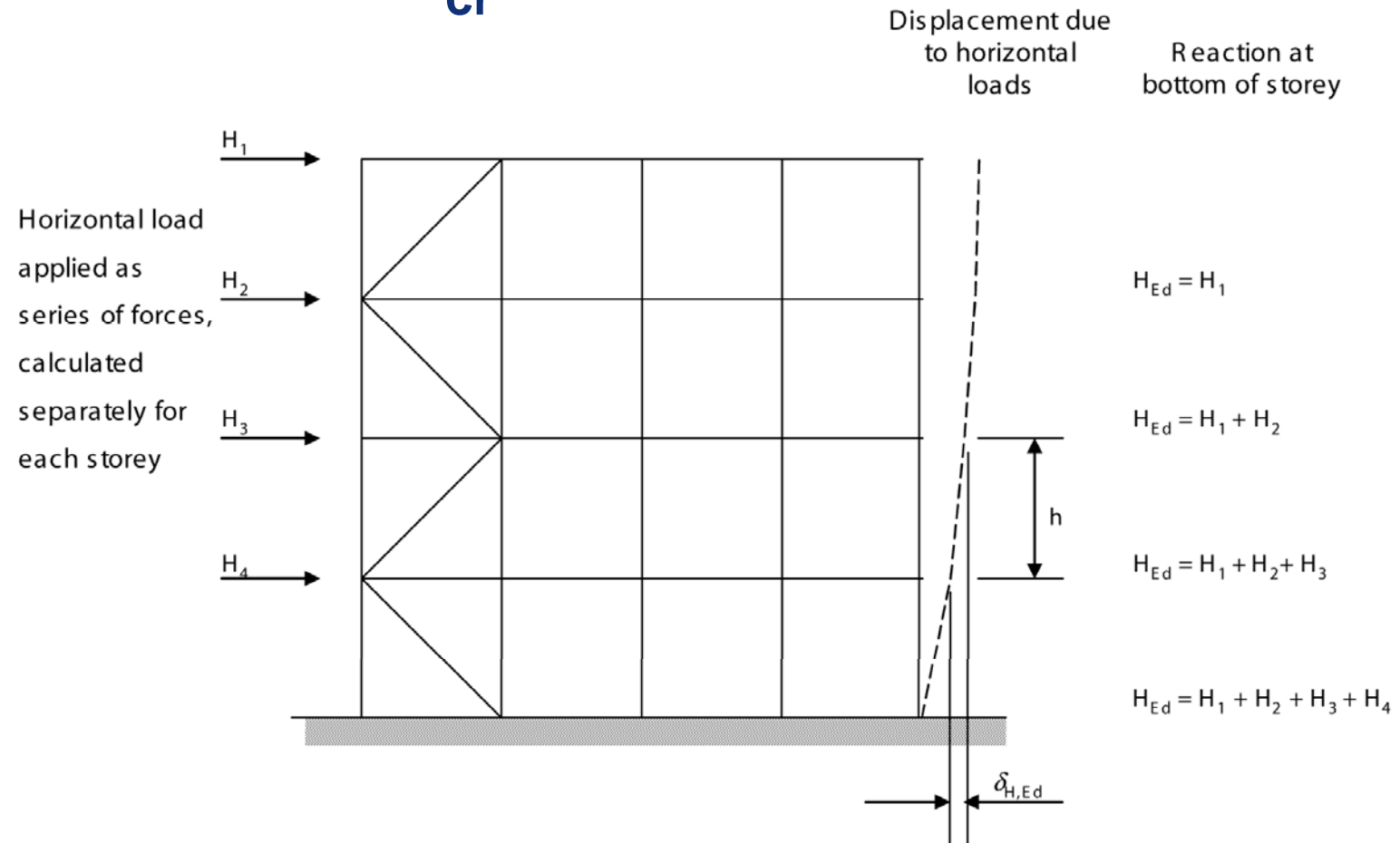# Calculation of α_{cr}

Displacement due to horizontal loads

Reaction at bottom of storey

H_1

Horizontal load applied as series of forces, calculated separately for each storey

H_2

H_3

H_4

$H_{Ed} = H_1$

$H_{Ed} = H_1 + H_2$

h

$H_{Ed} = H_1 + H_2 + H_3$

$H_{Ed} = H_1 + H_2 + H_3 + H_4$

$\delta_{H,Ed}$

Figure 1.1   Displacement of a multi-storey frame due to horizontal loads (deflection parameters for second storey only illustrated)

Calculation of α_{cr}

As an alternative to formula (5.2), in certain cases other checks may be more convenient or more appropriate. The following three alternatives may be considered:

Alternative (1)

Use formula (5.2) with H_{Ed} determined by the fictitious horizontal loads from the initial sway imperfections in 5.3.2(7) alone and with $\delta_{H,Ed}$ as the displacements arising from these fictitious horizontal loads (i.e. exclude the effects of any other horizontal loads, such as wind loads).

Alternative (2)

Calculate α_{cr} by computer by finding the first sway-mode from an eigenvalue analysis. When using this type of analysis, it is important to study the form of each buckling mode to see if it is a frame mode or a local column mode. In frames where sway stability is ensured by discrete bays of bracing (often referred to as "braced frames"), it is common to find that the eigenvalues of the column buckling modes are lower than the eigenvalue of the first sway mode of the frame. Local column modes may also appear in unbraced frames at columns hinged at both ends or at columns that are much more slender than the average slenderness of columns in the same storey.

Alternative (3)

F_{cr} may be found from design charts appropriate to the type of building considered.

Calculation of α_{cr}

2. Simplification of load distribution

In calculating F_{cr} for normal multi-storey building frames, it is adequate to model the frame with the loading applied only at the nodes, thereby ignoring the bending moments caused by load distribution. However, for long span portal frames in which the bending moments in the members give rise to significant axial compression in the rafters, the distribution of the load must be modelled when calculating α_{cr}. According to Note 2B of EN 1993-1-1 §5.2.1(4)B, the axial compression in a beam or a rafter may be assumed to be significant if:

$$\bar{\lambda} > 0{,}3\sqrt{\frac{Af_y}{N_{Ed}}}$$

in which

N_{Ed} is the design value of the compression force

$\bar{\lambda}$ is the in-plane non-dimensional slenderness calculated for the beam or rafter considered as hinged at its ends with the length equal to the system length measured along the beam or rafter.

3. Scope of application

The formula (5.2) in EN 1993-1-1 §5.2.1(4)B and alternatives (1) and (3) above apply to normal beam and column buildings and normal portals, because the global instability mode is a sway mode. For certain other forms of frame, such as arches, domes or pyramids, the lowest mode of buckling is not a sway mode, so formula (5.2) will not give a safe value of α_{cr}.

Effective lengths of columns

This NCCI gives information concerning the calculation of the buckling length for columns, to be used for the buckling verification (slenderness approach). Simple aids (e.g. tables, diagrams) are presented.

1. Basis

The buckling length L_{cr} of a compression member is the length of an otherwise similar member with "pinned ends" (ends restrained against lateral movement but free to rotate in the plane of buckling) which has the same elastic critical buckling load.

In the absence of more accurate information, the theoretical buckling length for elastic critical buckling may conservatively be adopted.

An equivalent buckling length may be used to relate the critical load of a member subject to non-uniform loading to that of an otherwise similar member subject to uniform loading.

An equivalent buckling length may also be used to relate the critical load of a non-uniform member to that of a uniform member under similar conditions of loading and restraint.

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Effective lengths of columns

2. Columns in building frames

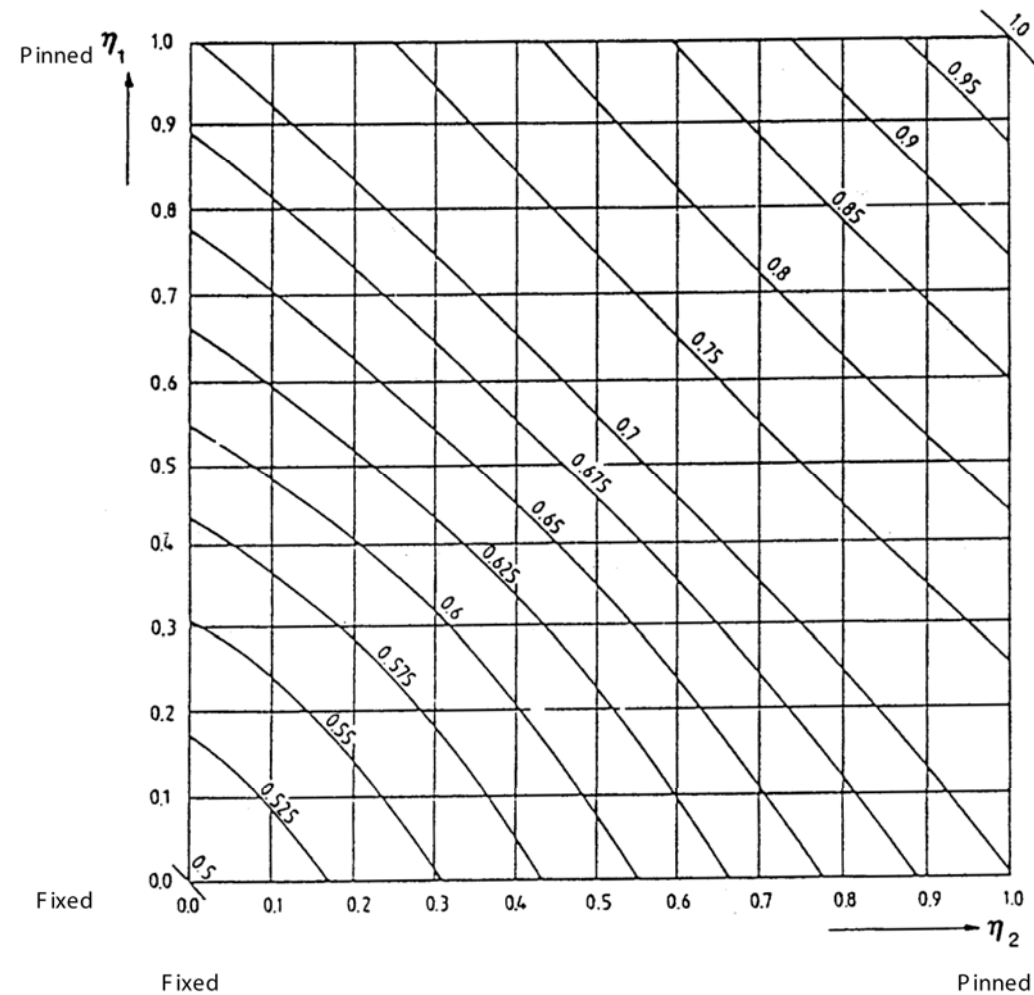The buckling length L_{cr} of a column in a non-sway mode may be obtained from Figure 2.1.

Figure 2.1   Buckling length ratio L_{cr}/L for a column in a non-sway mode

Effective lengths of columns

The buckling length L_{cr} of a column in a sway mode may be obtained from Figure 2.2.
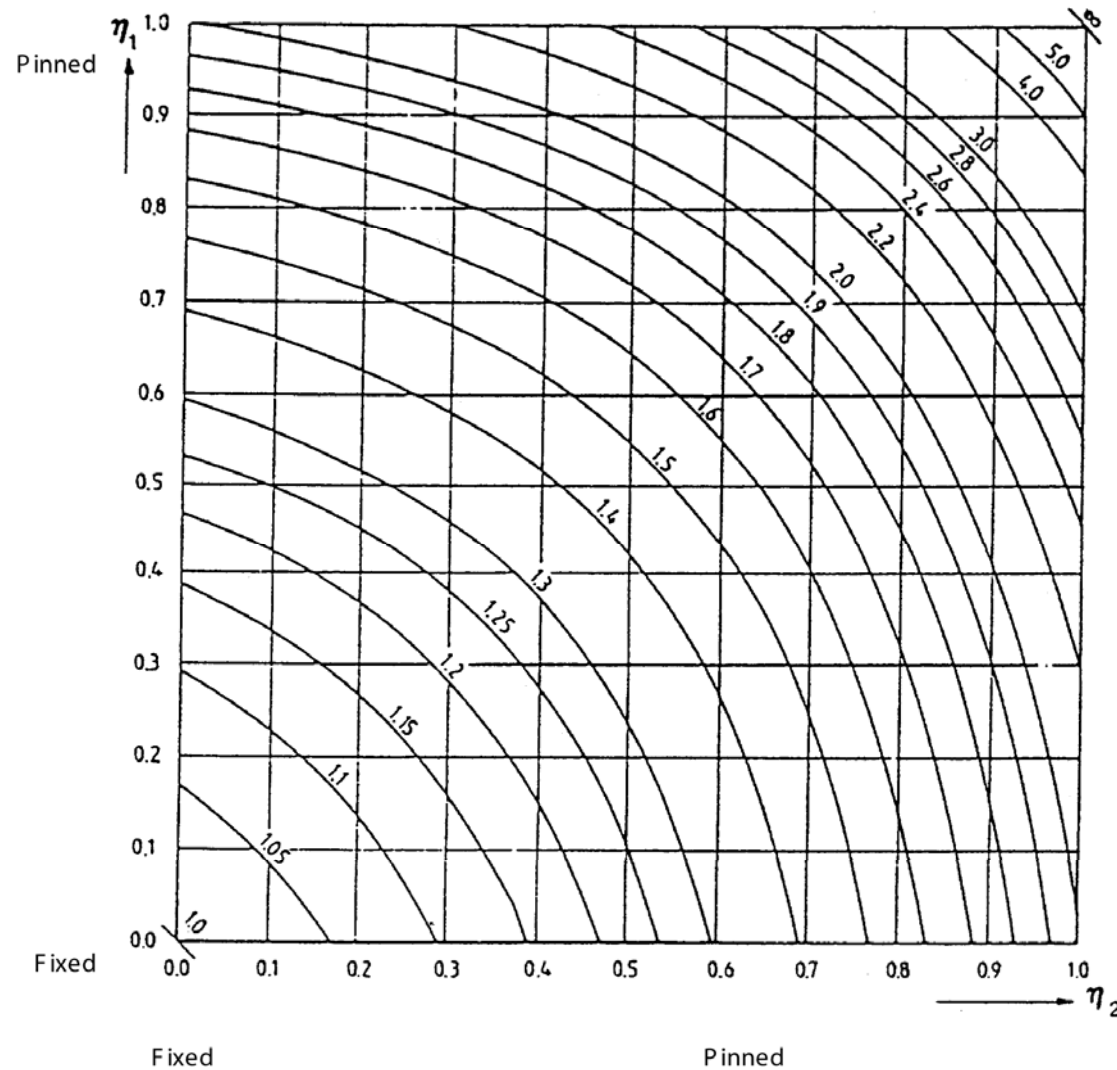

Figure 2.2   Buckling length ratio L_{cr}/L for a column in a sway mode

Effective lengths of columns

The following empirical expressions may be used as conservative approximations instead of reading values from Figure 2.1 and Figure 2.2:

a) non-sway mode (Figure 2.1)

$$\frac{L_{cr}}{L} = 0{,}5 + 0{,}14(\eta_1 + \eta_2) + 0{,}055(\eta_1 + \eta_2)^2$$
(2.1)

b) sway mode (Figure 2.2)

$$\frac{L_{cr}}{L} = \sqrt{\frac{1 - 0{,}2(\eta_1 + \eta_2) - 0{,}12\eta_1\eta_2}{1 - 0{,}8(\eta_1 + \eta_2) + 0{,}6\eta_1\eta_2}}$$
(2.2)

For the theoretical models shown in Figure 2.3 the distribution factors η_1 and η_2 are obtained from:

$$\eta_1 = \frac{K_c}{K_c + K_{11} + K_{12}}$$
(2.3)

$$\eta_2 = \frac{K_c}{K_c + K_{21} + K_{22}}$$
(2.4)

where K_c is the column stiffness coefficient I/L

and K_{ij} is the effective beam stiffness coefficient

Effective lengths of columns
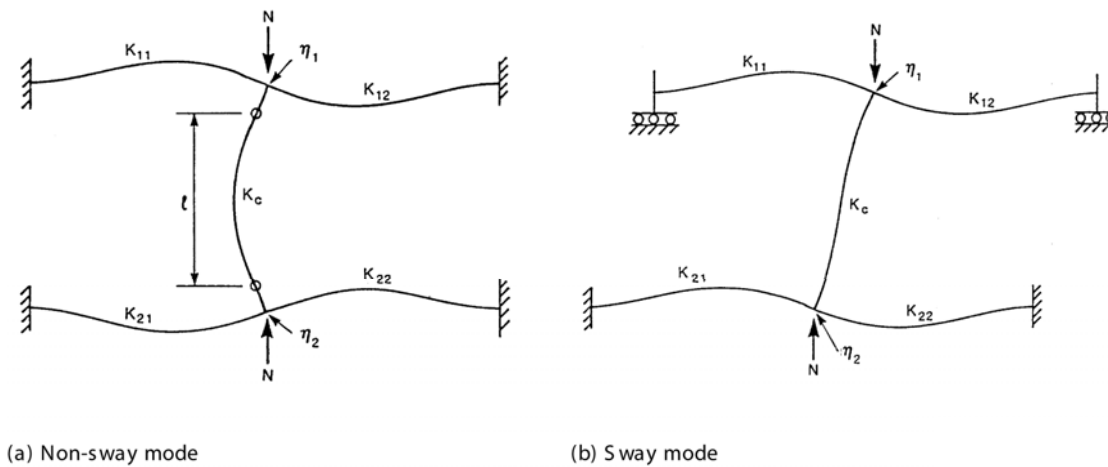

(a) Non-sway mode   (b) Sway mode

Figure 2.3 Distribution factors for columns

Where the beams are not subject to significant axial forces, their effective stiffness coefficients may be determined by reference to Table 2.1, provided that they remain elastic under the design moments.

Table 2.1 Effective stiffness coefficient for a beam

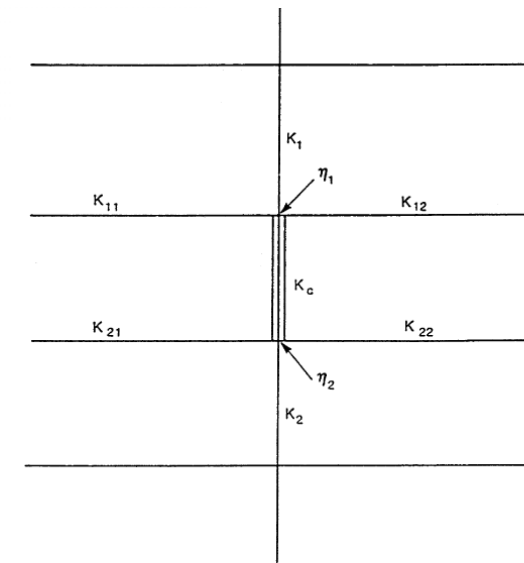| Conditions of rotational restraint at far end of beam	Effective beam stiffness coefficient K (provided that beam remains elastic)
Fixed at far end	$1,0\dfrac{I}{L}$
Pinned at far end	$0,75\dfrac{I}{L}$
Rotation as at near end (double curvature)	$1,5\dfrac{I}{L}$
Rotation equal and opposite to that at near end (single curvature)	$0,5\dfrac{I}{L}$
General case Rotation θ_a at near end and θ_b at far end	$1+0,5\dfrac{\theta_b}{\theta_a}\dfrac{I}{L}$

Figure 2.4 Distribution factors for continuous column

Effective lengths of columns

For building frames with concrete floor slabs, provided that the frame is of regular layout and the loading is uniform, it is normally sufficiently accurate to assume that the effective stiffness coefficients of the beams are as shown in Table 2.2.

Table 2.2   Effective stiffness coefficient for a beam in a building frame with concrete floor slabs

Loading conditions for the beam	Non-sway mode	Sway mode
Beams directly supporting concrete floor slabs	$1,0\dfrac{I}{L}$	$1,0\dfrac{I}{L}$
Other beams with direct loads	$0,75\dfrac{I}{L}$	$1,0\dfrac{I}{L}$
Beams with end moments only	$0,5\dfrac{I}{L}$	$1,5\dfrac{I}{L}$

Where, for the same load case, the design moment in any of the beams exceeds $W_{el}\,f_y/\gamma_{M0}$, as a conservative assumption the beam may be assumed to be pinned at the point or points concerned.

Where a beam has semi-rigid joints, its effective stiffness coefficient should be reduced accordingly.

Effective lengths of columns

Where the beams are subject to significant axial forces, their effective stiffness coefficients should be adjusted accordingly. Stability functions may be used. As a simple alternative, the increased stiffness coefficient due to axial tension may be neglected and the effects of axial compression (when $N/N_E > 0,1$) may be allowed for by multiplying the second moment of area of the beams I by the factor

$$1 - 0,4 \frac{N}{N_E}$$

Where $\quad N_E = \dfrac{\pi^2 EI}{L^2}$

or by using the conservative approximations given in Table 2.3.

Table 2.3   Approximate formulae for reduced beam stiffness coefficients due to axial compression

Conditions of rotational restraint at far end of beam	Effective beam stiffness coefficient K (provided that beam remains elastic)
Fixed	$1,0\dfrac{I}{L} \quad 1-0,4\dfrac{N}{N_E}$
Pinned	$0,75\dfrac{I}{L} \quad 1-1,0\dfrac{N}{N_E}$
Rotation as at near end (double curvature)	$1,5\dfrac{I}{L} \quad 1-0,2\dfrac{N}{N_E}$
Rotation equal and opposite to that at near end (single curvature)	$0,5\dfrac{I}{L} \quad 1-1,0\dfrac{N}{N_E}$

13 documents

- Calculation of α_{crit}
- Choosing a steel sub-grade
- Simply supported secondary composite beams
- Simply supported primary composite beams
- Simply supported beam with lateral restraint at load
- Pinned column with intermediate restraint
- Sway stability
- Tying and avoidance of disproportionate collapse
- Beam to beam fin plate connection
- Beam to column end plate connection
- Column splice (Non-Bearing / Bearing)
- Column base, axially loaded

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- Sway stability
- Tying and avoidance of disproportionate collapse
- Beam to beam fin plate connection
- Beam to column end plate connection
- Column splice (Non-Bearing / Bearing)
- Column base, axially loaded

Example: Choosing a steel sub-grade

This worked example on the choice of a steel sub-grade should clarify for a simple structure the procedure how to use Table 2.1 in EN 1993-1-10 and how to determine the input data element thickness, reference temperature and stress level.
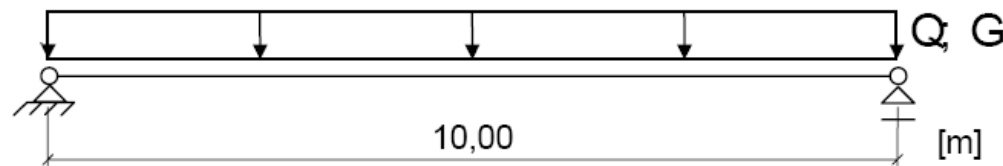
Basic data

Choice of a steel sub-grade for a beam of a multi-storey building according to the data given below.

- Span length :   10,00 m
- Bay width :   6,00 m
- Slab depth :   15 cm
- Partitions :   0,75 kN/m^2
- Imposed load :   2,50 kN/m^2
- Concrete density :   24 kN/m^3
- Steel grade :   S355

Weight of the slab : $0,15 \times 24$ kN/m^3 = 3,60 kN/m^2

IPE 500 – Steel grade S355

Depth	$h = 500$ mm	
Width	$b = 200$ mm	
Web thickness	$t_w = 10,2$ mm	
Flange thickness	$t_f = 16,0$ mm	
Fillet	$r = 21$ mm	
Mass	90,7 kg/m	
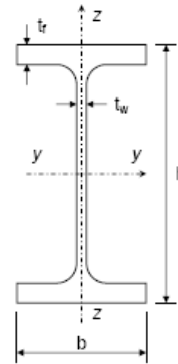
Euronorm 19-57

Section area	$A = 116$ cm^2
Second moment of area /yy	$I_y = 48200$ cm^4
Second moment of area /zz	$I_z = 2140$ cm^4
Torsion constant	$I_t = 89,30$ cm^4
Elastic section modulus /yy	$W_{el.y} = 1928$ cm^3
Plastic section modulus /yy	$W_{pl.y} = 2194$ cm^3

Self weight of the beam : $(90,7 \times 9,81) \times 10^{-3} = 0,89$ kN/m

Permanent load : $G = 0,89 + (3,6 + 0,75) \times 6,00 = 26,99$ kN/m

Variable load (Imposed load) : $Q = 2,5 \times 6,0 = 15,00$ kN/m

Yield strength

Steel grade S355

The maximum thickness is 16 mm < 40 mm, so : $f_y = 355$ N/mm^2

EN 1993-1-1
Table 3.1

Load combination (T$_{Ed}$ is leading action):

$$E_d = E \{ A[T_{Ed}] \text{ "+" } \sum G_K \text{ "+" } \psi_1 \, Q_{K1} \text{ "+" } \underbrace{\sum \psi_{2,i} \, Q_{Ki}} \}$$

not relevant for this example

EN 1993-1-10
§2.2(4)
eq.(2.1)

Where:

$\psi_1 = 0,5$

EN 1990
A1.2.2 (1)

Calculation of reference temperature T_{Ed}

$$T_{Ed} = T_{md} + \Delta T_r + \Delta T_\sigma + \Delta T_R + \Delta T_{\dot\varepsilon} + \Delta T_{\varepsilon_{cf}}$$

EN 1993-1-10
§2.2 eq.(2.2)

Where:

T_{md}	=	$-25\,°C$	(lowest air temperature)

National Annex
to EN 1991-1-5

ΔT_r = $-5\,°C$ (maximal radiation loss)

National Annex
to EN 1991-1-5

ΔT_σ = $0\,°C$ (adjustment for stress and yield strength)

EN 1993-1-10
§2.2 (5) Note 2

ΔT_R = $0\,°C$ (safety allowance to reflect different reliability levels for different applications)

EN 1993-1-10
§2.2 (5) Note 1

$\Delta T_{\dot\varepsilon}$ = $0\,°C$ (assumed strain rate equal to reference strain rate $\dot\varepsilon_0$)

EN 1993-1-10
§2.2 (5)

$\Delta T_{\varepsilon_{cf}}$ = $0\,°C$ (no cold forming for this member)

EN 1993-1-10
§2.2 (5)

T_{Ed} = $-30\,°C$

Calculation of relevant loads

$$Q_K + \psi_1\, G_{K1} = 26{,}99 + 0{,}5 \times 15{,}00 = 34{,}49 \text{ kN/m}$$

Moment diagram
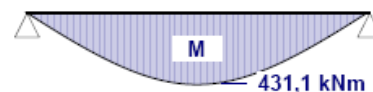
431,1 kNm

Maximum moment at mid span :

$$M_{y,Ed} = 34{,}49 \times 10^2 / 8 = 431{,}1 \text{ kNm}$$

Calculation of maximum bending stress:

$$\sigma_{Ed} = \frac{M_{y,Ed}}{W_{el,y}} = \frac{431{,}1 \times 1000}{1928} = 223{,}6 \ \text{N/mm}^2$$

Stress level as a proportion of nominal yield strength

$$\sigma_{Ed} = 223{,}6 \text{ N/mm}^2$$

$$f_y(t) = f_{y,nom} - 0{,}25 \times \frac{t}{t_0}$$

EN 1993-1-10
§2.3.2

where:

t = 16 mm (flange thickness)

t_0 = 1 mm

$$f_y(t) = 355 - 0,25 \times \frac{16}{1} = 351 \ \text{N/mm}^2$$

Note: $f_y(t)$ may also be taken as R_{eH}-value from the product standard EN10025

proportion of the nominal yield strength

$$\sigma_{Ed} = \frac{223,6}{351} \times f_y(t) = 0,64 \, f_y(t)$$

Choice of steel sub-grade

Note: Two different ways can be used to choose an appropriate steel subgrade. The first one is conservative without usage of interpolation. By linear interpolation the second possibility leads to more economic values. Within this example both methods are presented.

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Choosing a steel sub-grade

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1. Conservative method

Input values:

Proportion of yield strength: $\quad \sigma_{Ed} = 0,75 f_y(t) \qquad > \qquad \sigma_{Ed} = 0,64 f_y(t)$

Temperature: $\qquad\qquad\qquad T_{Ed} = -30 \ °C \qquad = \qquad T_{Ed} = -30 \ °C$

Element thickness: $\qquad\qquad t = 25 \ mm \qquad > \qquad t_f = 16 \ mm$

Table 1:   *Conservative determination of maximum permissible values of element thickness*

EN 1993-1-10
Table 2.1

Steel grade	Sub-grade	Charpy energy CVN at T [°C]	J_{min}	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
				$\sigma_{Ed} = 0,75 f_y(t)$							$\sigma_{Ed} = 0,50 f_y(t)$							$\sigma_{Ed} = 0,25 f_y(t)$						
S235	JR	20	27	60	50	40	35	30	25	20	90	75	65	55	45	40	35	135	115	100	85	75	65	60
	J0	0	27	90	75	60	50	40	35	30	125	105	90	75	65	55	45	175	155	135	115	100	85	75
	J2	-20	27	125	105	90	75	60	50	40	170	145	125	105	90	75	65	200	200	175	155	135	115	100
S275	JR	20	27	55	45	35	30	25	20	15	80	70	55	45	40	35	30	125	110	95	80	70	60	55
	J0	0	27	75	65	55	45	35	30	25	115	95	80	70	55	50	40	165	145	125	110	95	80	70
	J2	-20	27	110	95	75	65	55	45	35	155	130	115	95	80	70	55	200	190	165	145	125	110	95
	M,N	-20	40	135	110	95	75	65	55	45	180	155	130	115	95	80	70	200	200	190	165	145	125	110
	ML,NL	-50	27	185	160	135	110	95	75	65	200	200	180	155	130	115	95	230	200	200	200	190	165	145
S355	JR	20	27	40	35	25	20		15	10	65	55	45	40	30	25	25	110	95	80	70	60	55	45
	J0	0	27	65	55	45	35	**25**	20	15	95	80	65	55	45	40	30	150	130	110	95	80	70	60
	J2	-20	27	90	75	60	50	40	35	25	135	110	95	80	65	55	45	200	175	150	130	110	95	80
	K2,M,N	-20	40	110	90	75	60	50	40	35	155	135	110	95	80	65	55	200	200	175	150	130	110	95
	ML,NL	-50	27	155	130	110	90	75	60	50	200	180	155	135	110	95	80	210	200	200	200	175	150	130
S420	M,N	-20	40	95	80	65	55	45	35	30	140	120	100	85	70	60	50	200	185	160	140	120	100	85
	ML,NL	-50	27	135	115	95	80	65	55	45	190	165	140	120	100	85	70	200	200	200	185	160	140	120
S460	Q	-20	30	70	60	50	40	30	25	20	110	95	75	65	55	45	35	175	155	130	115	95	80	70
	M,N	-20	40	90	70	60	50	40	30	25	130	110	95	75	65	55	45	200	175	155	130	115	95	80
	QL	-40	30	105	90	70	60	50	40	30	155	130	110	95	75	65	55	200	200	175	155	130	115	95
	ML,NL	-50	27	125	105	90	70	60	50	40	180	155	130	110	95	75	65	200	200	200	175	155	130	115
	QL1	-60	30	150	125	105	90	70	60	50	200	180	155	130	110	95	75	215	200	200	200	175	155	130
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	Q	-20	30	50	40	30	25	20	15	10	80	65	55	45	35	30	20	140	120	100	85	75	60	50
	QL	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	QL	-40	30	75	60	50	40	30	25	20	115	95	80	65	55	45	35	190	165	140	120	100	85	75
	QL1	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	66	55	200	200	190	165	140	120	100

S355JO provides a limiting thickness of 25mm > t_f = 16mm

Choosing a steel sub-grade

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2. Exact Determination

Linear interpolation can be used in applying Table 2.1 of EN 1993-1-10 for the actual value of the proportion of the nominal yield strength.

Proportion of yield strength:

a) $\sigma_{Ed} = 0,75 f_y(t)$ **b)** $\sigma_{Ed} = 0,50 f_y(t)$

Temperature:

$T_{Ed} = -30\ °C$ $T_{Ed} = -30\ °C$

Max permissible element thickness for S355JR

$t(\sigma_{Ed} = 0,75 f_y(t)) = 15\ mm$ $t(\sigma_{Ed} = 0,50 f_y(t)) = 30\ mm$

with linear interpolation:

$t(\sigma_{Ed} = 0,64 f_y f_y(t)) = 21,6\ mm\ >\ t_f = 16\ mm$

Table 2: Exact determination of maximum permissible values of element thickness EN 1993-1-10 [Table 2.1](#)

Steel grade	Sub-grade	Charpy energy CVN at T [°C]	t_{min}	$\sigma_{Ed} = 0,75 f_y(t)$							$\sigma_{Ed} = 0,50 f_y(t)$							$\sigma_{Ed} = 0,25 f_u(t)$						
				10	0	-10	-20	**-30**	-40	-50	10	0	-10	-20	**-30**	-40	-50	10	0	-10	-20	-30	-40	-50
S235	JR	20	27	60	50	40	35	30	25	20	90	75	65	55	45	40	35	135	115	100	85	75	65	60
	J0	0	27	90	75	60	50	40	35	30	125	105	90	75	65	55	45	175	155	135	115	100	85	75
	J2	-20	27	125	105	90	75	60	50	40	170	145	125	105	90	75	65	200	200	175	155	135	115	100
S275	JR	20	27	55	45	35	30	25	20	15	80	70	55	50	40	35	30	125	110	95	80	70	60	55
	J0	0	27	75	65	55	45	35	30	25	115	95	80	70	55	50	40	165	145	125	110	95	80	70
	J2	-20	27	110	95	75	65	55	45	35	155	130	115	95	80	70	60	200	190	165	145	125	110	95
	M,N	-20	40	135	110	95	75	65	55	45	180	155	130	115	95	80	70	200	200	190	165	145	125	110
	ML,NL	-50	27	185	160	135	110	95	75	65	200	200	180	155	130	115	95	230	200	200	200	190	165	145
S355	JR	20	27	40	35	25	20	**15**	15	10	65	55	45	40	**30**	25	25	110	95	80	70	60	55	45
	J0	0	27	60	50	40	35	25	20	15	95	80	65	55	45	40	35	150	130	110	95	80	70	60
	J2	-20	27	90	75	60	50	40	35	25	135	110	95	80	65	55	45	200	175	150	130	110	95	80
	K2,M,N	-20	40	110	90	75	60	50	40	35	155	135	110	95	80	65	55	200	200	175	150	130	110	95
	ML,NL	-50	27	155	130	110	90	75	60	50	200	180	155	135	110	95	80	210	200	200	200	175	150	130
S420	M,N	-20	40	95	80	65	55	45	35	30	140	120	100	85	70	60	50	200	185	160	140	120	100	85
	ML,NL	-50	27	135	115	95	80	65	55	45	190	165	140	120	100	85	70	200	200	200	185	160	140	120
S460	Q	-20	30	70	60	50	40	30	25	20	110	95	75	65	55	45	35	175	155	130	115	95	80	70
	M,N	-20	40	90	70	60	50	40	30	25	130	110	95	75	65	55	45	200	175	155	130	115	95	80
	QL	-40	30	105	90	70	60	50	40	30	155	130	110	95	75	65	55	200	200	175	155	130	115	95
	ML,NL	-50	27	125	105	90	70	60	50	40	180	155	130	110	95	75	65	200	200	200	175	155	130	115
	QL1	-60	30	150	125	105	90	70	60	50	200	180	155	130	110	95	75	215	200	200	200	175	155	130
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	Q	-20	30	50	40	30	25	20	15	10	80	65	55	45	35	30	20	140	120	100	85	75	60	50
	QL	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	QL	-40	30	75	60	50	40	30	25	20	115	95	80	65	55	45	35	190	165	140	120	100	85	75
	QL1	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100

S355JR provides a limiting thickness of 21,6mm > t_f = 16mm

Buckling resistance

Example: Buckling resistance of a pinned column with intermediate restraints

This worked example shows the procedure to determine the buckling resistance of a pinned column with intermediate restraints.
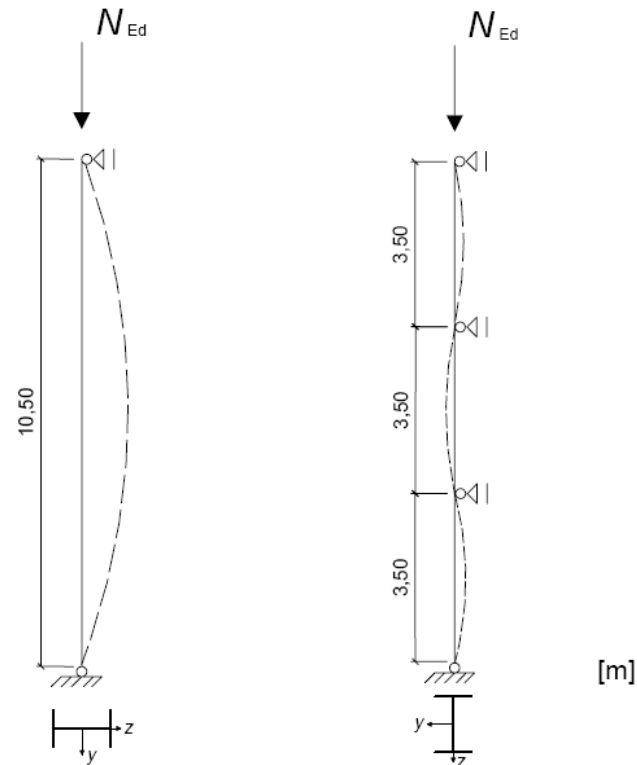
The column is a rolled HE profile which is laterally restrained every 3,5 meters.

$N_{Ed} = 1000$ kN

Class 4 cross-sections are not covered.

Partial factors

- $\gamma_{M0} = 1,0$

- $\gamma_{M1} = 1,0$

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§ 6.1 (1)

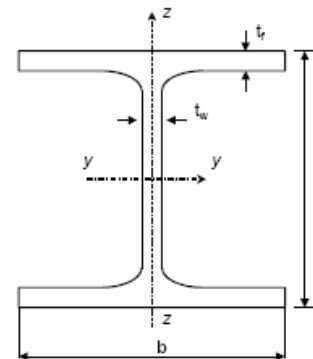## Basic data

Design a pinned column of a multi-storey building based on the following data:

- Column length :   10,50 m
- Intermediate restraints (about y-y axis) every   3,50 m
- Steel grade :   S235
- Section classification: Class 1

HE 260 A – Steel grade S235 | | | | Euronorm 19-57

Depth	$h = 250$ mm	
Width	$b = 260$ mm	
Web thickness	$t_w = 7,5$ mm	
Flange thickness	$t_f = 12,5$ mm	
Fillet	$r = 24$ mm	

Section area	$A = 86,8$ cm^2
Second moment of area /yy	$I_y = 10450$ cm^4
Second moment of area /zz	$I_z = 3668$ cm^4

Yield strength

Steel grade S235   EN 1993-1-1
Table 3.1

The maximum thickness is 12,5 mm < 40 mm, so : $f_y = 235$ N/mm^2

Note :   The National Annex may impose either the values of f_y from the Table 3.1 or the values from the product standard.

Design buckling resistance of a compression member

To determine the design column buckling resistance $N_{b,Rd}$, the reduction factor χ for the relevant buckling curve must be defined. This factor is determined by calculation of the non-dimensional slenderness $\overline{\lambda}$ based on the elastic critical force for the relevant buckling mode and the cross sectional resistance to normal forces.

Elastic critical force for the relevant buckling mode N_{cr}

The critical force may be calculated from the following expression :

$$N_{cr,y} = \frac{\pi^2 \times EI_y}{L_{cr,y}^2} = \frac{\pi^2 \times 21000 \times 10450}{1050^2} = 1964,5 \text{ kN}$$

$$N_{cr,z} = \frac{\pi^2 \times EI_z}{L_{cr,z}^2} = \frac{\pi^2 \times 21000 \times 3668}{350^2} = 6206,0 \text{ kN}$$

E is the Young´s modulus : $E = 210000 \text{ N/mm}^2$

L is the buckling length in the buckling plane considered: $L_{cr,y} = 10,50$ m

$L_{cr,z} = 3,50$ m

Non-dimensional slenderness

The non-dimensional slenderness is given by :

$$\overline{\lambda}_y = \sqrt{\frac{A f_y}{N_{cr,y}}} = \sqrt{\frac{86,8 \times 23,5}{1965}} = 1,019$$

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§ 6.3.1.2 (1)

$$\overline{\lambda}_z = \sqrt{\frac{A f_y}{N_{cr,z}}} = \sqrt{\frac{86,8 \times 23,5}{6206}} = 0,573$$

Reduction factor

For axial compression in members the value of χ for the appropriate non-dimensional slenderness $\overline{\lambda}$ should be determined from the relevant buckling curve according to:

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \overline{\lambda}^2}} \quad but \quad \chi \le 1,0$$

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§ 6.3.1.2 (1)

where : $\varphi_{LT} = 0,5 \left[1 + \alpha \left(\overline{\lambda} - 0,2 \right) + \overline{\lambda}^2 \right]$

α is an imperfection factor.

For $h/b = 250/260 = 0,96 < 1,2$ and $t_f = 12,5 < 100$ mm

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Table 6.1
Table 6.2

- buckling about axis y-y:

  Buckling curve b, imperfection factor $\alpha = 0,34$

  $$\varphi_y = 0,5 \left[1 + 0,34 \left(1,019 - 0,2 \right) + 1,019^2 \right] = 1,158$$

  $$\chi_y = \frac{1}{1,158 + \sqrt{1,158^2 - 1,019^2}} = 0,585$$

- buckling about axis z-z:

  Buckling curve c, imperfection factor $\alpha = 0,49$

  $$\varphi_z = 0,5 \left[1 + 0,49 \left(0,573 - 0,2 \right) + 0,573^2 \right] = 0,756$$

  $$\chi_z = \frac{1}{0,756 + \sqrt{0,756^2 - 0,573^2}} = 0,801$$

$\chi = \min \left(\chi_y; \chi_z \right)$

$\chi = 0,585 \; < \; 1,0$

Design buckling resistance of a compression member

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§ 6.3.1.1

$$N_{b,Rd} = \chi \frac{A \times f_y}{\gamma_{M1}} = 0,585 \frac{86,8 \times 23,5}{1,0} = 1193 \text{ kN}$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{1000}{1193} = 0,84 < 1,0 \quad \textbf{OK}$$

Conceptual design and design examples for multi-storey buildings

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