Ekonomikas ministrija

INŽENIERU KOMPETENCES CENTRS

Tiešsaistes apmācību seminārs būvspeciālistiem un projektētājiem

ID Nr. EM 2023/28

Rīga, 2023

Ekonomikas ministrija

CMB | INŽENIERU KOMPETENCES CENTRS

Training seminar / Apmācību seminārs

"SALIEKAMO DZELZSBETONU KONSTRUKCIJU PROJEKTĒŠANA"

DESIGN OF PRECAST CONCRETE STRUCTURES

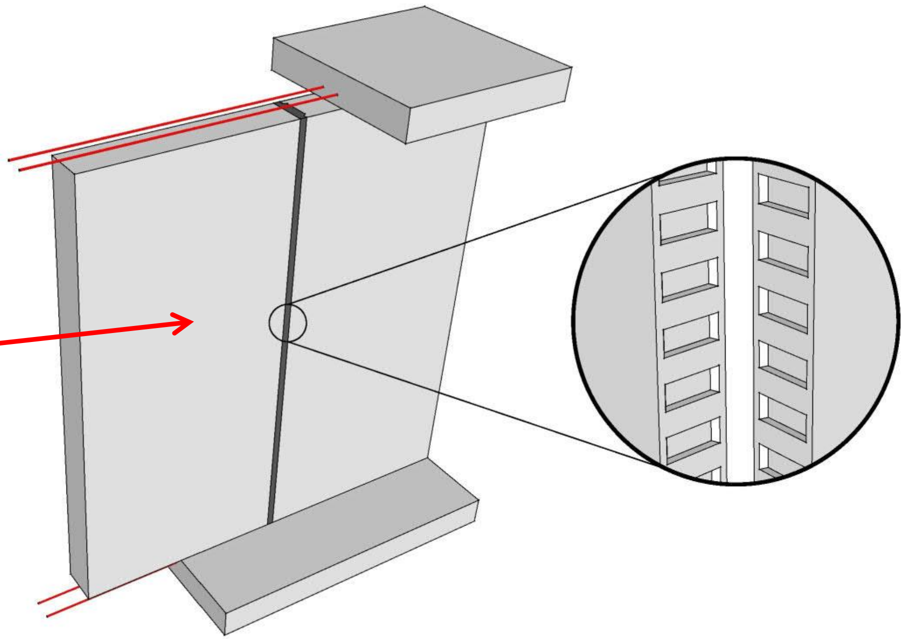
October 18th, 2023

Dick van Keulen (Nīderlande) un Gabriel Tarta (Rumānija)

Introduction

Closer introduction lecturer:

- Dick van Keulen, PhD, MSc
- Founder of structural design firm (2004): INGENIEURSSTUDIO DCK
  - Specialised in design and engineering of precast concrete buildings
- Research topics related to precast concrete:
  - High-Rise Building Design and Construction
  - PhD research “Narrow Vertical Mortar Connections for Precast Concrete Shear Walls
  - Robustness of building structures
  - Member of several committees



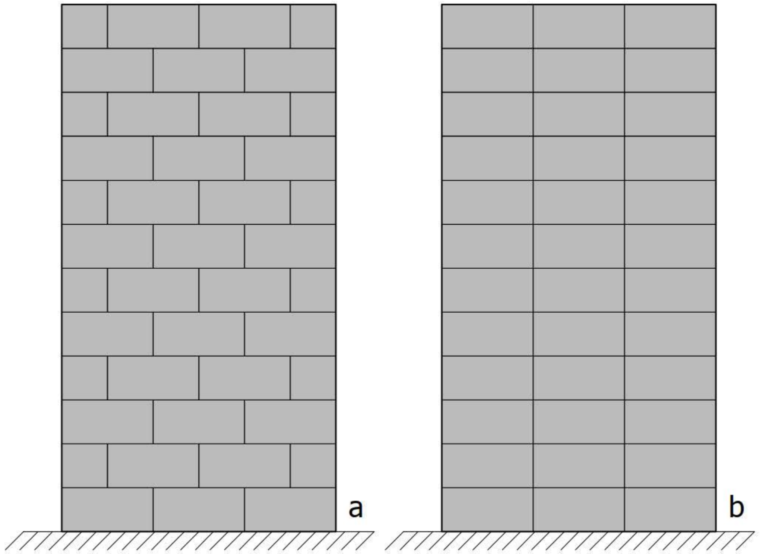
Agenda/ 10:00 – 11:40

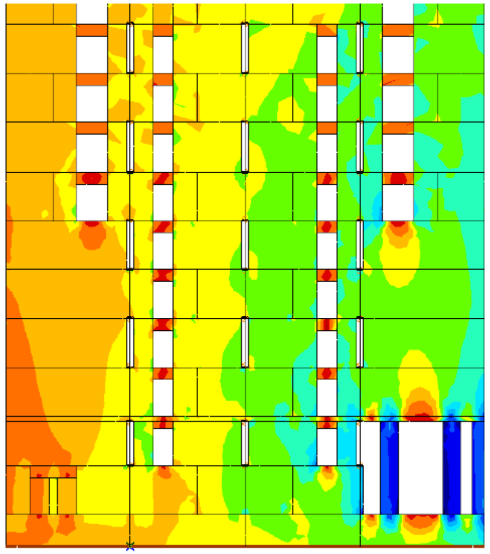
- Introduction
- Design and calculations for precast concrete building structures:
  - Specifics for element lay-out design
  - Requirements in current and next generation of Eurocodes
  - FEM modeling and analysis of structures composed of HCS and Shear Walls
- Q&A session

Introduction

- Session 1: Design and calculations
  - Specifics for element lay-out design for buildings
    - Composition of wall elements and connections
  - Requirements in Eurocodes

    $\tau_{Rdi} = c_{v2} \sqrt{(f_{ck})}/\gamma_C + \mu_v \sigma_n + k_t \rho f_{yd} \mu_v + k_f \rho \sqrt{(f_{yd} f_{cd})}$
  - FEM modeling and analysis of structures composed of HCS and Shear Walls





Introduction

Precast Concrete Construction methode

Construction wall element
in concrete factory

Transport over
public roads

Vertical transport

Assembly at construction site

Introduction

Why precast concrete construction?

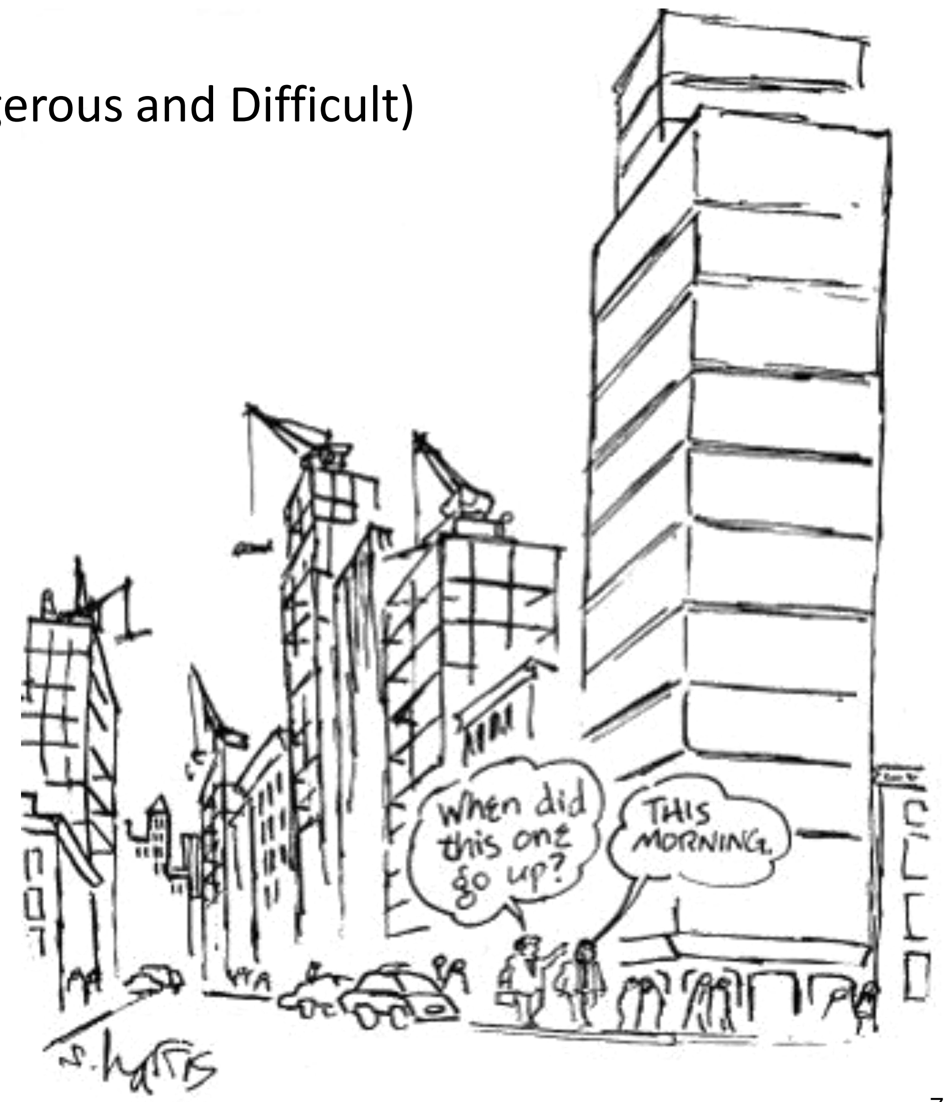
- Traditional construction sites suffer 3-D syndrome (Dirty, Dangerous and Difficult)

Advantages

- Ready-made components only need to be assembled
- High quality due to concrete factory production
- Less complex logistics on construction sites
- High construction speed!

Disadvantages

- Relatively long engineering process
- No flexibility for late changes
- Size and weight of elements limited (due to road transport)



Introduction

When are the benefits really derived?

- Construction without traditional scaffolding
- Ready-made façade element (including glazed frames)
- Self-supporting slabs with pre-attached balconies

Introduction

Examples of buildings constructed without scaffolding

Maanplein Den Haag (70m)

De Brabander Tilburg (60m)

Grote Beer Rotterdam (70m)

Zalmhaventoren Rotterdam (215m)

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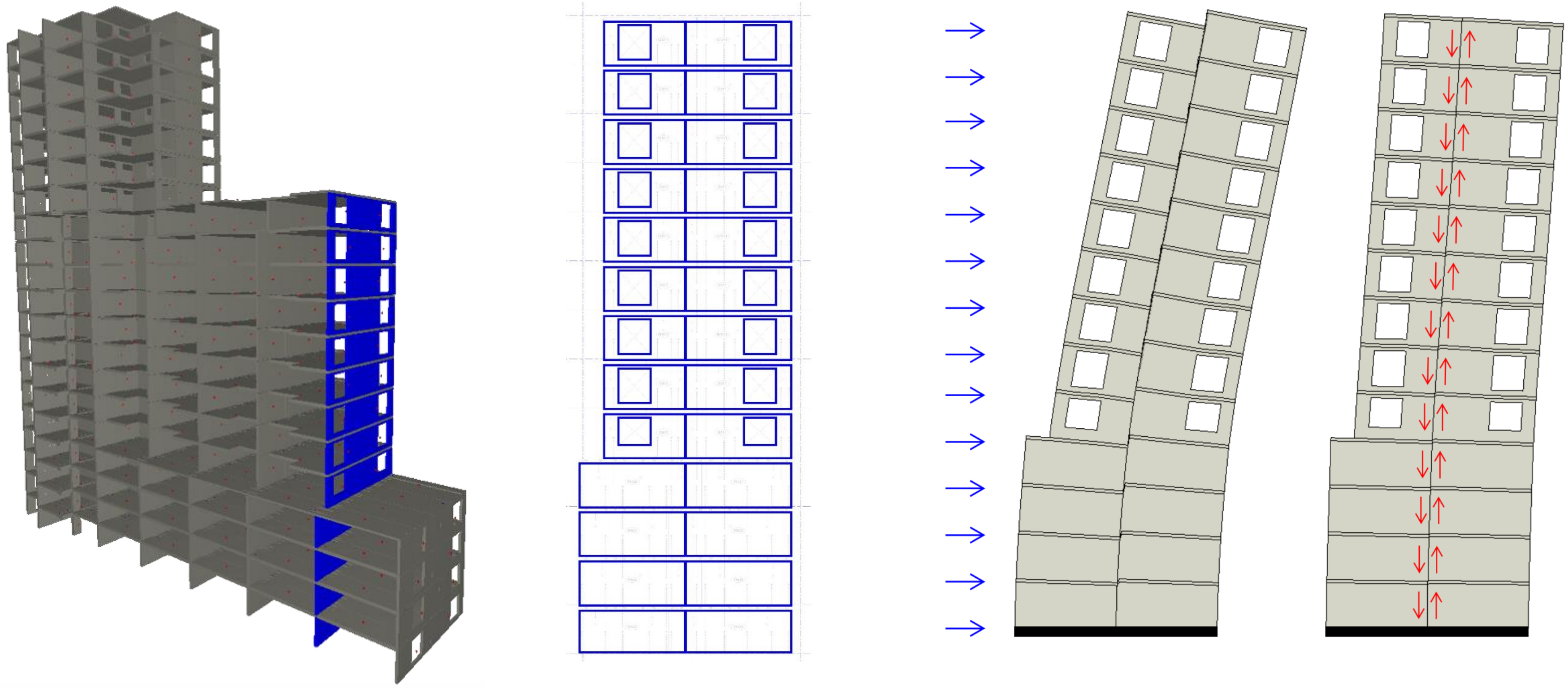
Element lay-out design

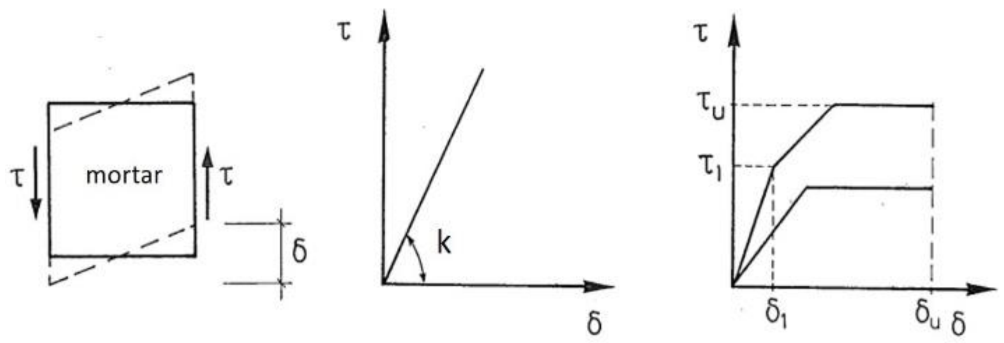
Specifics for element lay-out design



Element lay-out design

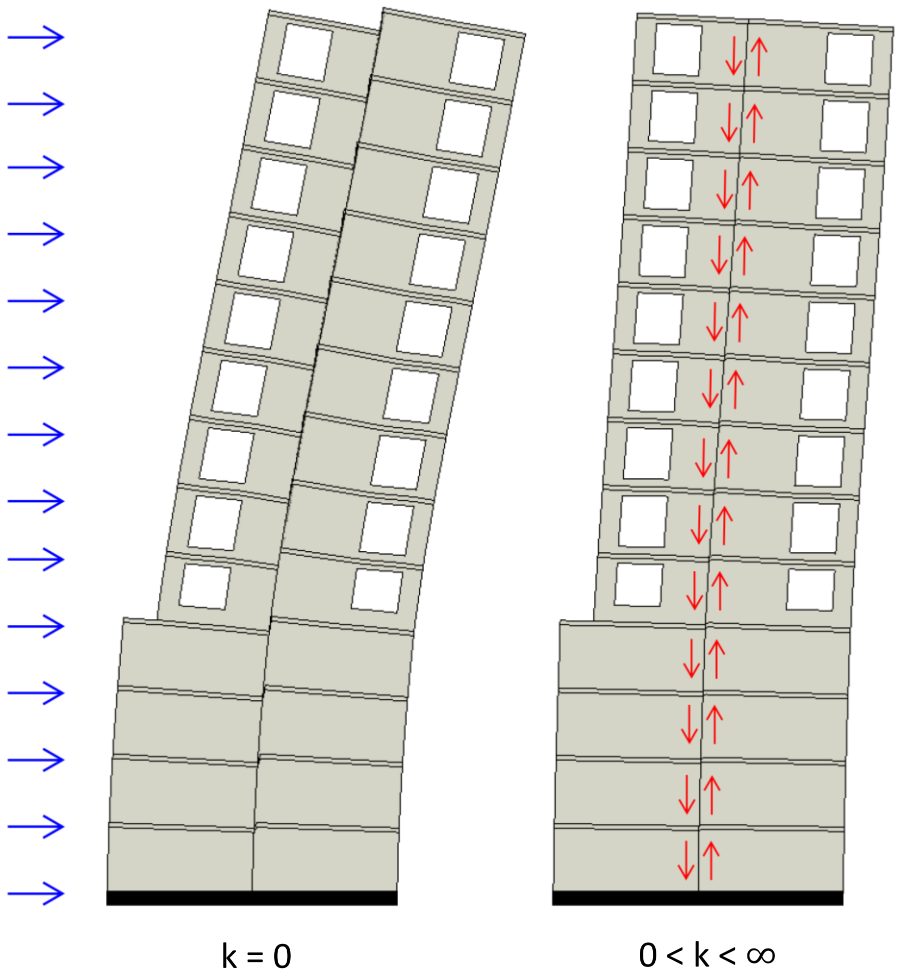
Specifics for element lay-out design

- Shear behaviour of a vertical connection



Shear stiffness $k = \frac{\tau}{\delta}$

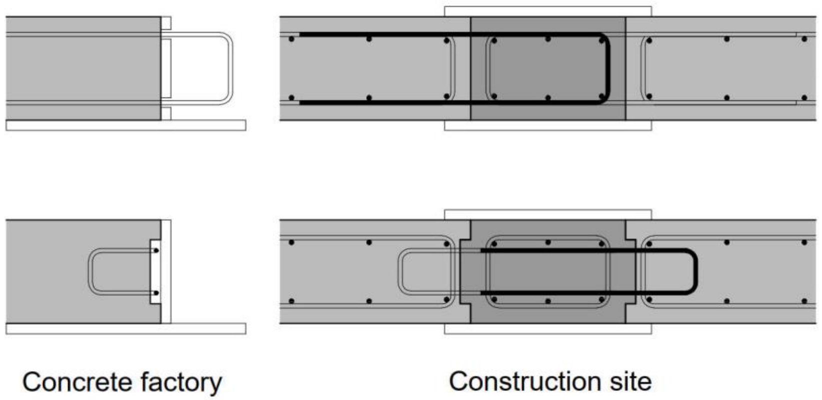
Shear capacity τ_u



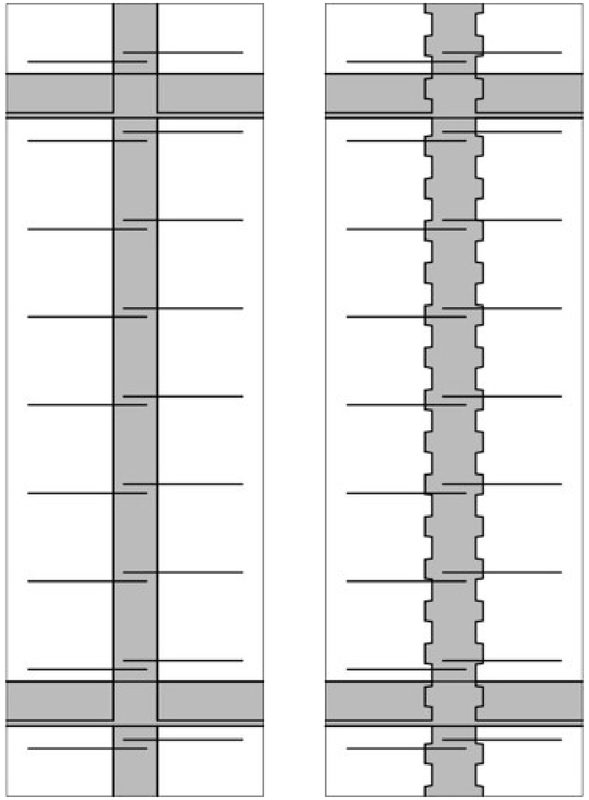
Element lay-out design

Specifics for element lay-out design

- Cast-in-place loop connection
- Continuous connection
- For the transfer of heavy forces
- Very laborious !



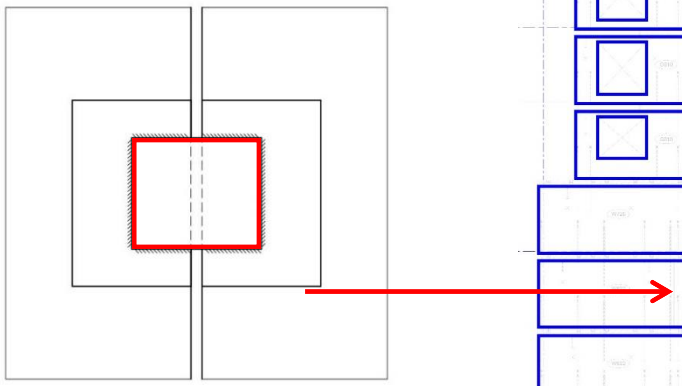
Rebend connection



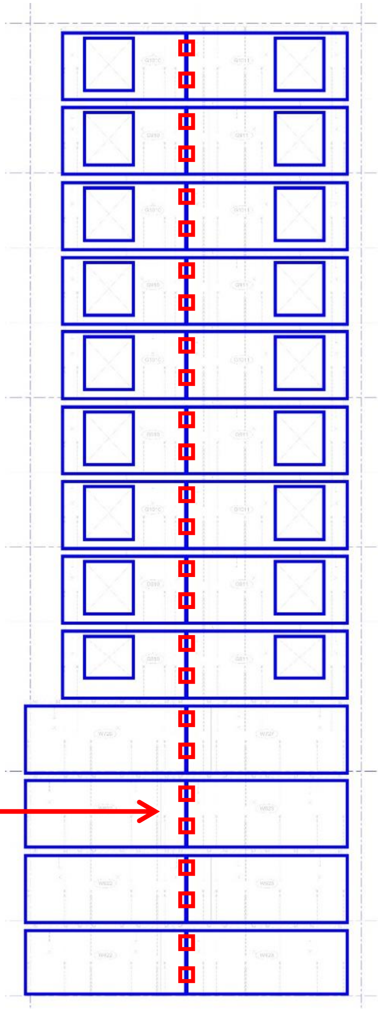
Element lay-out design

Specifics for element lay-out design

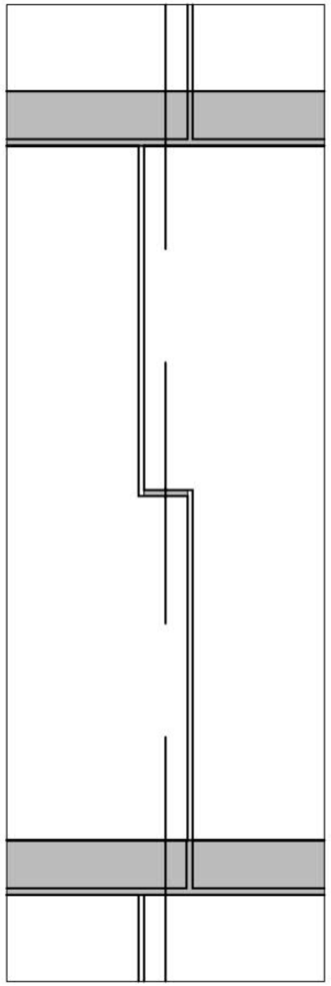
- Discrete connections
  - Welded plates
  - Corbels
  - Bolted
- Interlocking elements



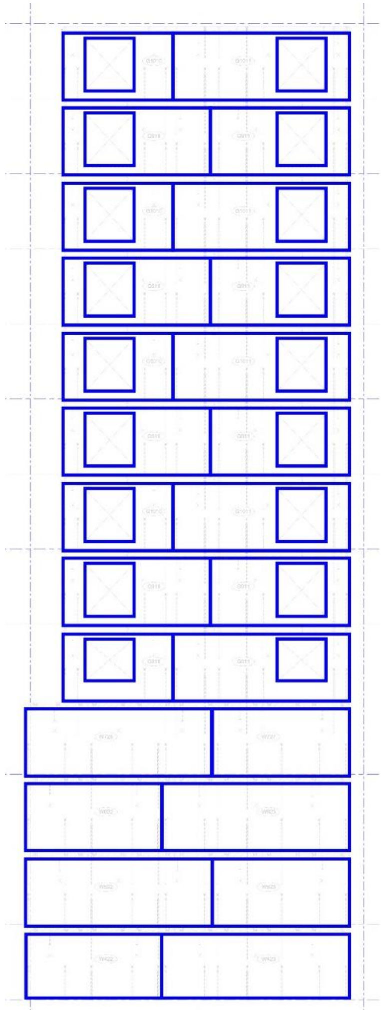
Welded connection



Corbel connection



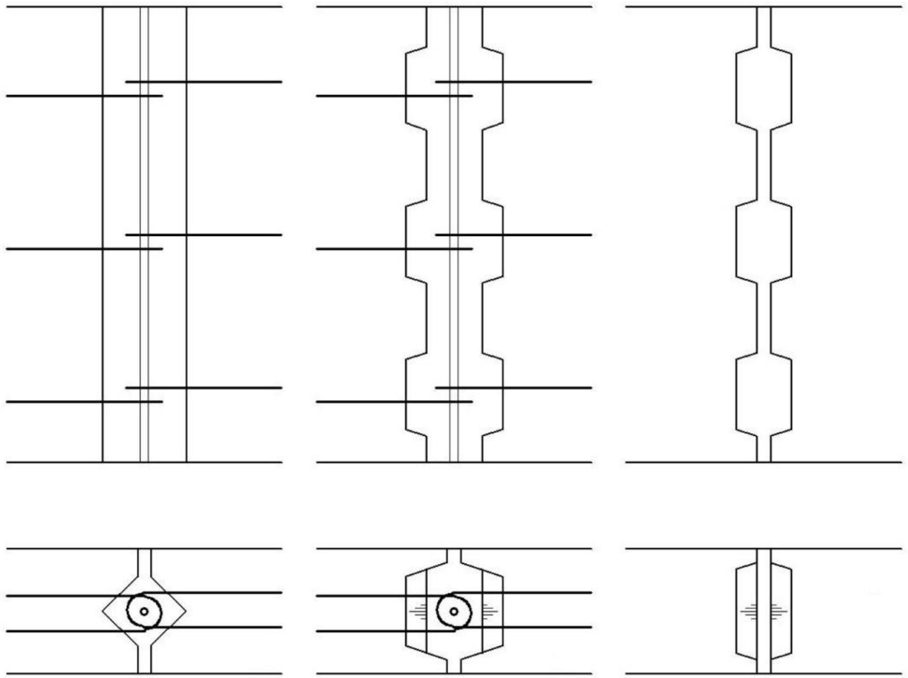
Interlocking wall elements

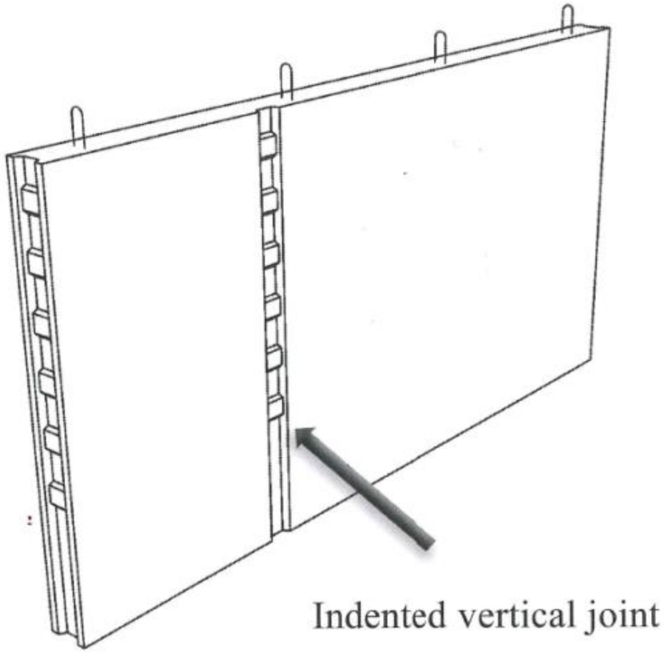


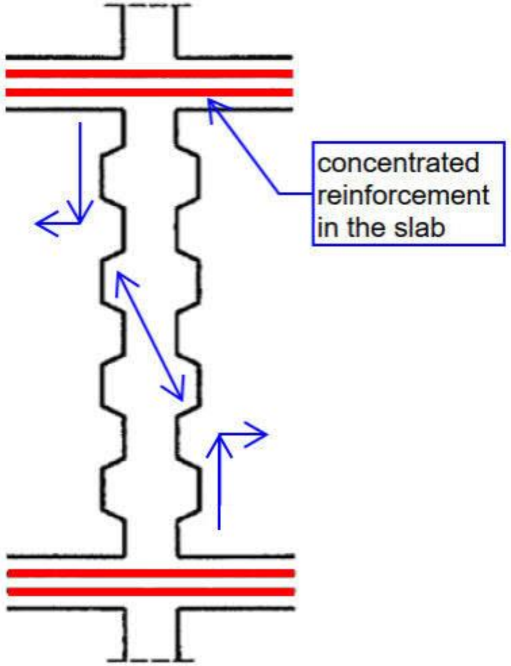
Element lay-out design

Specifics for element lay-out design

- Mortar connections
- Combinations of connections (e.g. profiled mortar connection with welded plates)

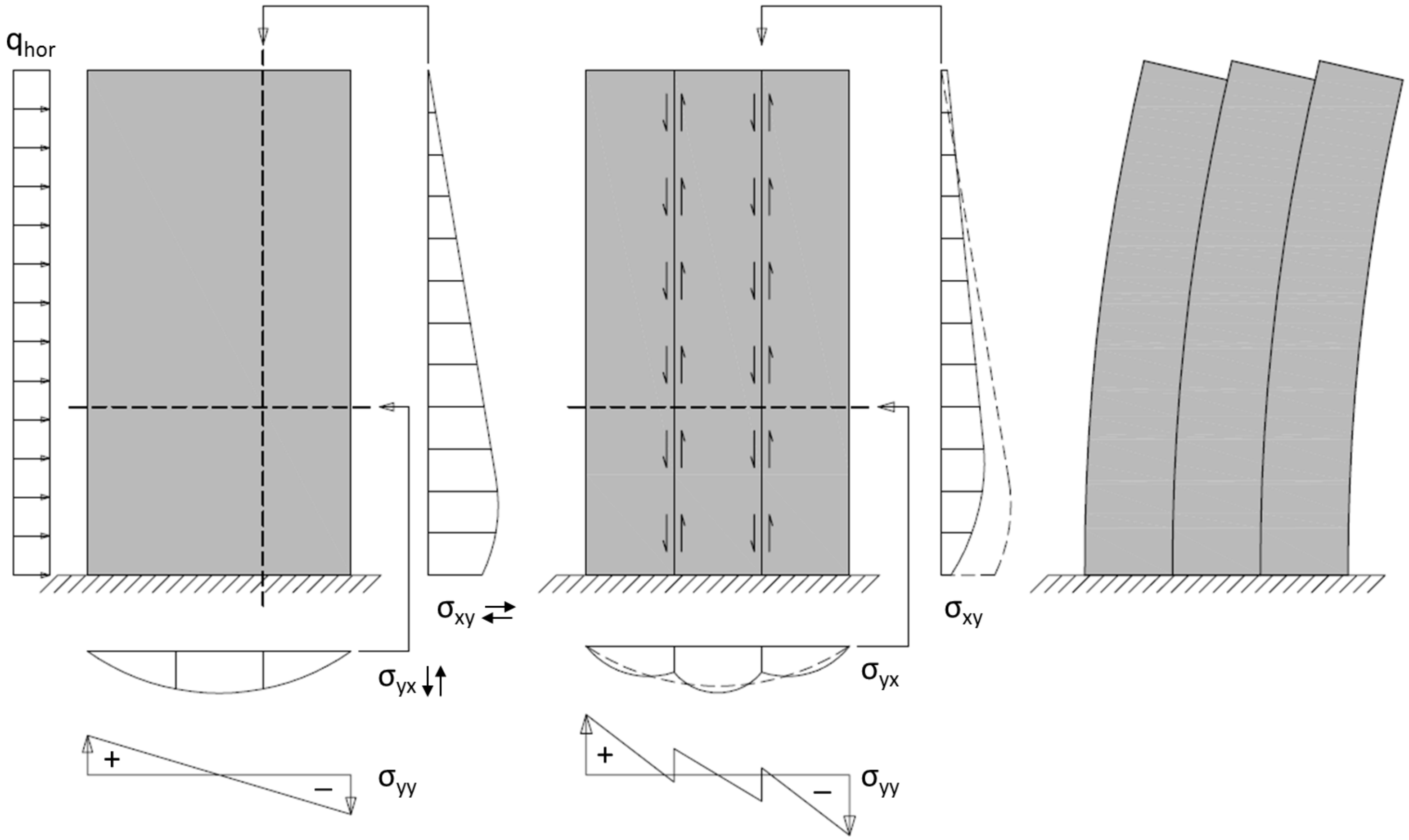






Specifics for element lay-out design

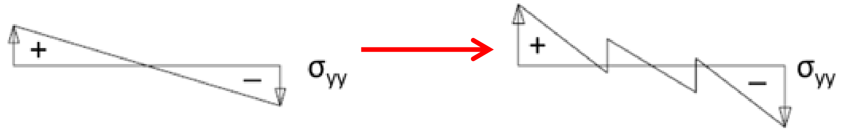
- Effect of a vertical shear connection on cantilever shear wall behaviour

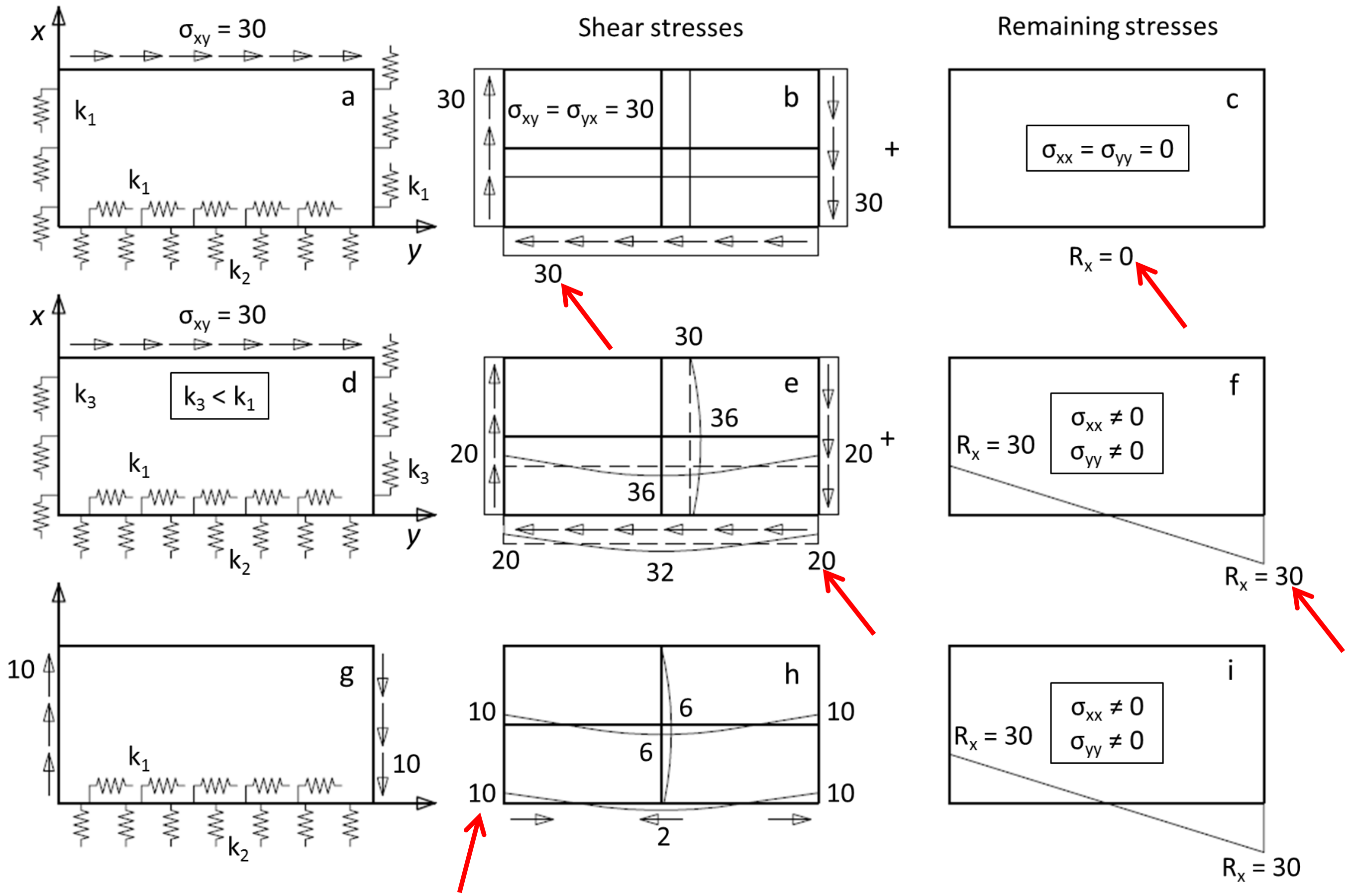


Element lay-out design

Specifics for element lay-out design

- Experiment with shear stress distributions
- Plate model with uniform shear stresses and vertical supports stiffnesses k_1:
- In case with reduced stiffness k_3: representing reduced connection shear stiffness:
  - Causes vertical (bending) stresses R_x
- Remaining stresses (10) to be transferred via bending in the three wall element stacks:

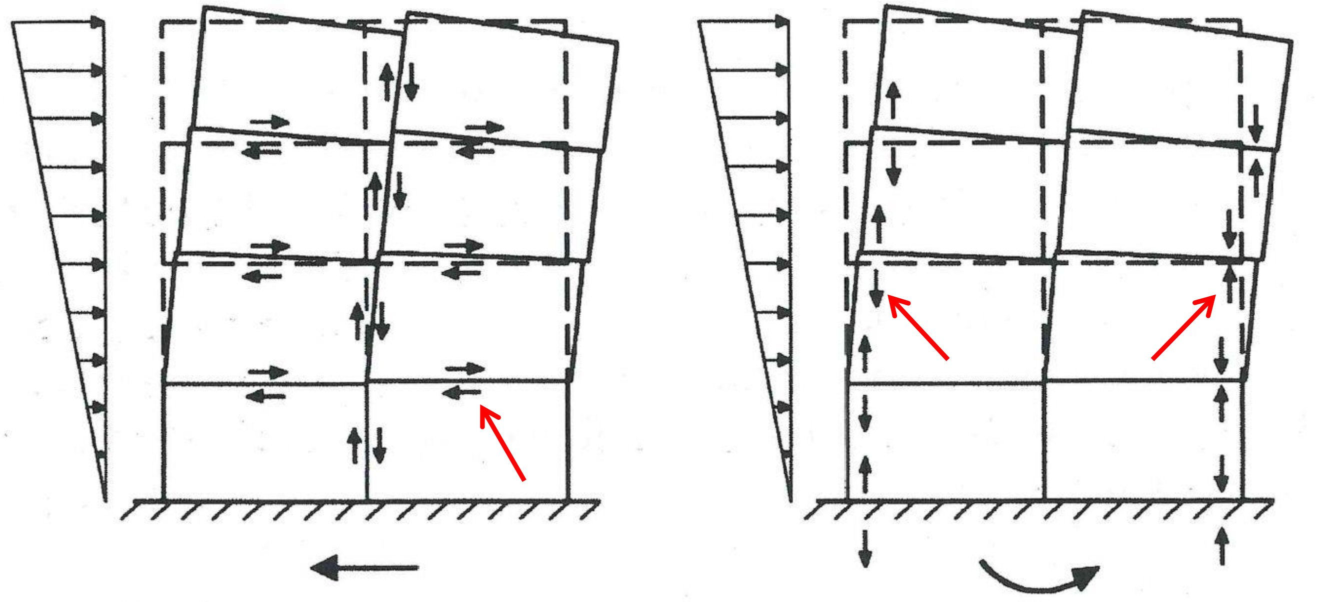




Element lay-out design

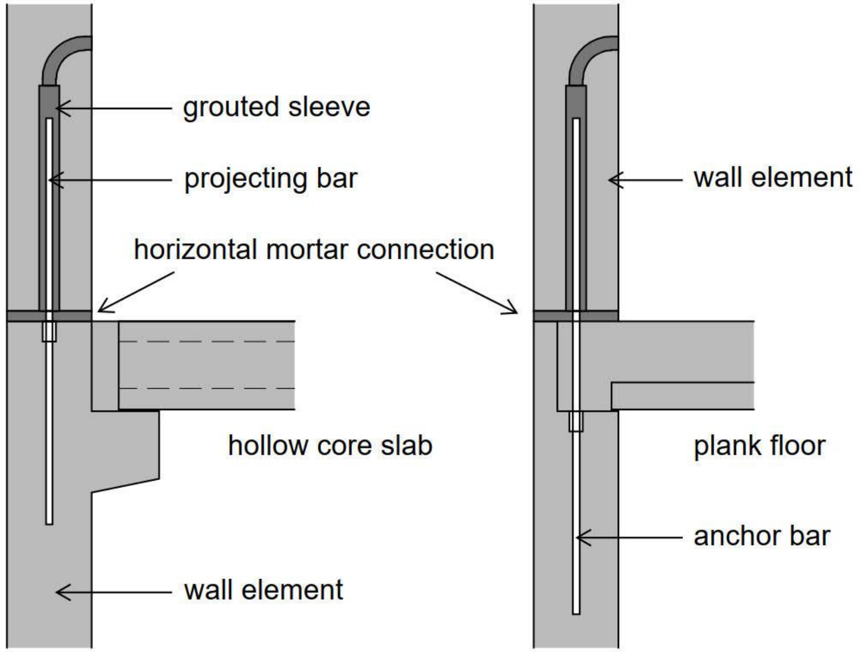
Specifics for element lay-out design

- Traditional horizontal mortar connection



Shear forces

Bending moments
(Tension and Compression forces)

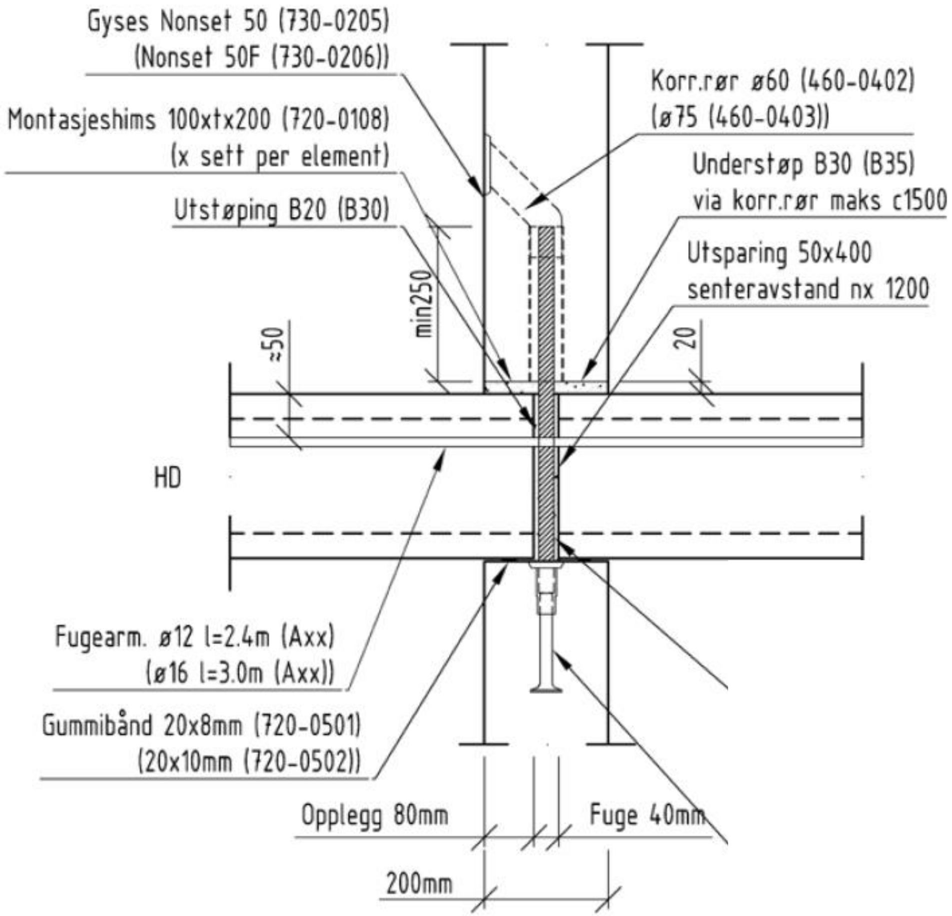


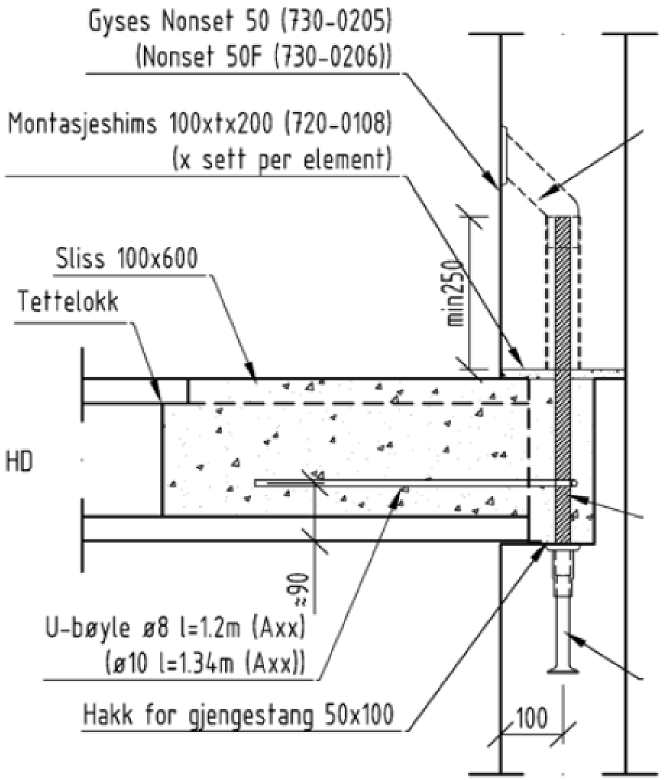
Element lay-out design

Specifics for element lay-out design

- Hollow Core Slab (HCS) –

  Wall intersections
- Consolis Spenconn





Element lay-out design

Specifics for element lay-out design

- Hollow Core Slab (HCS) –

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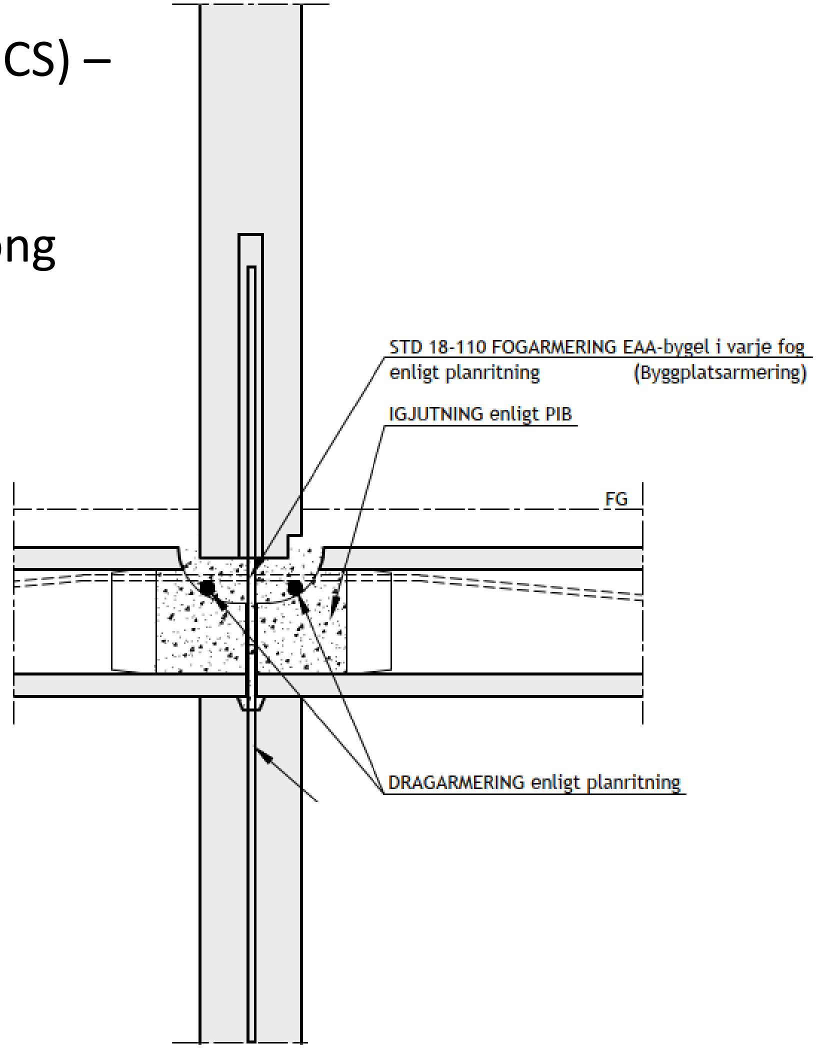
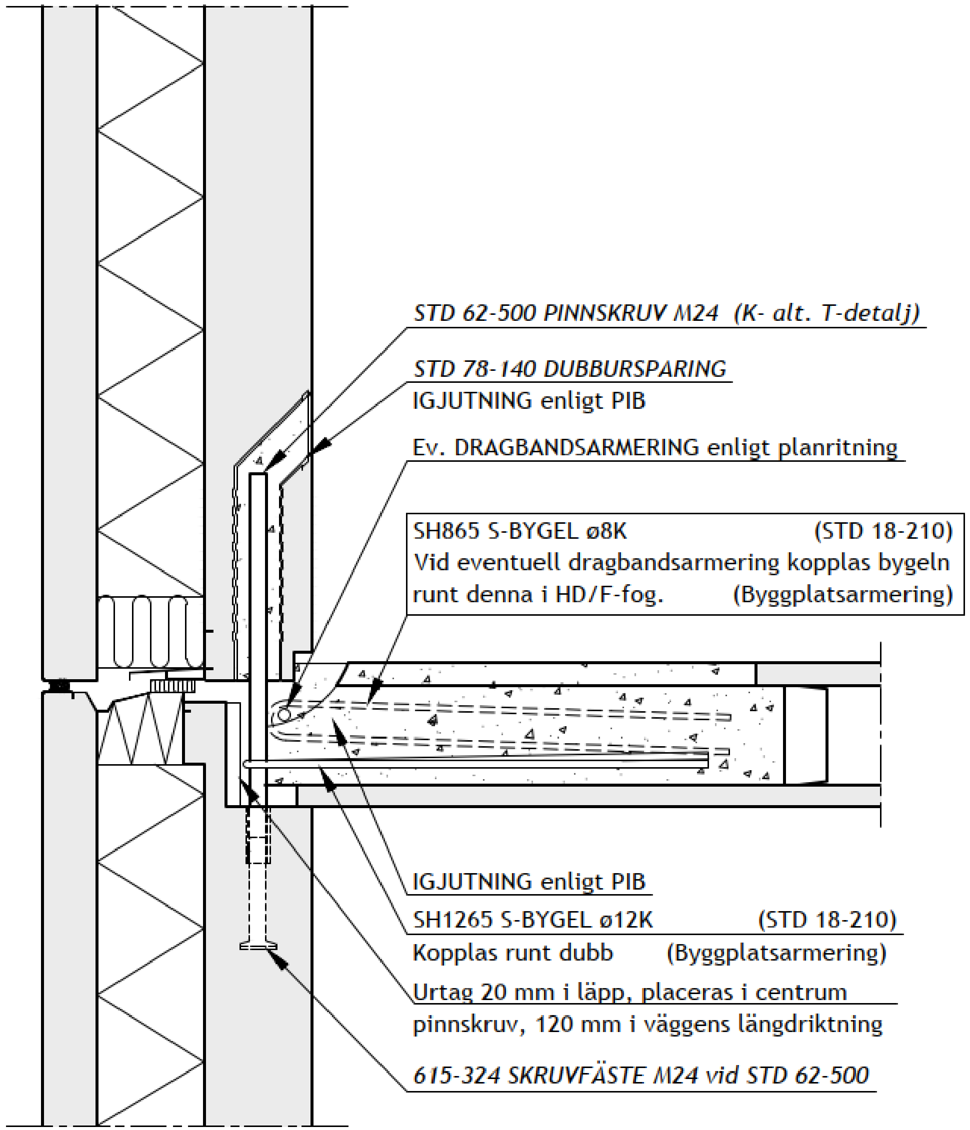


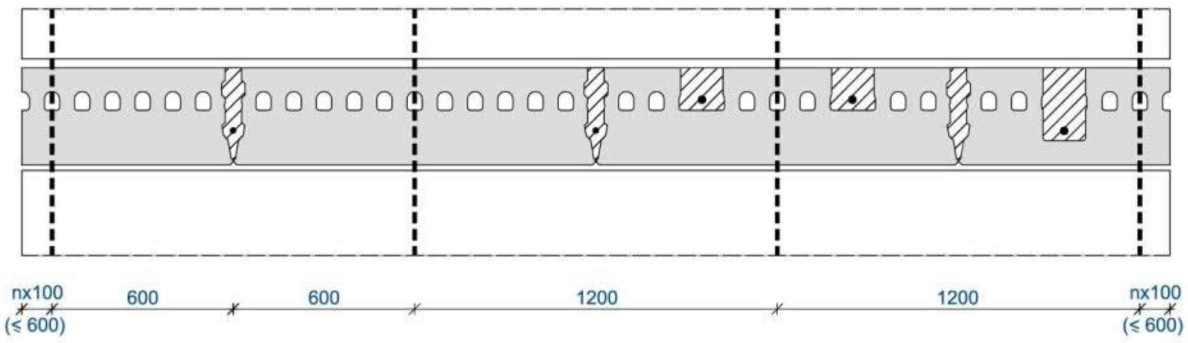


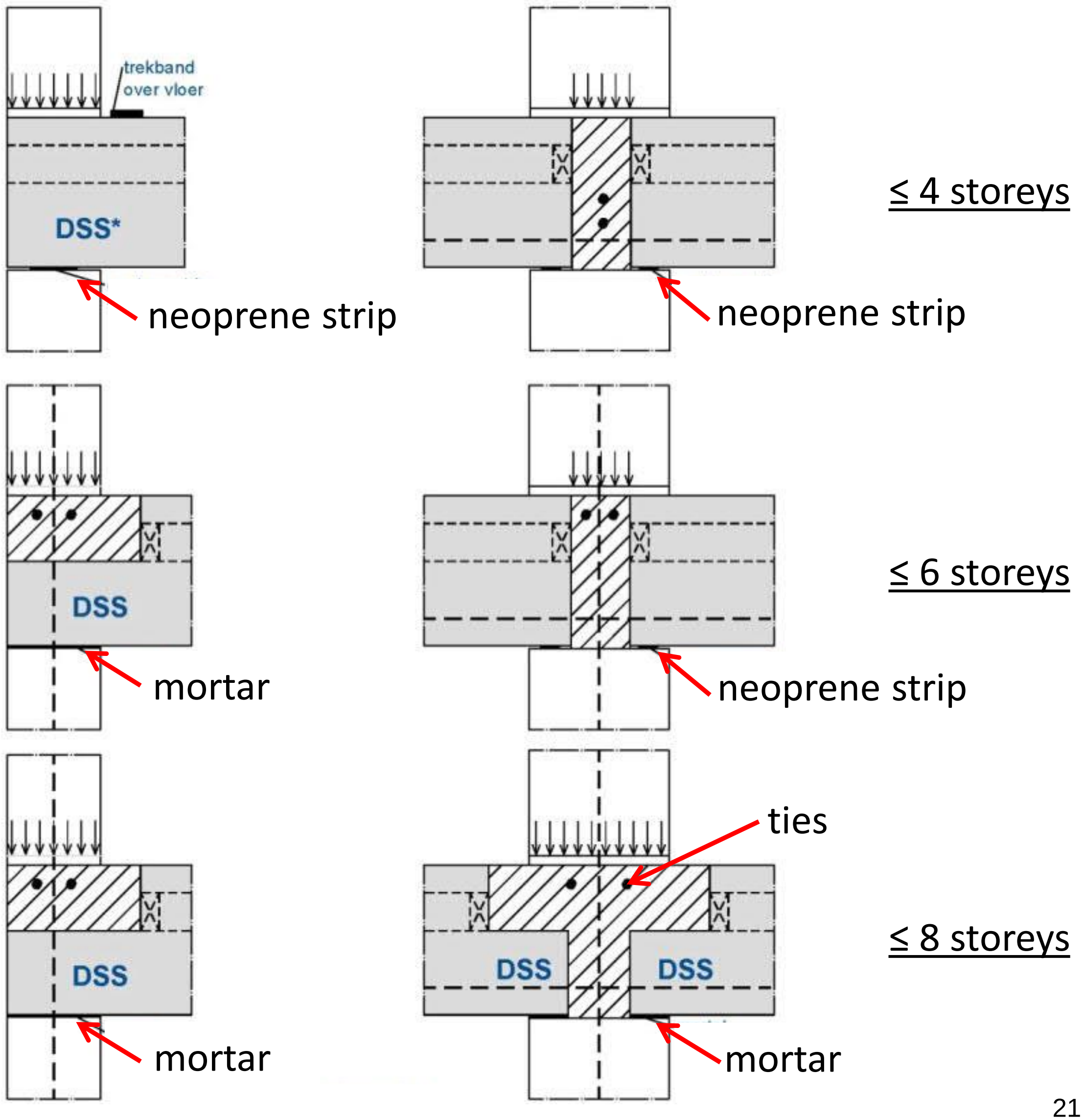
Element lay-out design

Specifics for element lay-out design

- DDS concept of Consolis VBI
- For multi-storey buildings
- Neoprene strip application restricted

- Horizontal and vertical tying reinforcement

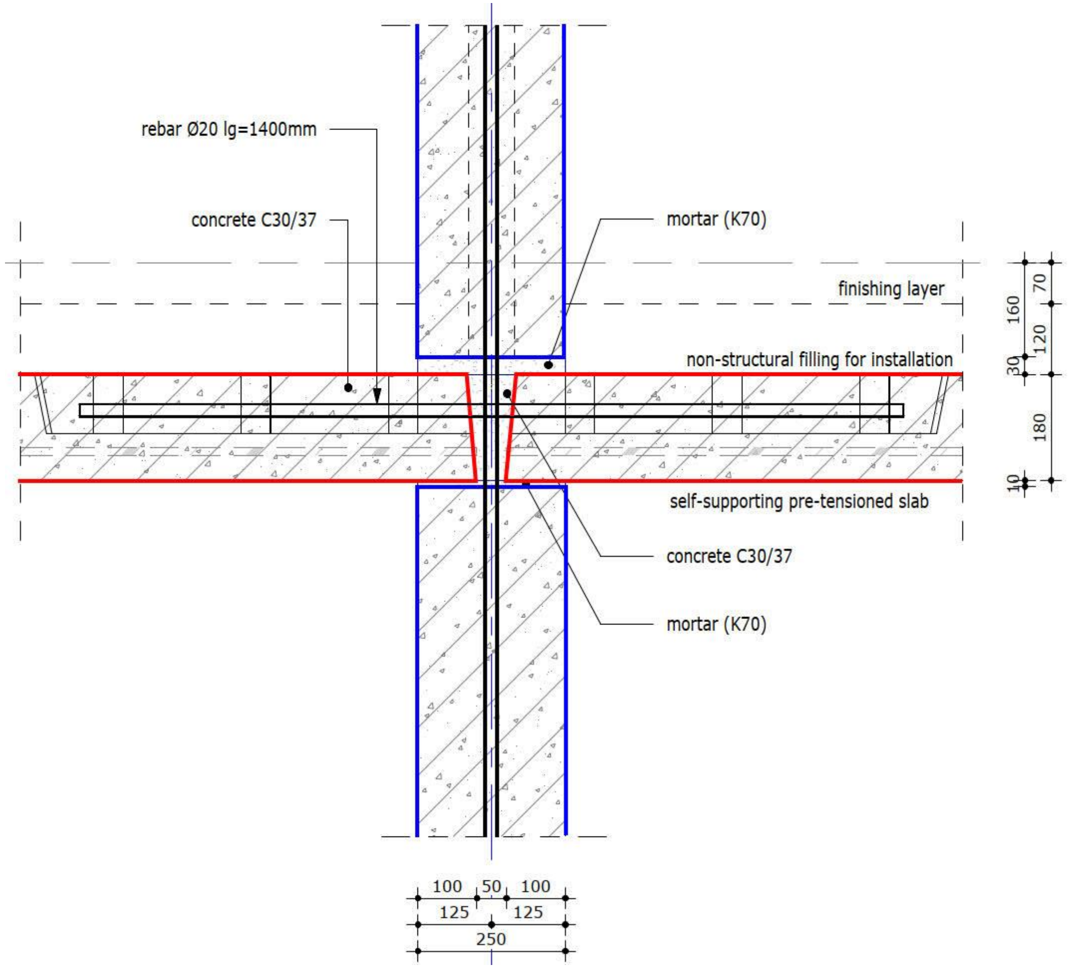


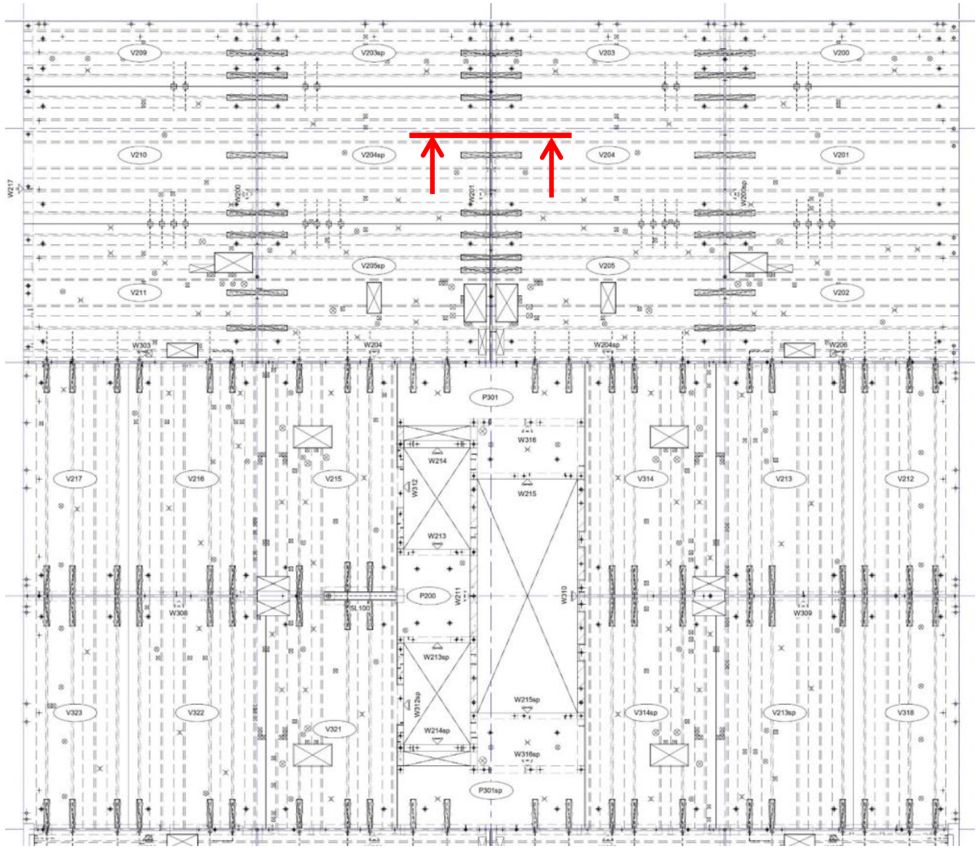


Element lay-out design

Specifics for element lay-out design

- Slab-Wall intersection for multi-storey buildings

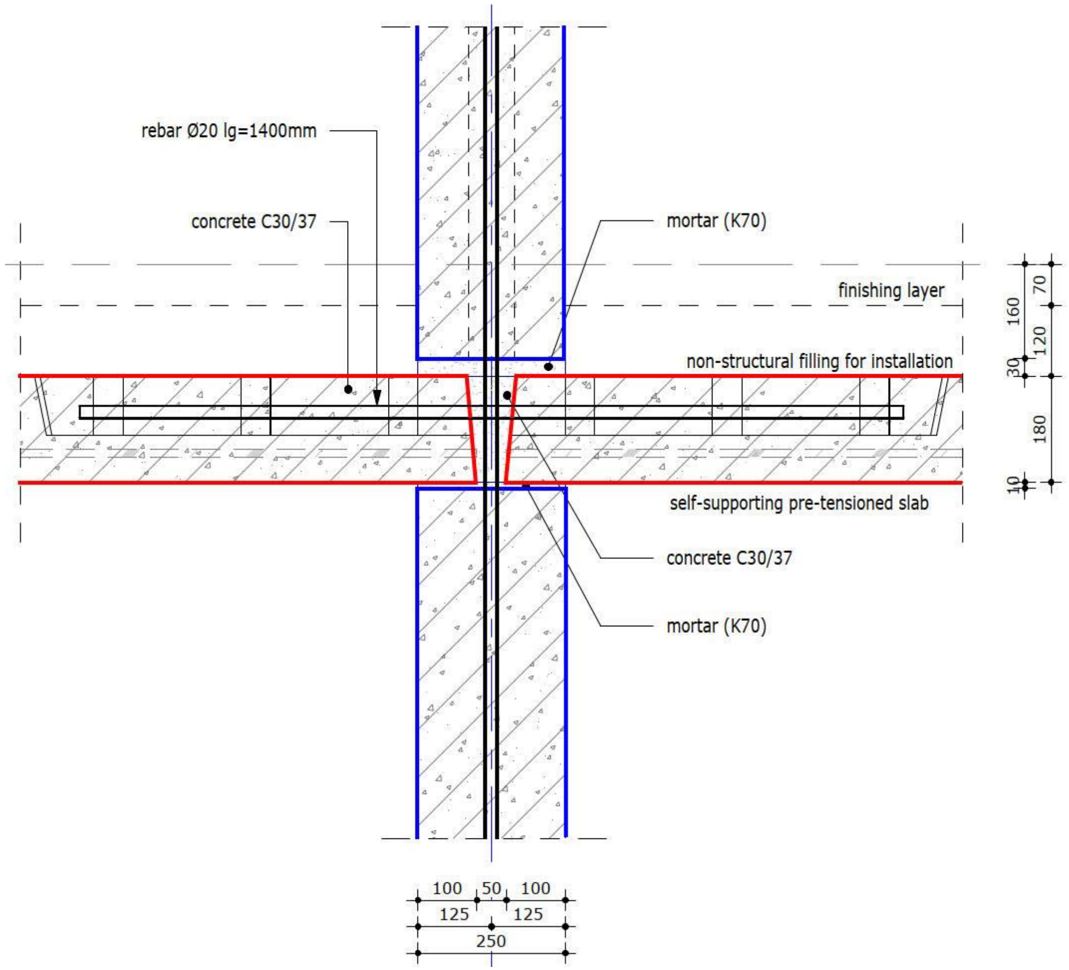


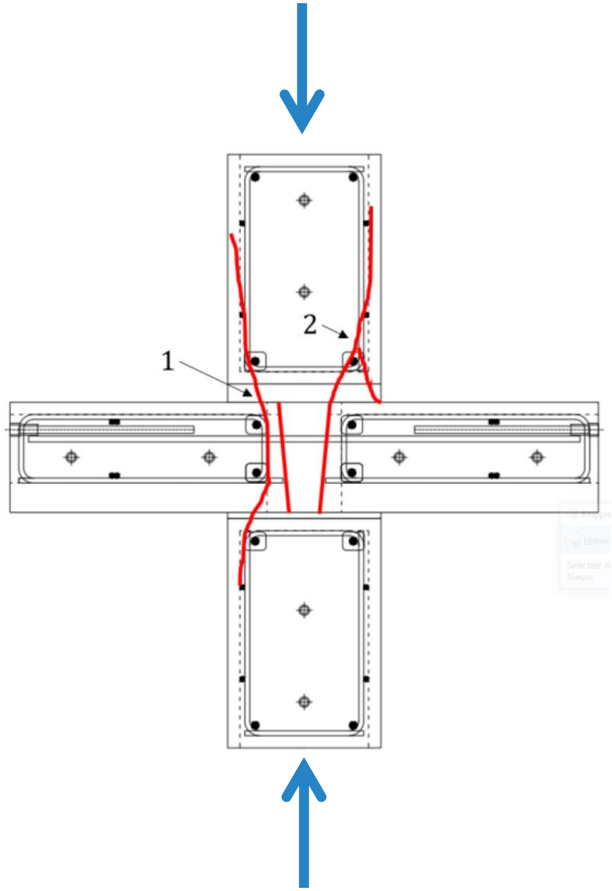


Element lay-out design

Specifics for element lay-out design

- Effect of vertical loads on slab-wall intersection





Element lay-out design

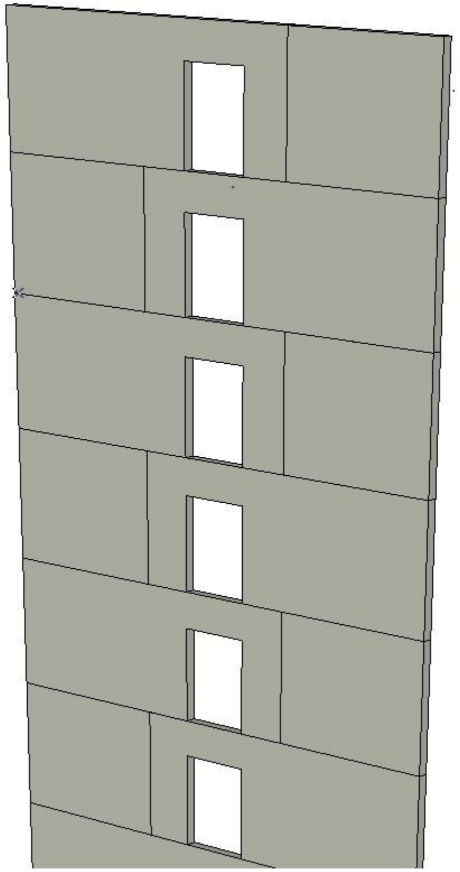
Specifics for element lay-out design

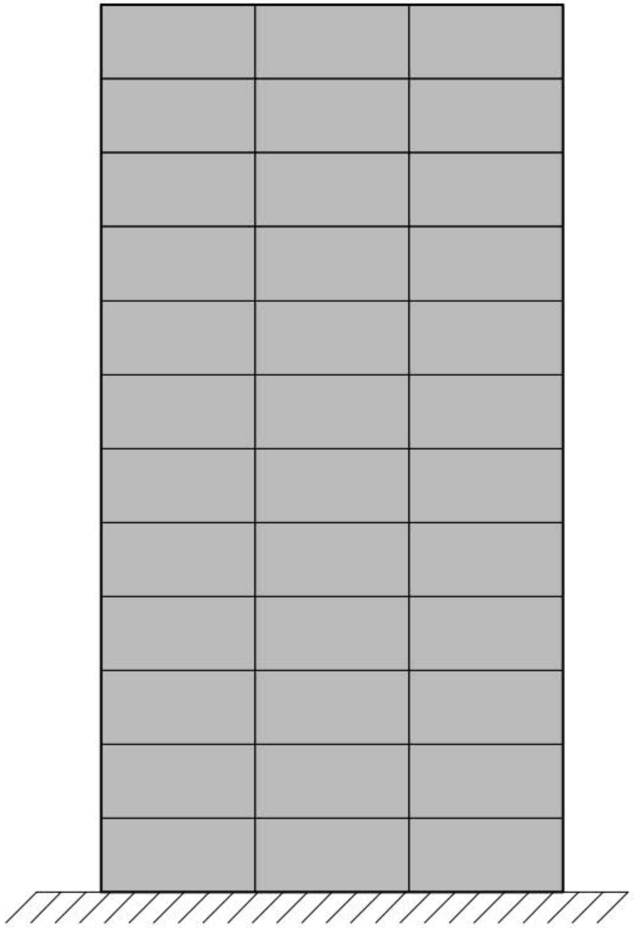
- Design of Hollow Core Slabs by Gabriel Tarta

Composition of precast concrete shear wall structures:

- Interlocking element lay-out or
- Vertical stacks of walls with vertical connection
- Horizontal mortar connection

- Limit number of beams and columns
- Make frames around window and door openings
- Avoid cast-in-situ concrete construction (with formwork) construction





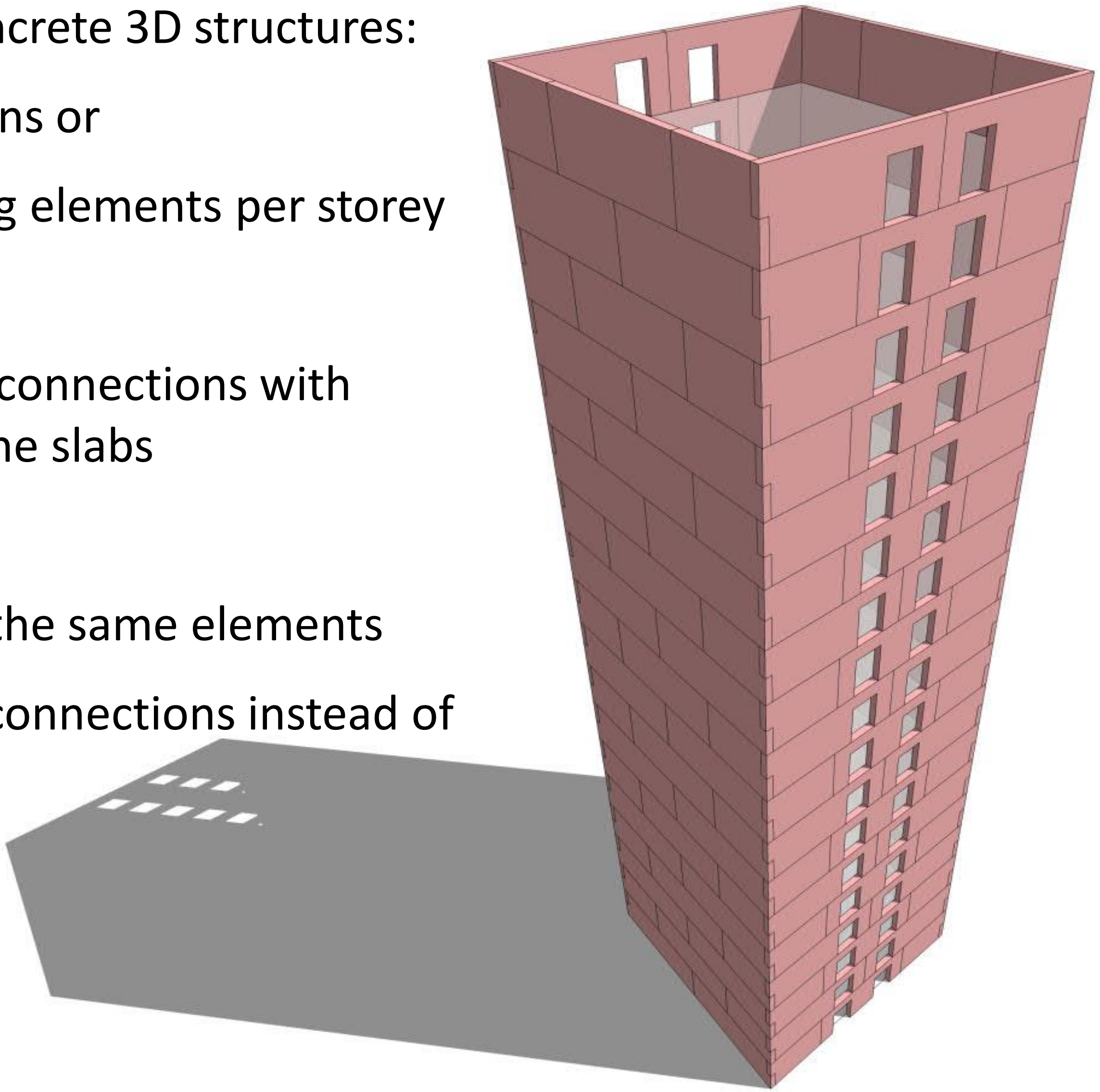
Element lay-out design

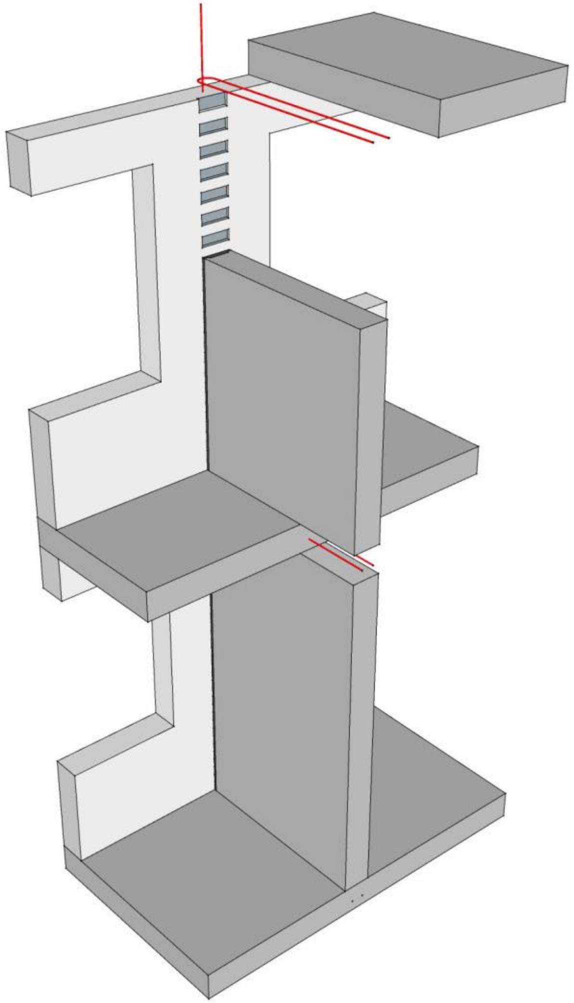
Composition of precast concrete 3D structures:

- Corbel corner connections or
- Corners with interlocking elements per storey

- Vertical profiled mortar connections with tying reinforcement in the slabs

- Make as far as possible the same elements
- Apply precast concrete connections instead of traditional connections





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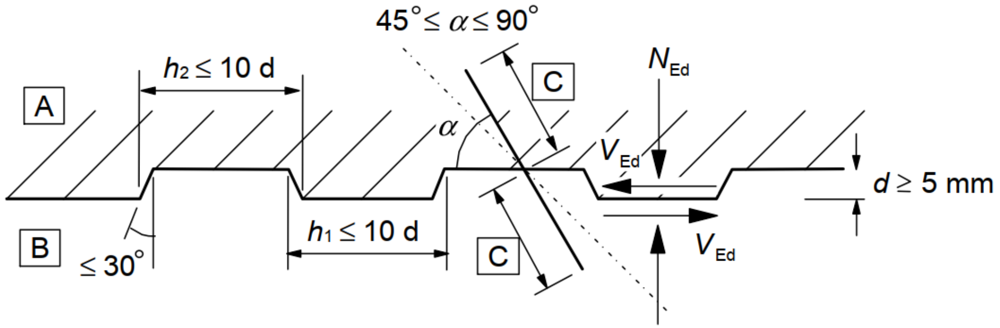
Eurocodes requirements

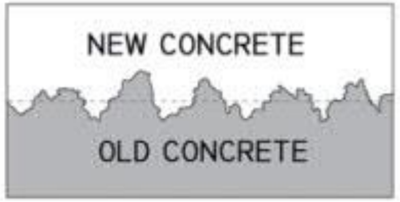
Interface shear transfer EN-1992-1-1:

- Section 6.2.5: Old-to-new concrete interfaces

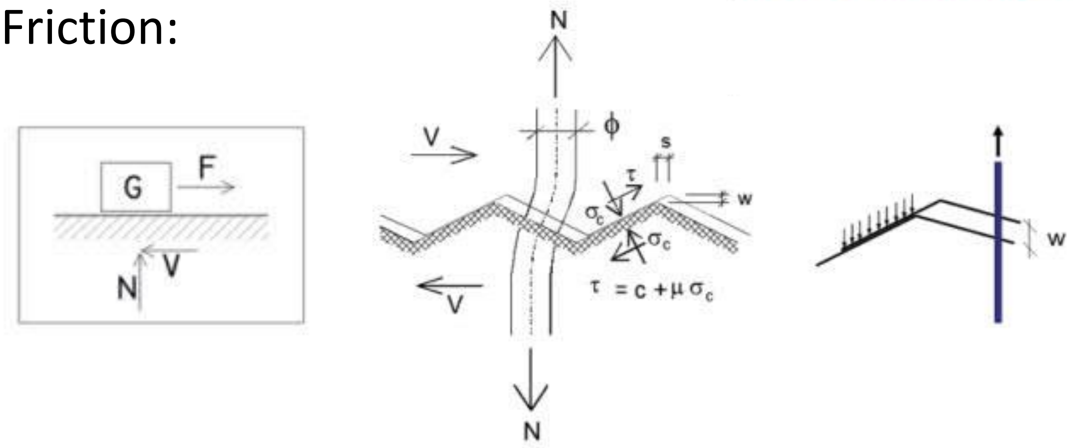


Individual shear mechanisms:

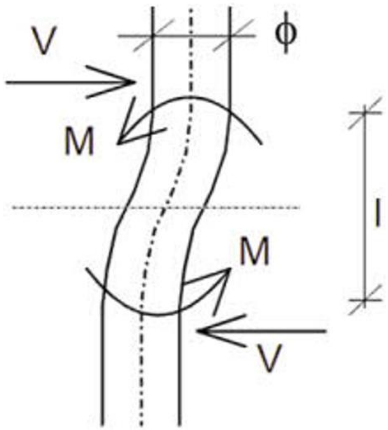
- Adhesive bonding + Mechanical interlock:



- Friction:

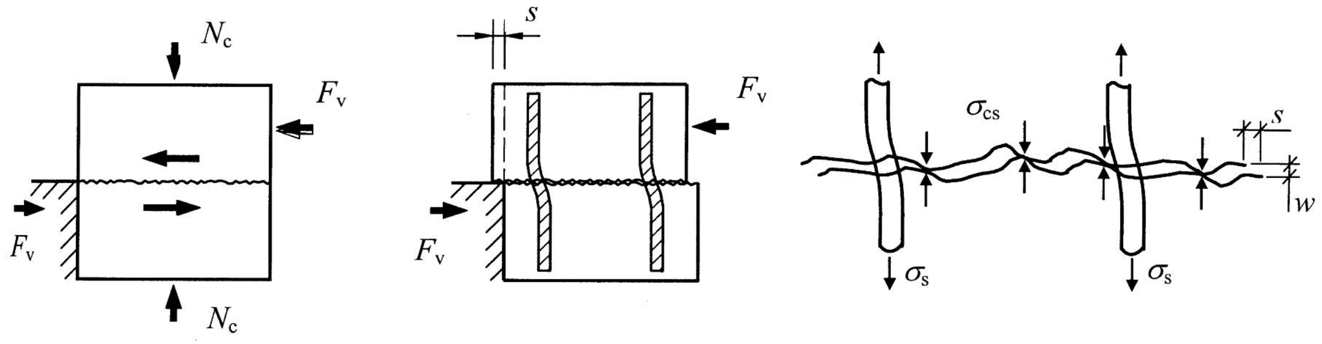


- Dowel action:



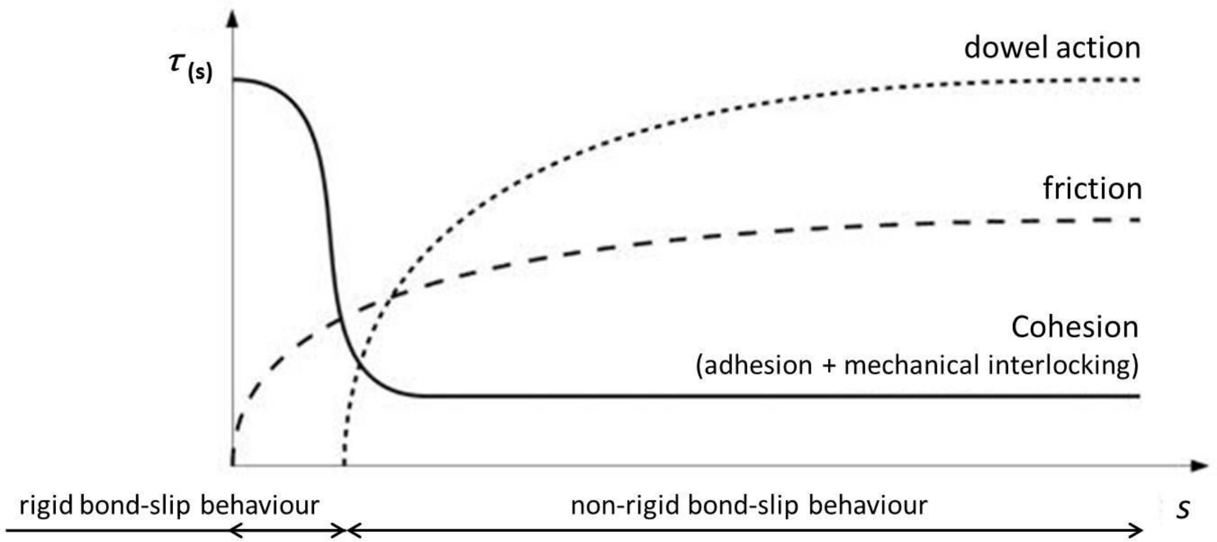
Eurocodes requirements

Interface shear transfer



Interaction between 3 mechanisms:

- Adhesive bonding + Mechanical interlock
- Friction
- Dowel action



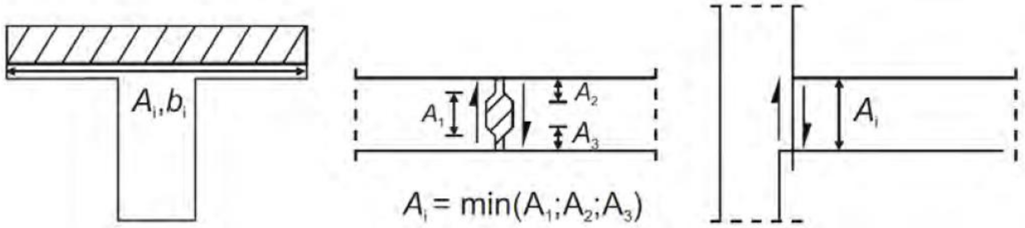
Modelcode 2010

- Rigid bond-slip behaviour (typically s ≤ 0,05 mm)
- Non-rigid bond-slip behaviour (0,5 ≤ s ≤ 1,5 mm)

Eurocodes requirements

Interface shear transfer

- 2^{nd} generation Eurocode 2: EN 1992-1-1, section 8.2.6



- Reinforcement **sufficiently** anchored for yield strength:

$$\tau_{Rdi} = c_{v1} \sqrt{(f_{ck})}/\gamma_C + \mu_v \sigma_n + \rho f_{yd} (\mu_v \sin\alpha + \cos\alpha) \leq \nu f_{cd} \qquad (8.55)$$

N.B.: Contribution of $c_{v1}\sqrt{(f_{ck})}/\gamma_c$ corresponds to $c f_{ctd}$ in current EC2

- Reinforcement **insufficiently** anchored for yield strength:

$$\tau_{Rdi} = c_{v2} \sqrt{(f_{ck})}/\gamma_C + \mu_v \sigma_n + k_t \rho f_{yd} \mu_v + k_f \rho \sqrt{(f_{yd} f_{cd})} \qquad (8.56)$$

	Equation 8.55		*Equation 8.56*		
Surface roughness	c_{v1}	μ_v	c_{v2}	k_t	k_f
very smooth	0,0095	0,5	0	0	1,5
smooth	0,075	0,6	0	0,5	1,1
rough	0,15	0,7	0,075	0,5	0,9
very rough	0,19	0,9	0,15	0,5	0,9
keyed	0,37	0,9	-	-	-

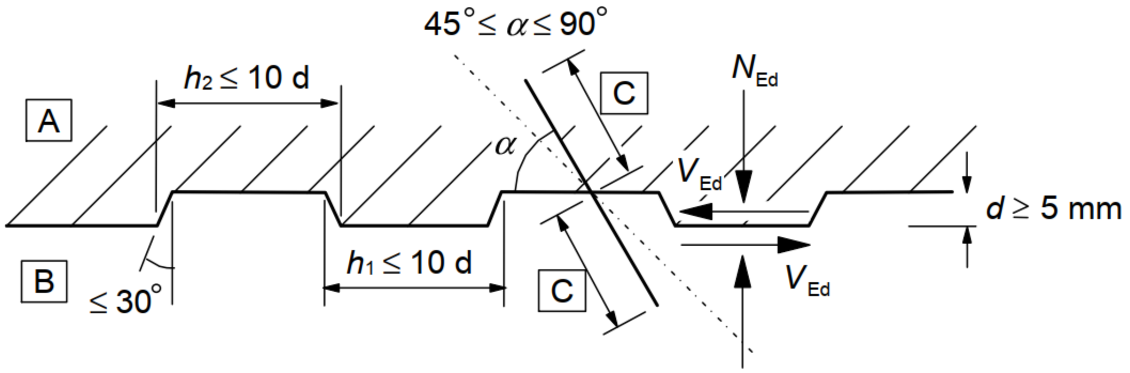
Note: The factors for keyed interfaces shall be applied for the area of each key considering its concrete strength.

Eurocodes requirements

Interface shear transfer

- 1st generation Eurocode 2: EN 1992-1-1, section 6.2.5

- Shear capacity V_{Rdi}
- Shear displacements (s) measured during tests
- Shear displacements (s) for ULS and SLS



- Shear stiffness $k_x = \frac{V_{Rdi}}{s}$

Displacements s (mm) for V_{Rdi}	Shear friction equation	ULS Slips (s)	SLS Slips (s)
- Strong bonding	$v_{Rdi} = cf_{ctd} + \mu\sigma_n \leq 0,5\ v\ f_{cd}$	0,6-1,0	0,2
- Weak bonding	$v_{Rdi} = \mu\sigma_n + \mu\rho f_{yd} \leq 0,5\ v\ f_{cd}$	1,5	1,0

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FEM global structural design

- Simplified 2D or Finite Element Method (FEM) based 3D global design?

  - Simplified/Manual 2D
    - "It's how we've done design for ages"
    - Basic
    - Simplifying assumptions

    - Robust
    - Easy to verify

  - FEM based 3D model
    - "Not trustworthy"
    - Complex
    - Spatial interaction

    - Sensitive to input
    - Difficult to verify

FEM global structural design

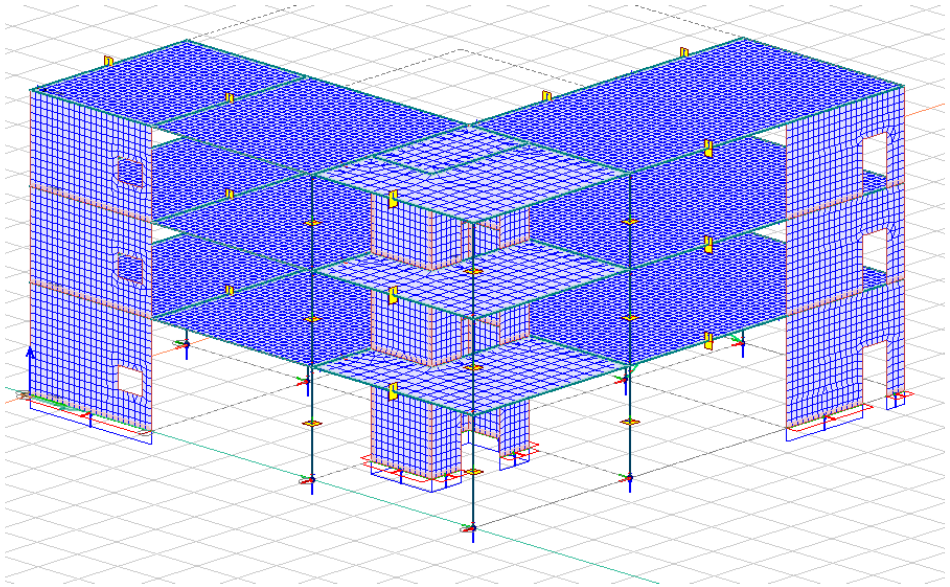
- Criteria for structural regularity in seismic design – allowed simplifications

Regularity		**Allowed Simplification**		**Behaviour factor**
Plan	**Elevation**	**Model**	**Linear-elastic Analysis**	**(for linear analysis)**
Yes	Yes	**Planar**	Lateral force	Reference value
Yes	No	**Planar**	Modal	Decreased value*
No	Yes	**Spatial**	Lateral force	Reference value
No	No	**Spatial**	Modal	Decreased value*

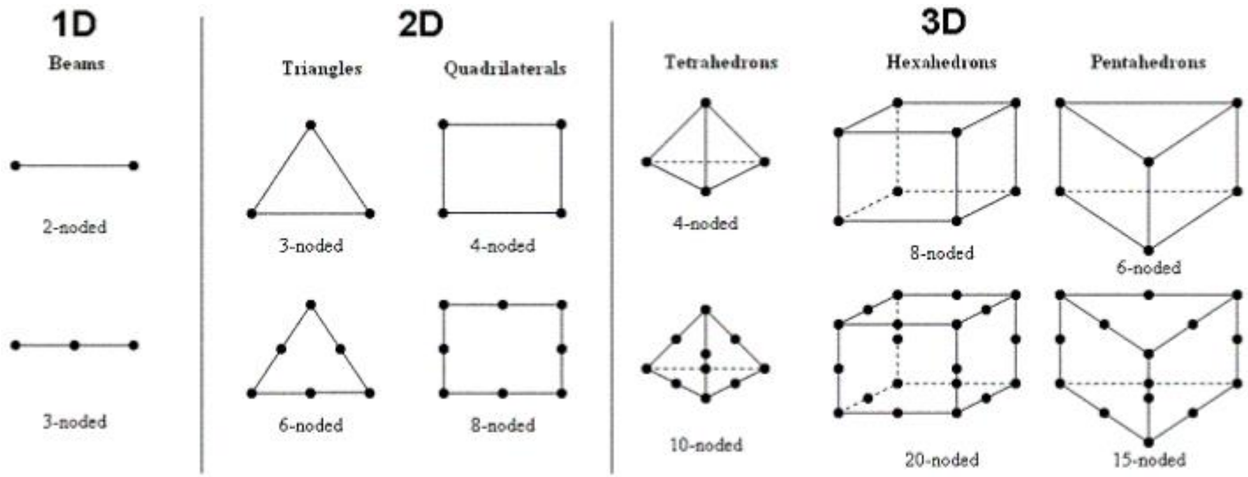
Consequences of structural regularity on seismic analysis and design (acc. to EN 1998)

FEM global structural design

- Finite Element Method
  - Subdivides a large system into smaller, simpler parts called finite elements;
  - Method consists in the assembly of a global stiffness matrix from the stiffness matrix of elements with consideration of boundary and support conditions;
  - The mathematical model of a structure **idealizes the building**



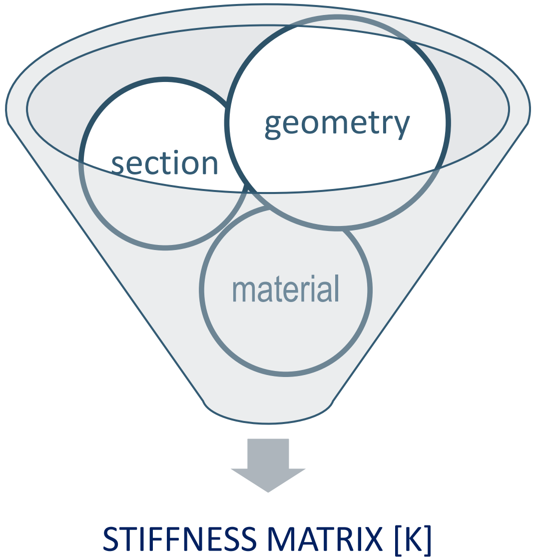
$$[\boldsymbol{F}] = [\boldsymbol{k}][\boldsymbol{\Delta}]$$

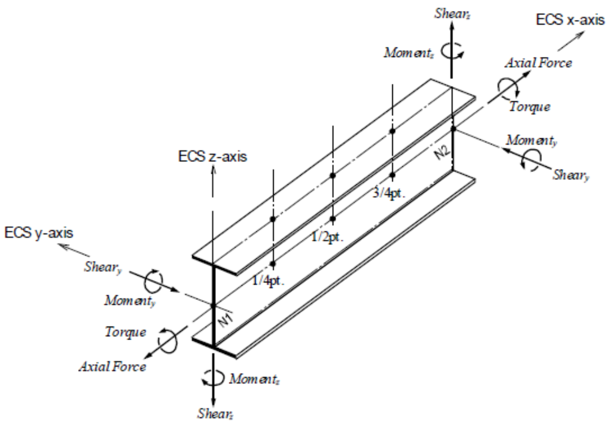


FEM global structural design

- Finite Element Method

$$[\boldsymbol{F}] = [\boldsymbol{k}][\boldsymbol{\Delta}]$$

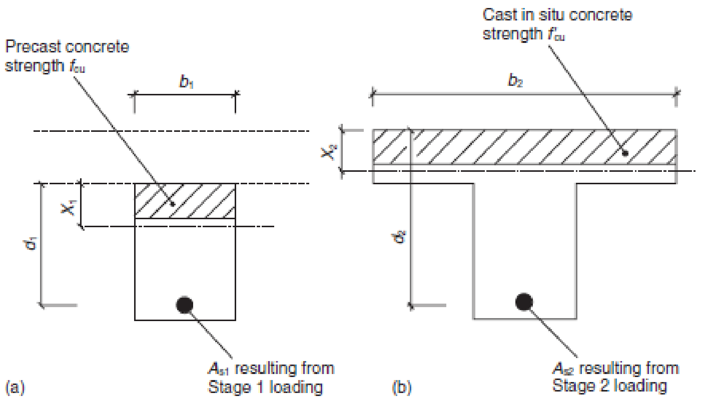


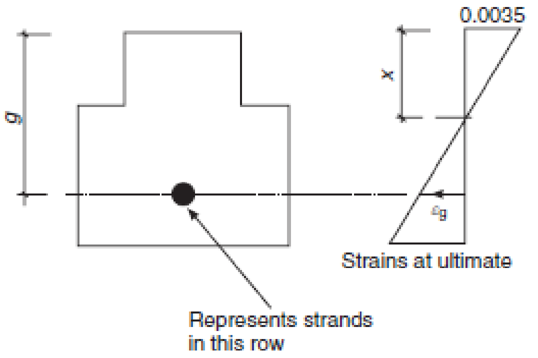


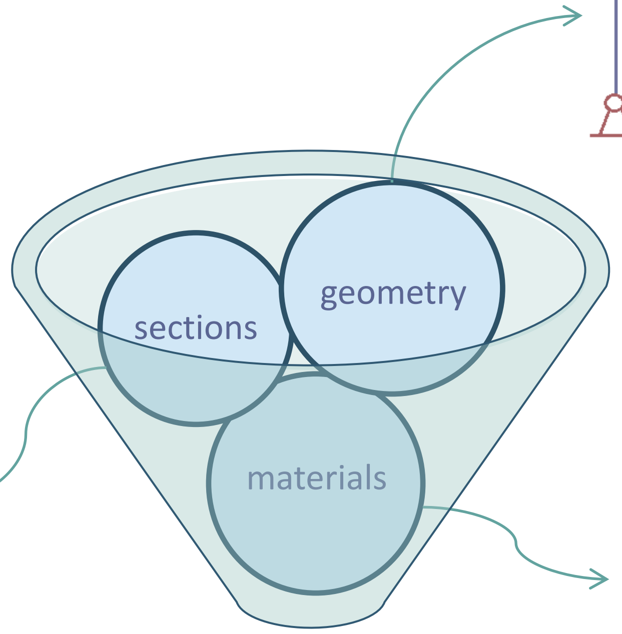
$$\begin{bmatrix} F_{x1} \\ F_{y1} \\ F_{z1} \\ M_{x1} \\ M_{y1} \\ M_{z1} \\ F_{x2} \\ F_{y2} \\ F_{z2} \\ M_{x2} \\ M_{y2} \\ M_{z2} \end{bmatrix} = E \begin{bmatrix}
\frac{A}{L} & 0 & 0 & 0 & 0 & 0 & -\frac{A}{L} & 0 & 0 & 0 & 0 & 0 \\
0 & \frac{12I_z}{L^3} & 0 & 0 & 0 & \frac{6I_z}{L^2} & 0 & -\frac{12I_z}{L^3} & 0 & 0 & 0 & \frac{6I_z}{L^2} \\
0 & 0 & \frac{12I_y}{L^3} & 0 & -\frac{6I_y}{L^2} & 0 & 0 & 0 & -\frac{12I_y}{L^3} & 0 & -\frac{6I_y}{L^2} & 0 \\
0 & 0 & 0 & \frac{J}{2(1+v)L} & 0 & 0 & 0 & 0 & 0 & -\frac{J}{2(1+v)L} & 0 & 0 \\
0 & 0 & -\frac{6I_y}{L^2} & 0 & \frac{4I_y}{L} & 0 & 0 & 0 & \frac{6I_y}{L^2} & 0 & \frac{2I_y}{L} & 0 \\
0 & \frac{6I_z}{L^2} & 0 & 0 & 0 & \frac{4I_z}{L} & 0 & -\frac{6I_z}{L^2} & 0 & 0 & 0 & \frac{2I_z}{L} \\
-\frac{A}{L} & 0 & 0 & 0 & 0 & 0 & \frac{A}{L} & 0 & 0 & 0 & 0 & 0 \\
0 & -\frac{12I_z}{L^3} & 0 & 0 & 0 & -\frac{6I_z}{L^2} & 0 & \frac{12I_z}{L^3} & 0 & 0 & 0 & -\frac{6I_z}{L^2} \\
0 & 0 & -\frac{12I_y}{L^3} & 0 & \frac{6I_y}{L^2} & 0 & 0 & 0 & \frac{12I_y}{L^3} & 0 & \frac{6I_y}{L^2} & 0 \\
0 & 0 & 0 & -\frac{J}{2(1+v)L} & 0 & 0 & 0 & 0 & 0 & \frac{J}{2(1+v)L} & 0 & 0 \\
0 & 0 & -\frac{6I_y}{L^2} & 0 & \frac{2I_y}{L} & 0 & 0 & 0 & \frac{6I_y}{L^2} & 0 & \frac{4I_y}{L} & 0 \\
0 & \frac{6I_z}{L^2} & 0 & 0 & 0 & \frac{2I_z}{L} & 0 & -\frac{6I_z}{L^2} & 0 & 0 & 0 & \frac{4I_z}{L}
\end{bmatrix} \begin{bmatrix} u_1 \\ v_1 \\ w_1 \\ \theta_{x1} \\ \theta_{y1} \\ \theta_{z1} \\ u_2 \\ v_2 \\ w_2 \\ \theta_{x2} \\ \theta_{y2} \\ \theta_{z2} \end{bmatrix}$$

FEM global structural design

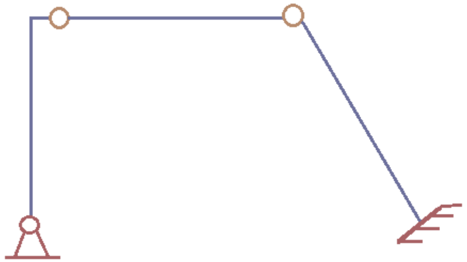
- FEM – Structural system

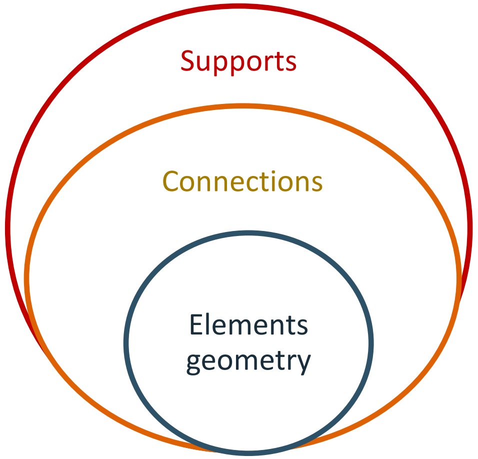


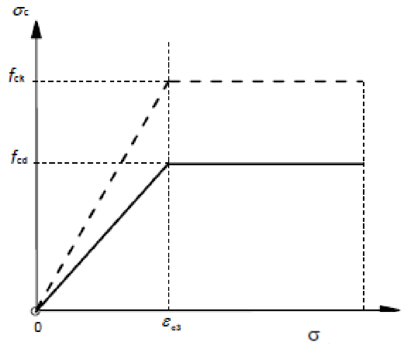


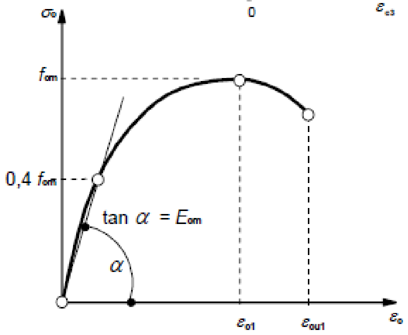


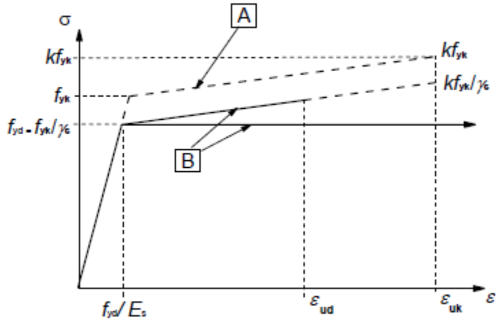
MATHEMATICAL MODEL OF THE STRUCTURE





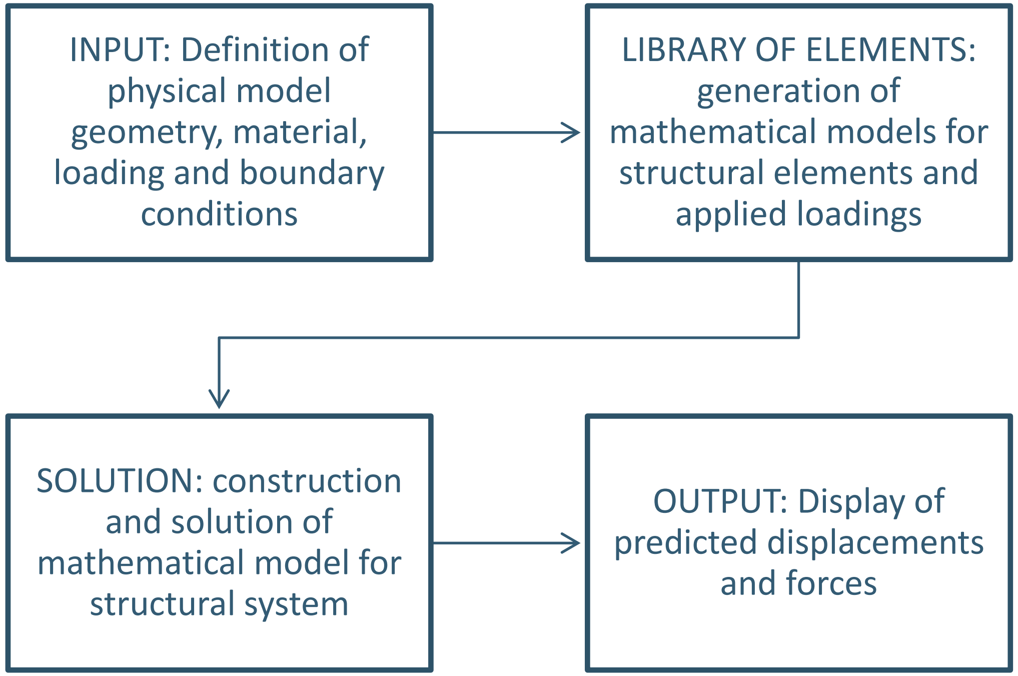






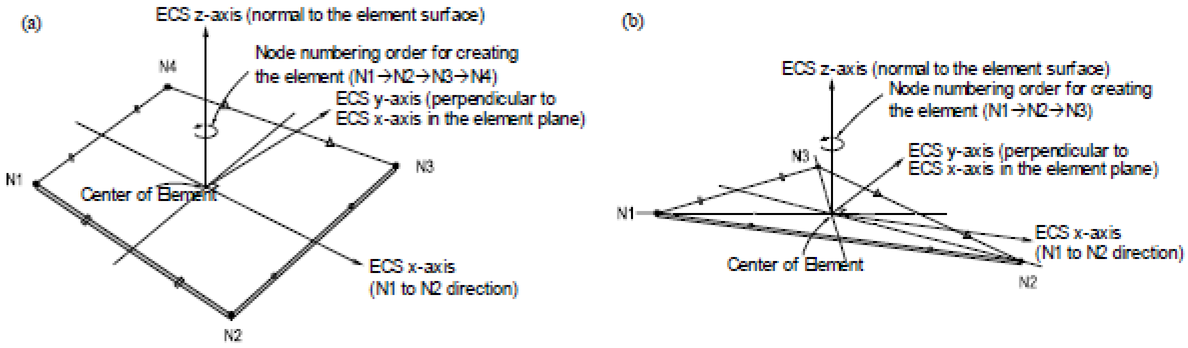
FEM global structural design

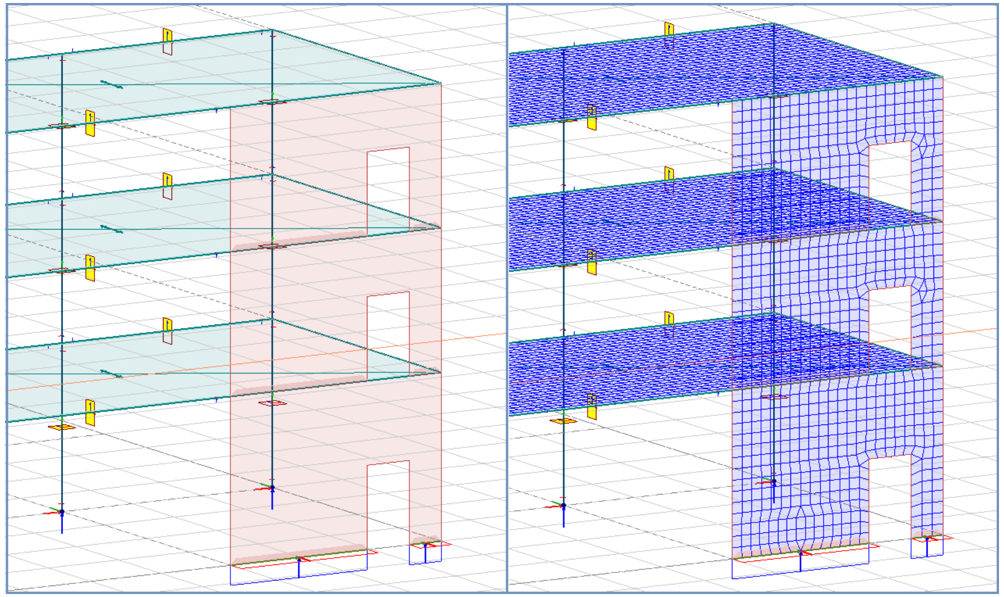
- Finite Element Method
  - General elements:
    - Truss element (1D)
    - Beam element (1D)
    - Plane element (2D)
    - Volumetric elements (3D)



FEM global structural design

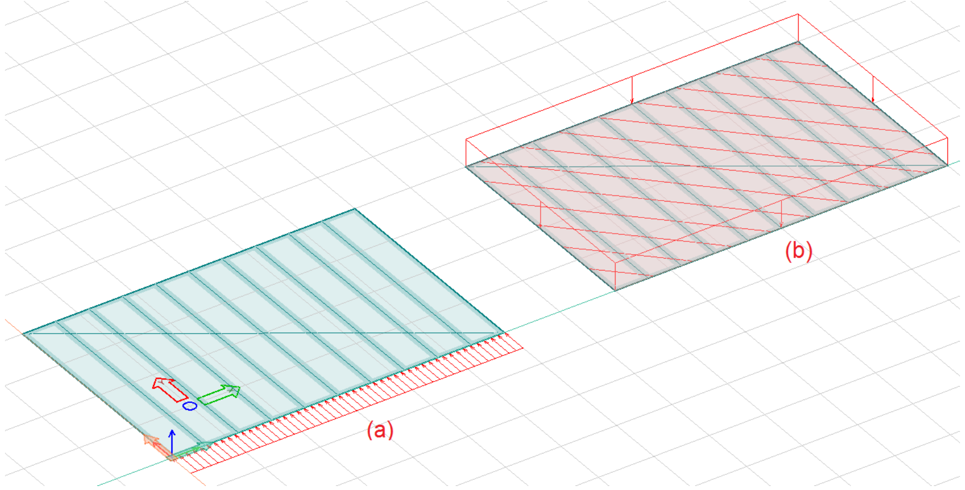
- Finite Element Method
  - Plane elements
    - 2D elements: slabs, walls
    - Depending on their type and complexity:
      - In-plane tension/compression
      - In-plane/out of plane shear
      - Out-of-plane bending
    - In FEM based methods meshing is prerequisite to analysis

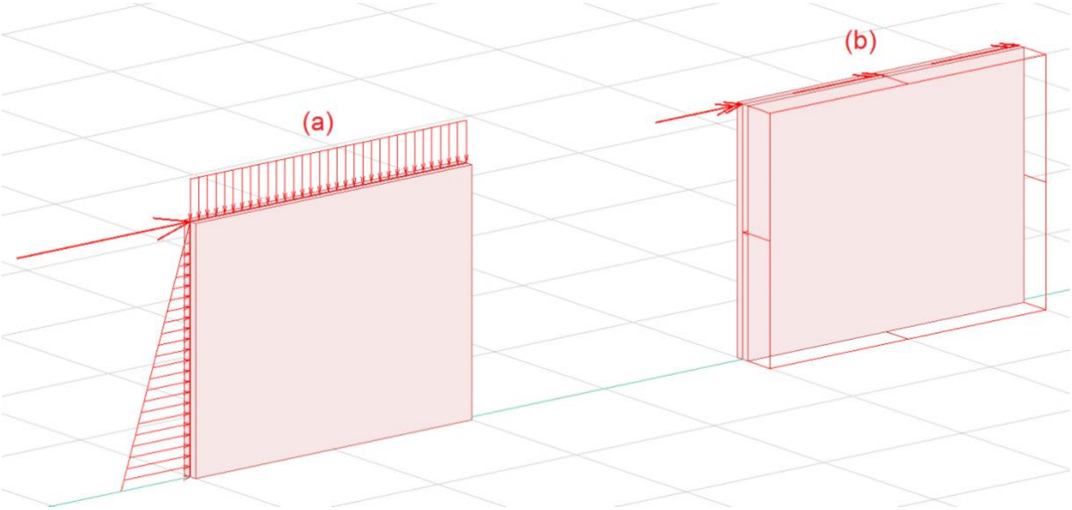


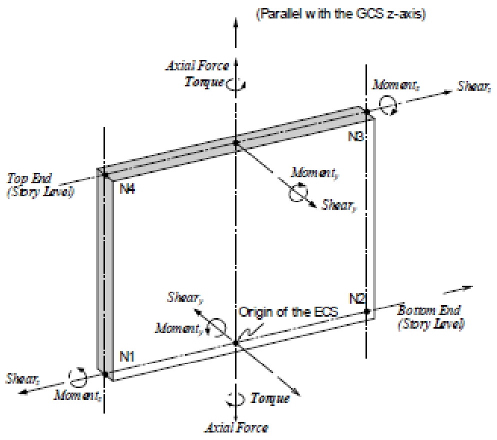


FEM global structural design

- Finite Element Method
  - Plane elements
    - 2D elements: slabs, walls
    - Depending on their type and complexity:
      - In-plane tension/compression
      - In-plane/out of plane shear
      - Out-of-plane bending

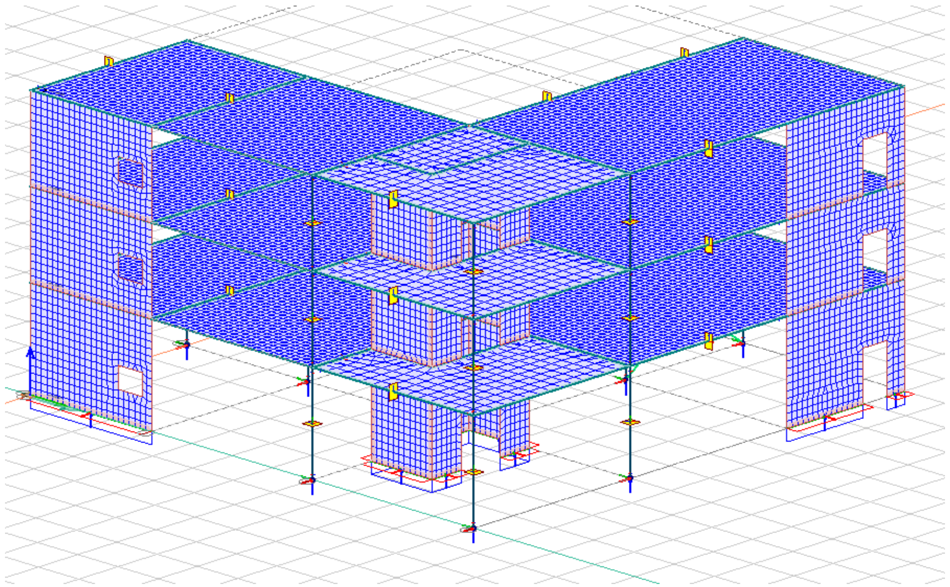






FEM global structural design

- Finite Element Method
  - Finite element mesh
    - Smaller mesh sizes usually provide more accurate results
    - Mesh size depends on the considered problem
    - „Rule of thumb” optimal mesh sizes for global stability analysis:
      - 2D elements: 2 or 3 x Element thickness
      - 1D elements: Length / 6 (for stability analysis)
    - „Rule of thumb” optimal mesh sizes for element calculation:
      - 2D elements: Equal or less than Element thickness



FEM global structural design

- materials
- geometry
- sections
- loads

MODEL

ANALYSIS

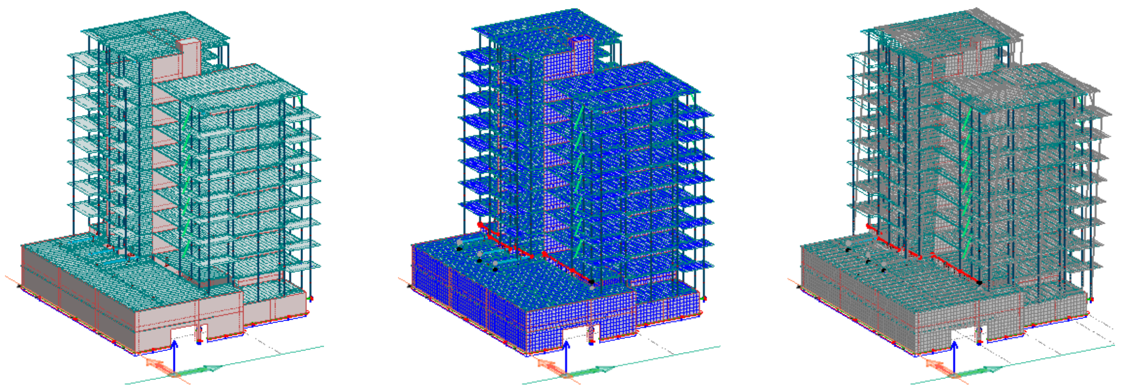
- static
- stability
- modal
- seismic

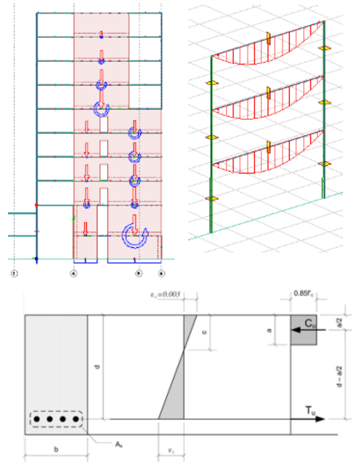
- global verifications
- overall assessment

STRUCTURAL RESPONSE

DESIGN

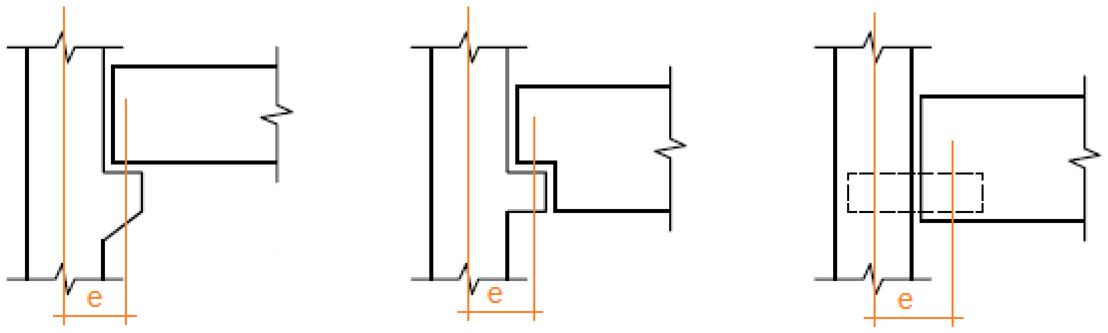
- element design
- connection design
- ULS/SLS verifications

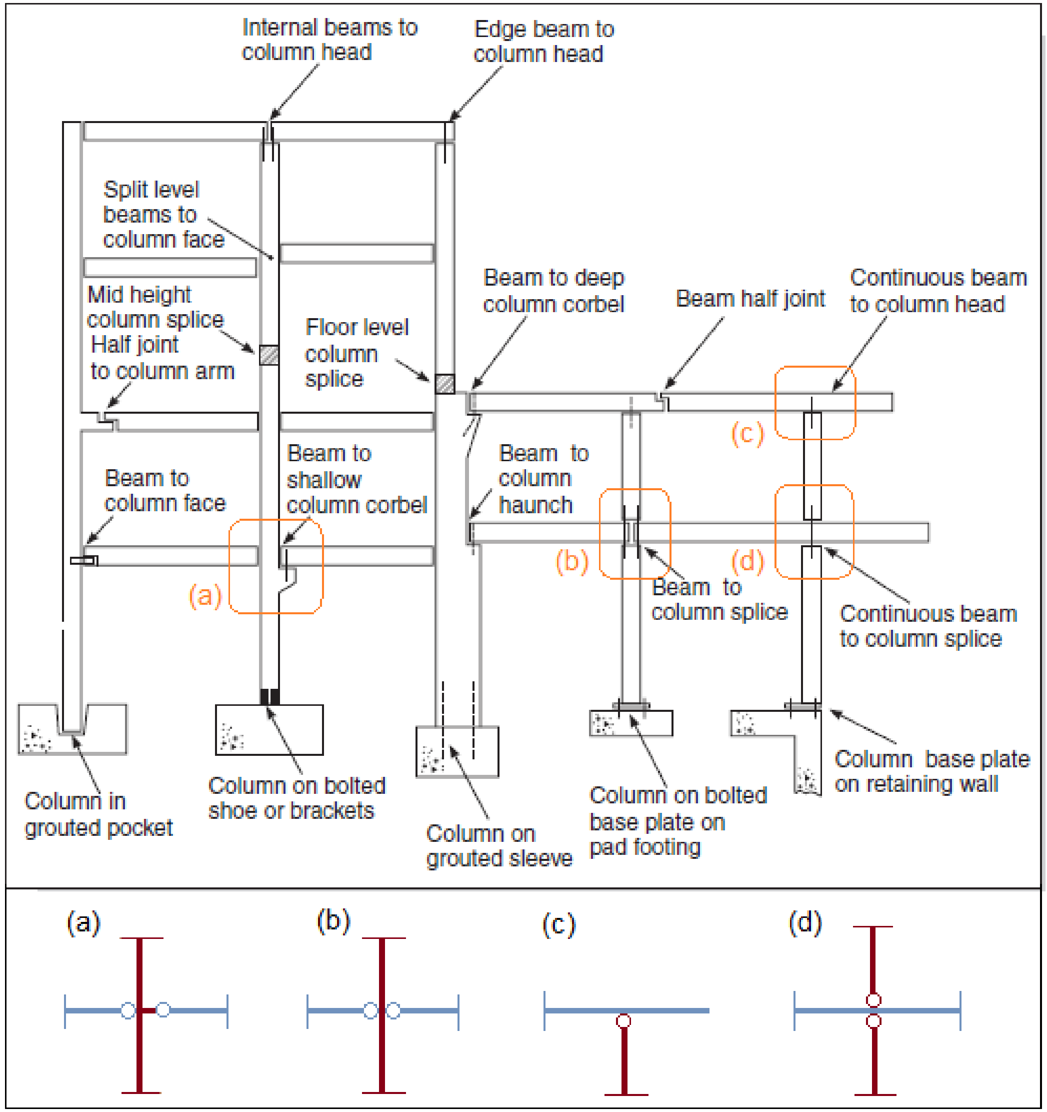




FEM global structural design

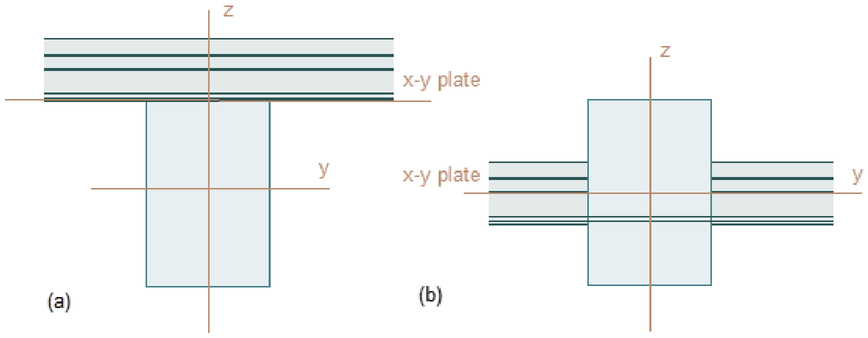
- FEM – Precast connections
  - Beam-column
    - Commonly precast beam to column connections are considered fixed or hinged
    - Important to account for:
      - Geometry of the connection
      - Eccentricity between the axis of the column and application point (center of bearing pad)



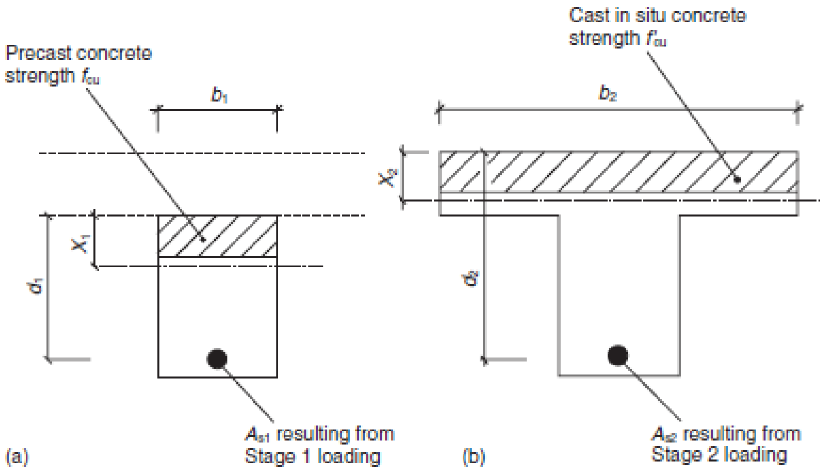


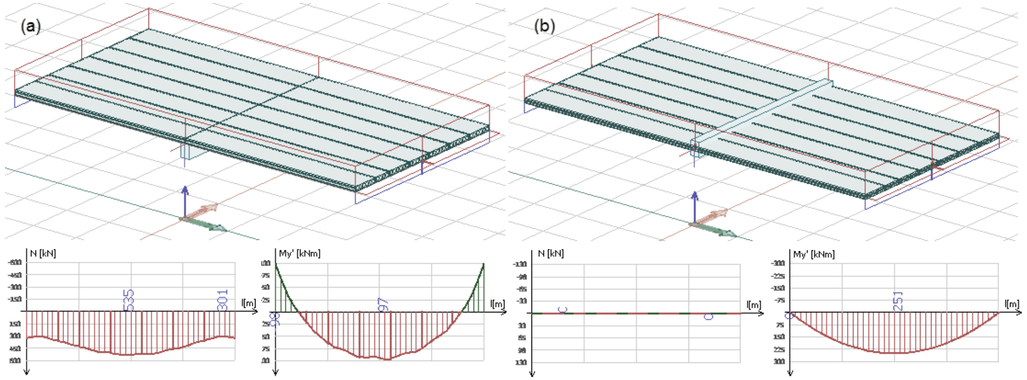
FEM global structural design

- FEM – Precast connections
  - Beam-slab
    - “composite” vs. “non-composite” behavior
    - “composite” behavior yields unrealistic structural response
    - “non-composite” behavior best suits precast structures
    - Recommendations:
      - Axis of elements should be kept in the centroid of the section
      - Median plane of plate elements should be kept in the middle of the section)



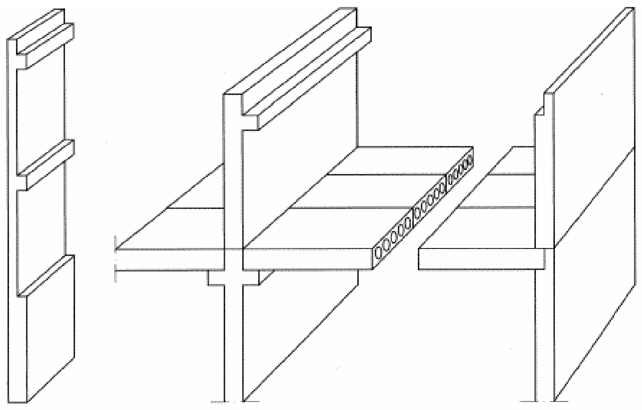
    - c

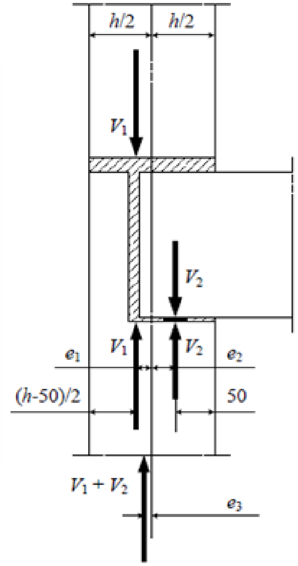




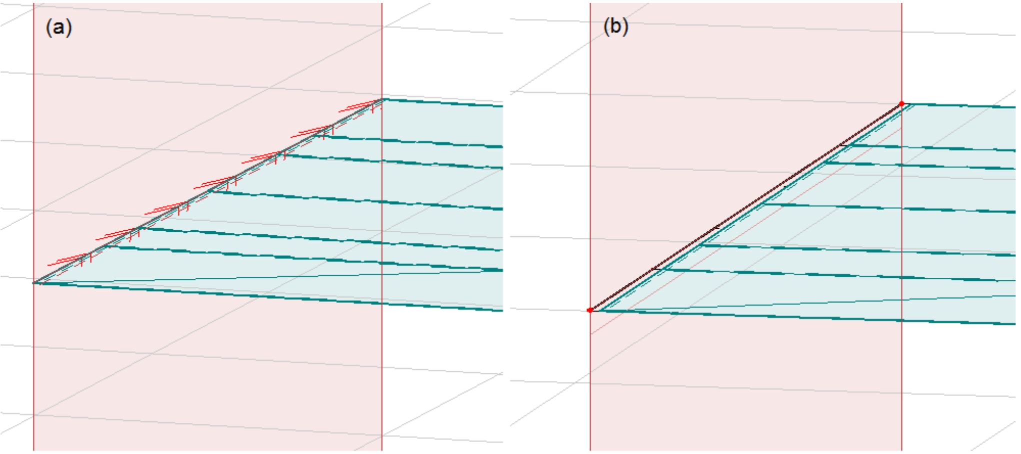
FEM global structural design

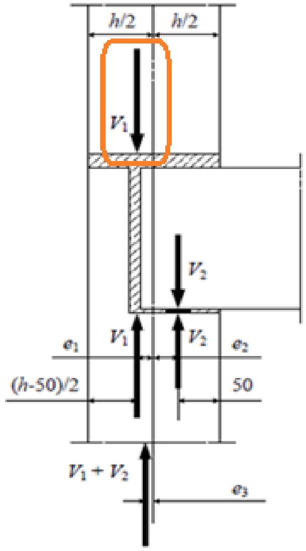
- FEM – Precast connections
  - Wall to wall connections
    - “theoretical” hinge models have restricted applicability
    - Rough evaluation of connection stiffness is recommended
    - Eccentricity of internal compression forces (V1) is difficult to consider in the model





  - Slab to wall connections
    - Modelled as “theoretical” hinge
    - Important to account for:
      - Geometry of the connection
      - Eccentricities





  - Should eccentricities be modeled in global models?

FEM global structural design

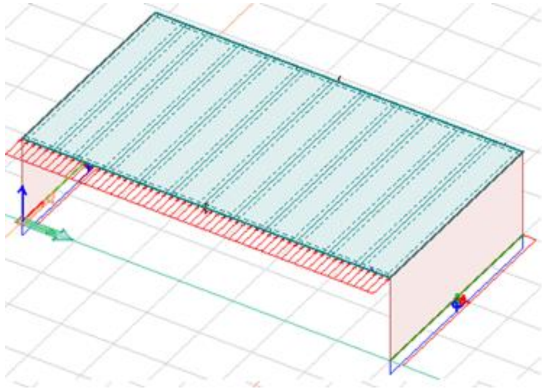
- Precast slabs
  - Focus on specifics of hollow core (HC) slabs

  - In-plane behavior models
    - Lateral load distribution
    - Flexible diaphragm
    - Rigid diaphragm

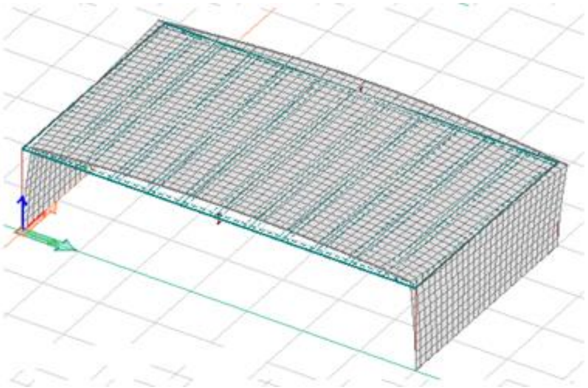
  - Out-of-plane behavior
    - Gravitational load distribution
    - Transverse load distribution according to EN1168 Annex C

FEM global structural design

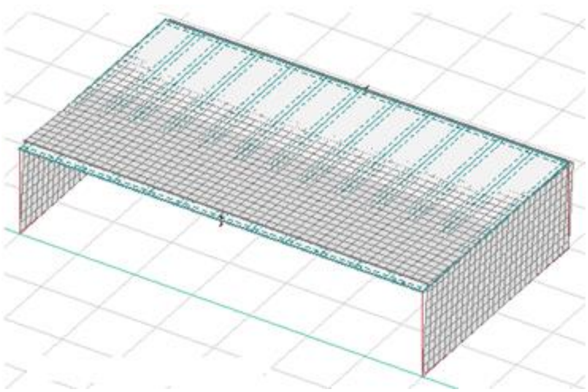
- Precast slabs
  - In-plane behavior analytical models
    - Flexible diaphragm
      - Slab’s stiffness is considered in the global stiffness matrix
      - Distribution of lateral forces is influenced by the in-plane stiffness of the slab
    - Rigid diaphragm
      - Slab’s stiffness is not considered
      - The slab acts as an infinite stiff horizontal element
      - Distribution of lateral forces is not influenced by the in-plane stiffness of the slab
      - Fundamental assumption of the classical 2D design approach



FEM model



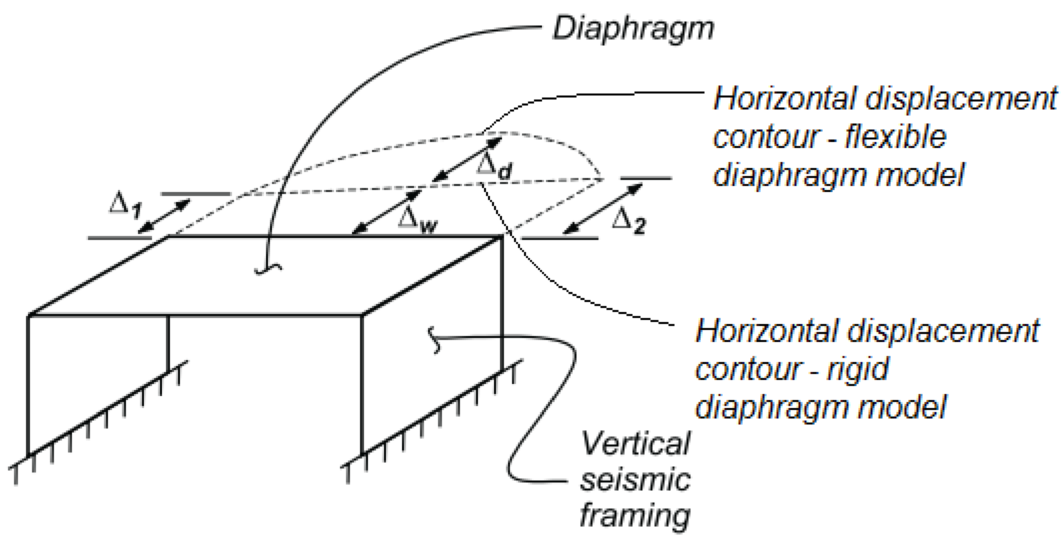
Flexible diaphragm



Rigid diaphragm

FEM global structural design

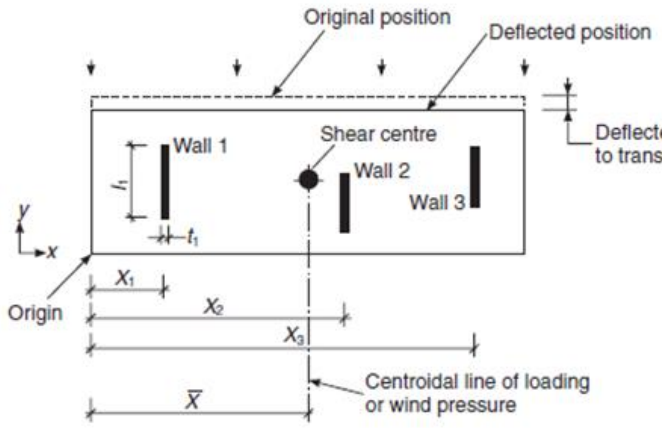
- Precast slabs – diaphragm models
  - Rigid or flexible diaphragm model?
    - Eurocode 2
      - No clear recommendations
      - Rigid diaphragm model seems to be universally suitable
    - Eurocode 8
      - Clear limitations for the rigid diaphragm model
      - Rigid diaphragm can be used when max 10% difference between horizontal displacements of the two models



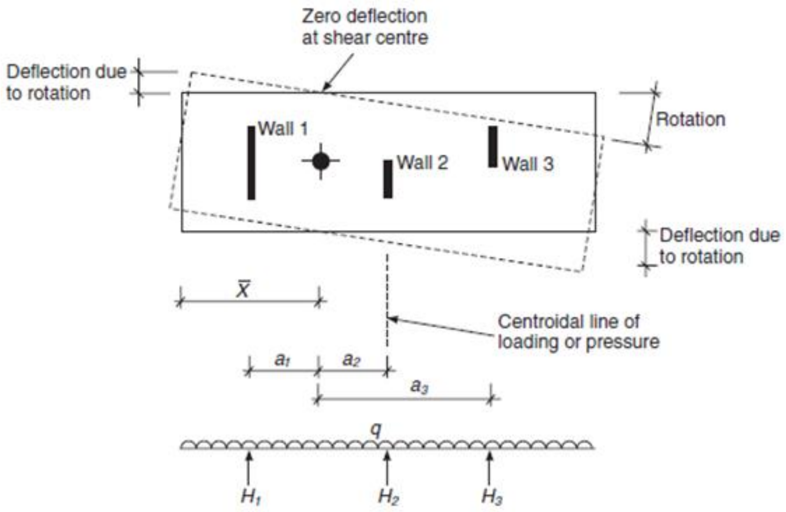
Comparison between rigid and flexible diaphragm maximum lateral displacement

FEM global structural design

- Precast slabs – rigid diaphragm model
  - Distribution of horizontal loading
    - Analysis of a multi-wall system of bracings



(a) Definitions & floor deflections due to translations



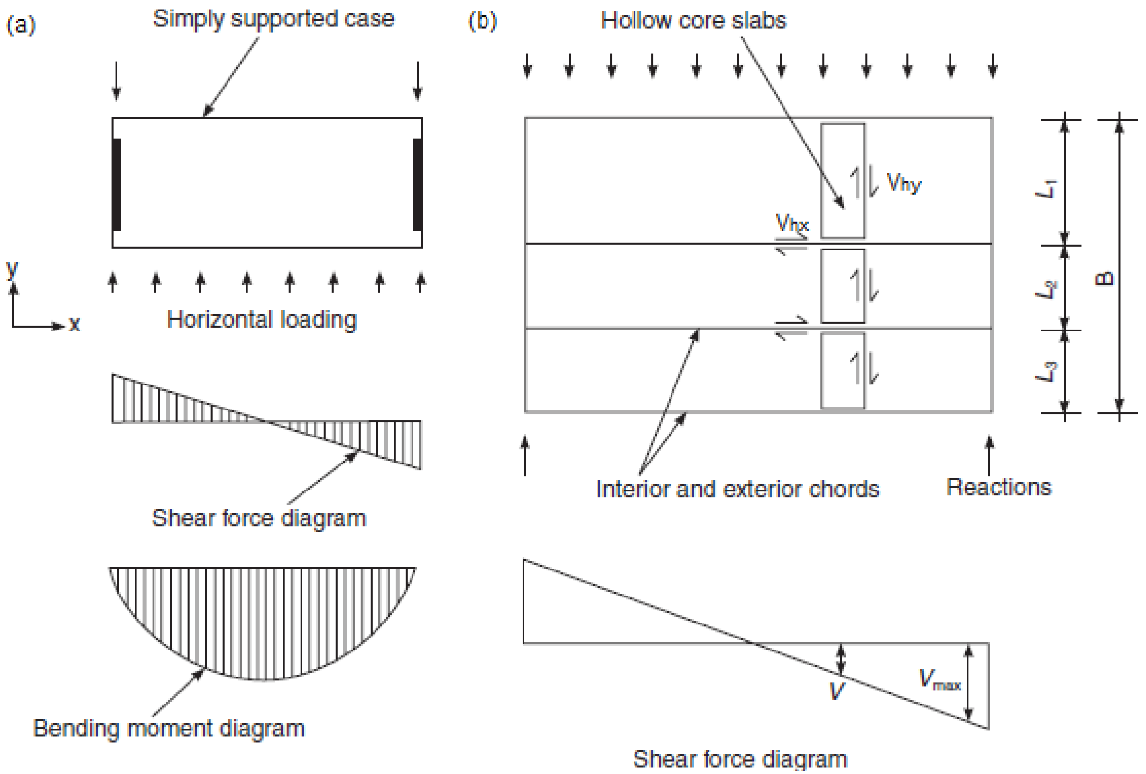
(b) Floor deflections due to rotations

$$\frac{H_n}{H} = \frac{E_n I_n}{\sum E_i I_i} \pm \frac{e E_n I_n \; a_n}{\sum E_i I_i \; a_i^2}$$

*Kim S. Elliott, *Precast Concrete Structures*

FEM global structural design

- Precast slabs – rigid diaphragm model
  - Internal forces in the diaphragm connections
    - Shear in the longitudinal and transversal directions of the floor units



$$V_{hy} = V_h \cdot \frac{L_1}{B} \; [N]$$

$$V_{hx} = V_h \cdot \frac{S}{I} [N/mm]$$

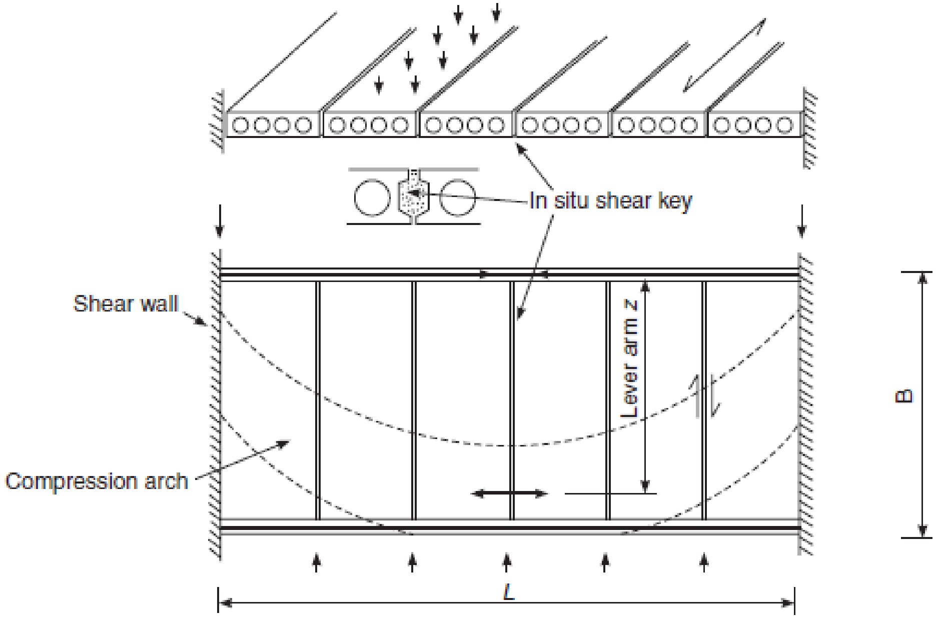
$$V_{hx} = V_h \cdot \frac{6 \cdot L_1 \cdot (B - L_1)}{B^3} \; [N/mm]$$

(a) Shear and bending moment diagrams in slab
(b) Shear in HC connections

*fib bulletin 6, *Special design considerations for precast prestressed hollow core floors*

FEM global structural design

- Precast slabs – rigid diaphragm model
  - Internal forces in the diaphragm connections
    - Tie forces



$$T_h = T_q + T_b = \frac{V_h}{(n+1)\acute{\mu}} + \frac{M_h}{z}$$

- Bending - deep beam model B/L<1

B/L	z/B
<0.5	0.9
0.5<1.0	0.8

$$T_b = \frac{M_h}{z}$$

- Bending - strut-and tie arch model B/L>1

$$T_b = 0.5\, V_h/(B/L)$$

- Shear wedging + shear friction

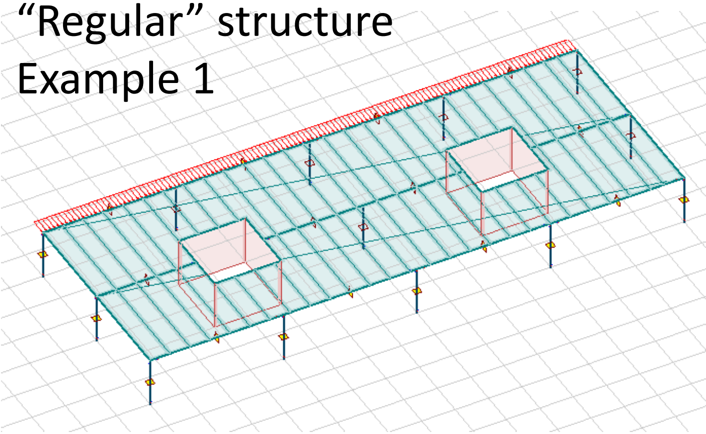
$$T_q = \frac{V_h}{\acute{\mu}}$$

*fib bulletin 6, *Special design considerations for precast prestressed hollow core floors*

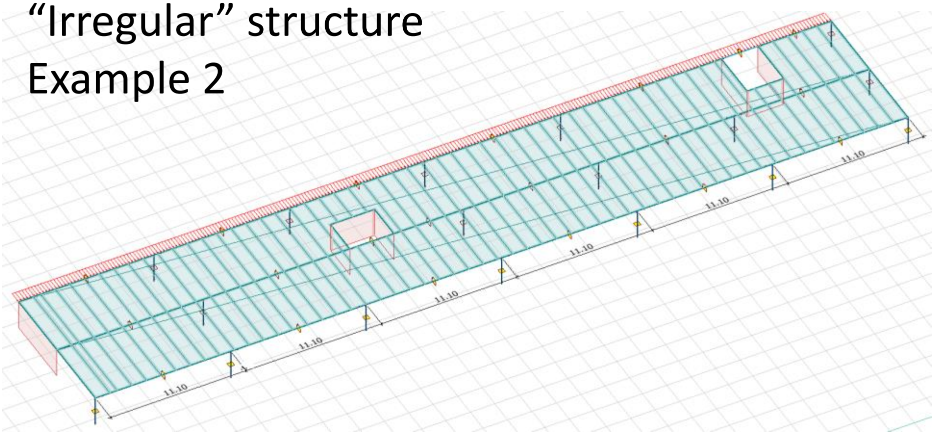
FEM global structural design

- Precast slabs – examples
  - Floor diaphragm behavior - comparative study

"Regular" structure
Example 1



"Irregular" structure
Example 2

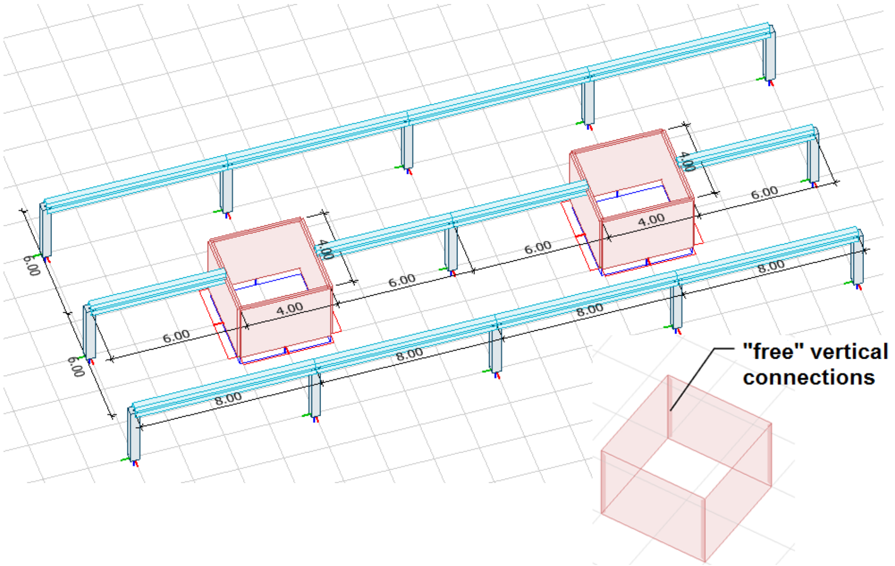


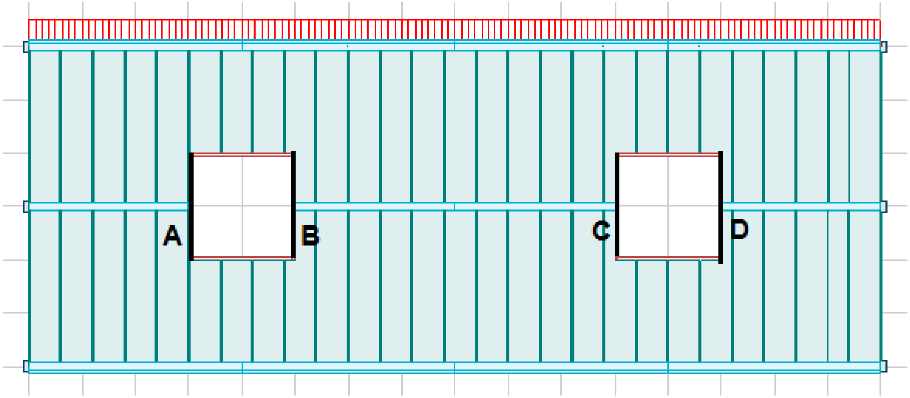
- Design approaches:
  A. manual calculation (rigid diaphragm)
  B. FEM analysis (rigid diaphragm)
  C. FEM analysis (flexible diaphragm)

- Structural output :
  - shear in wall panels/ distribution of lateral forces
  - internal forces in the HC slab connections

FEM global structural design

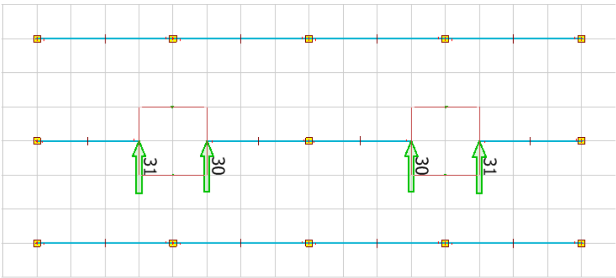
- Precast slabs – examples
  - Example 1: regular structure
    - Structural system:
      - 1 story (3m high)
      - untopped two bay HD-F 120-20 slab
      - precast concrete beams
      - (inverted T 500x500-250-400mm)
      - double hinged columns (400x400mm)
    - Load:
      - 4 kN/m linear distributed wind load)
    - No vertical edge connectivity between the longitudinal and transversal wall panels!
    - Supports are "rigid"



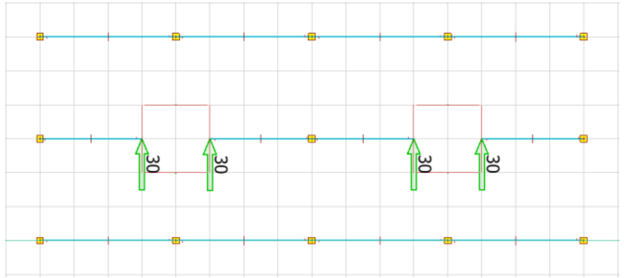


FEM global structural design

- Precast slabs – examples
  - Example 1: regular structure
    - Structural output comparison: lateral force distribution & lateral displacements - rigid vs. flexible diaphragm model

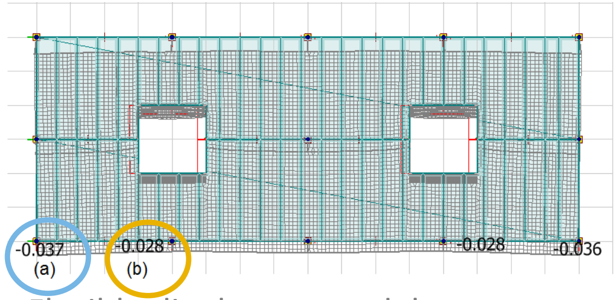


Flexible diaphragm model

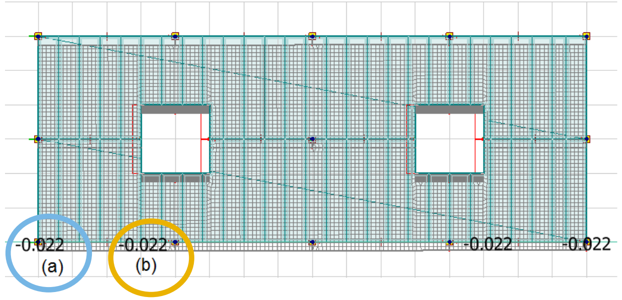


Rigid diaphragm model

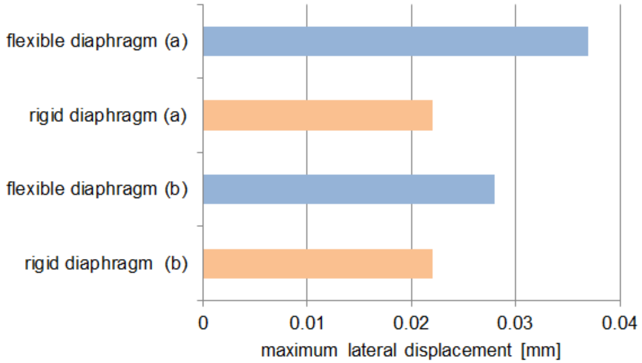
Lateral displacements



Flexible diaphragm model



Rigid diaphragm model



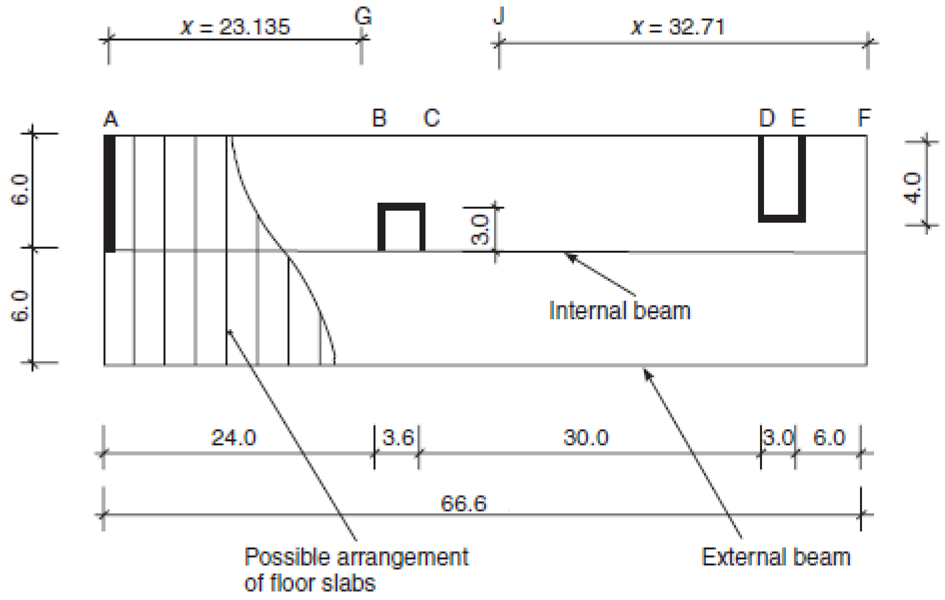
FEM global structural design

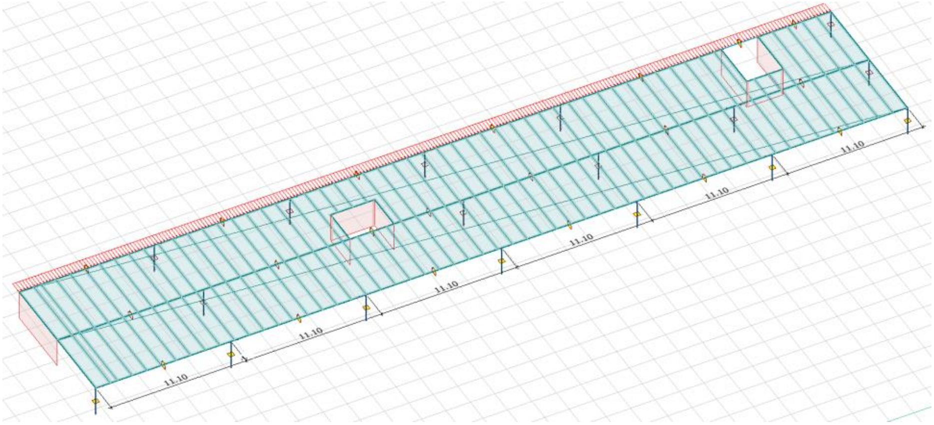
- Precast slabs – examples
  - Example 2: irregular structure

    - Structural system:
      - 1 story (3m high)
      - untopped two bay HD-F 120-20 slab
      - precast concrete beams
    (inverted T 600x600-350-400mm)
      - double hinged columns (400x400mm)
      - precast wall panels (200mm)

    - Load:
      - 4 kN/m linear distributed wind load)

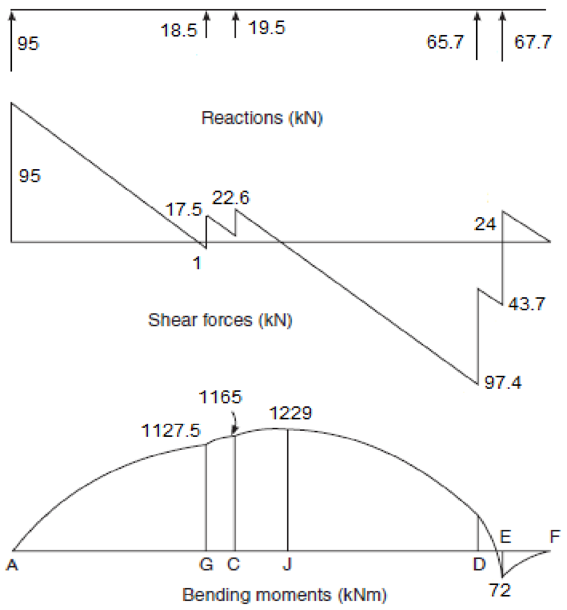
    - No vertical edge connectivity between the longitudinal and transversal wall panels!
    - Supports are "rigid"

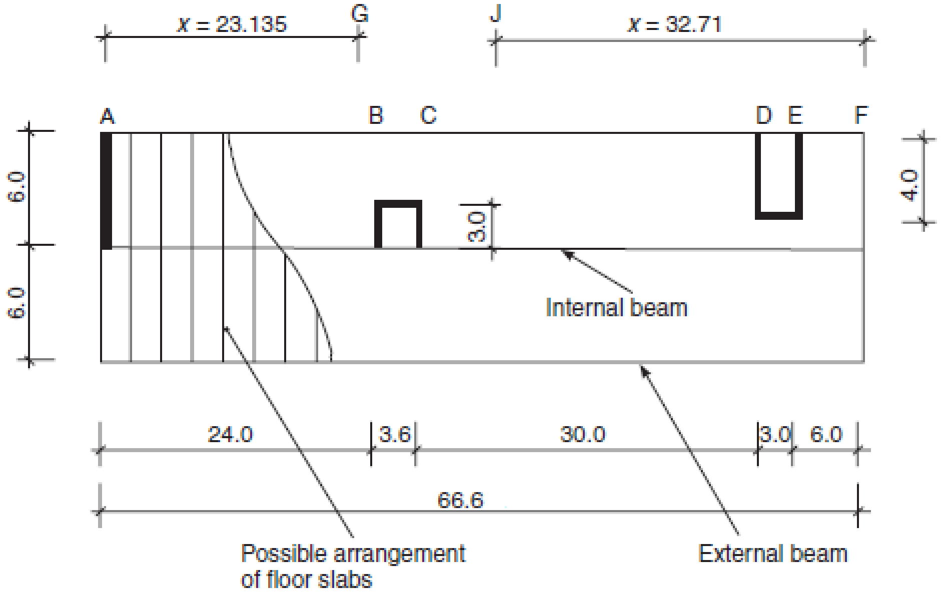




**The structural system is provided as a design example in the Second Edition of Precast Concrete Structures, by K.S. Elliott*

- Precast slabs – examples
  - Example 2: irregular structure
    A. Manual calculation based on rigid diaphragm hypothesis: distribution of horizontal loading

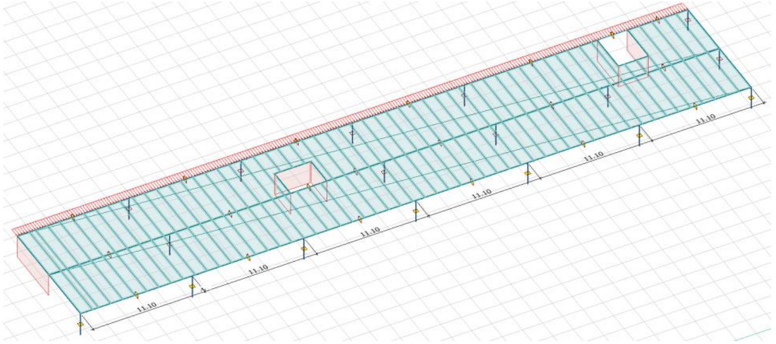


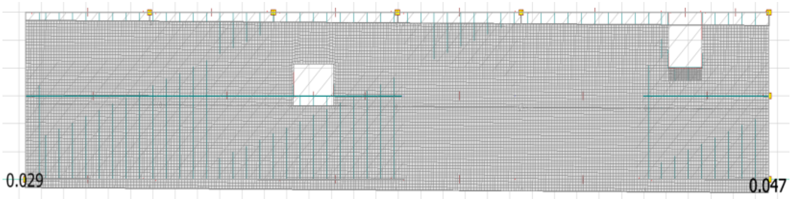


wall	L[m]	Ii [m^3]	xi [m]	Iixi	X	ai=xi-X [m]	$I_i a_i^2$	e[m]	Hi/H	H[kN]	Hi
A	66.6	216	0	0	22.51	22.51	109423.3	10.79	0.357	266.4	95.0
B		27	24	648		1.49	60.14098		0.069		18.5
C		27	27.6	745.2		5.09	700.1957		0.073		19.5
D		64	57.6	3686.4		35.09	78814.78		0.247		65.7
E		64	60.6	3878.4		38.09	92866.28		0.254		67.7
Total		398							1.000		

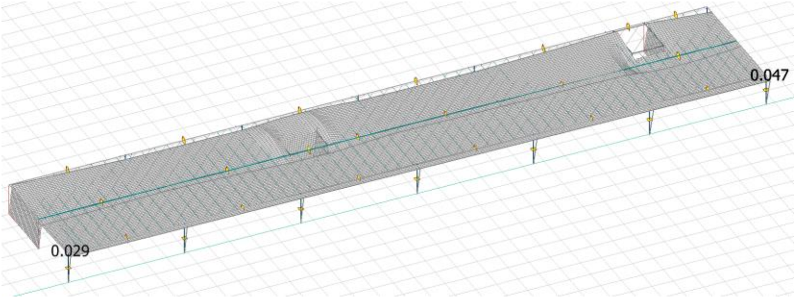
FEM global structural design

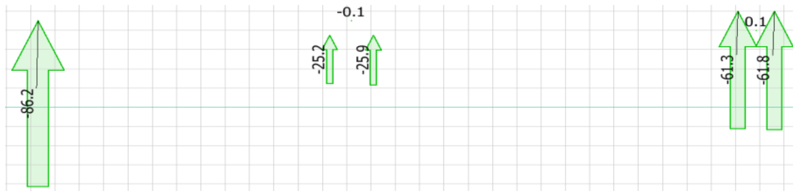
- Precast slabs – examples
  - Example 2: irregular structure
    - B. FEM-Design analysis based on rigid diaphragm hypothesis:
      - Distribution of horizontal loading
      - Lateral displacements
    - Objective: “validate” results of manual calculation
    - Structural output:
      - limited to displacements and reactions
      - no output with regards to internal forces in the slab and beams





Lateral displacement [mm]

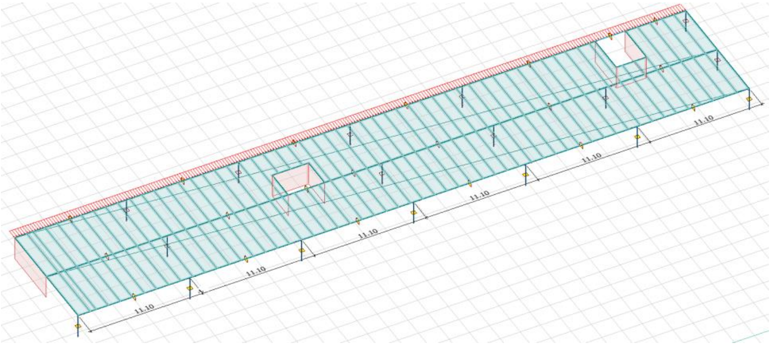


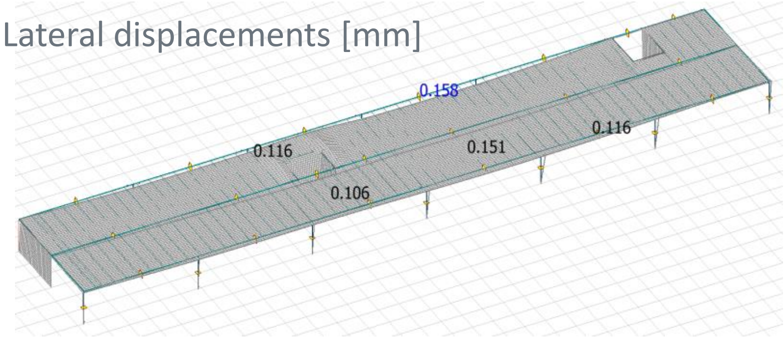


Lateral load distribution to walls [kN]

FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    C. FEM-Design analysis – flexible diaphragm hypothesis
      - Lateral displacements
      - Lateral force distribution
      - Internal force distribution in the HC slab





Lateral displacements [mm]

- *Flexible diaphragm* - mathematical model of the structure accounts for the stiffness of the slab

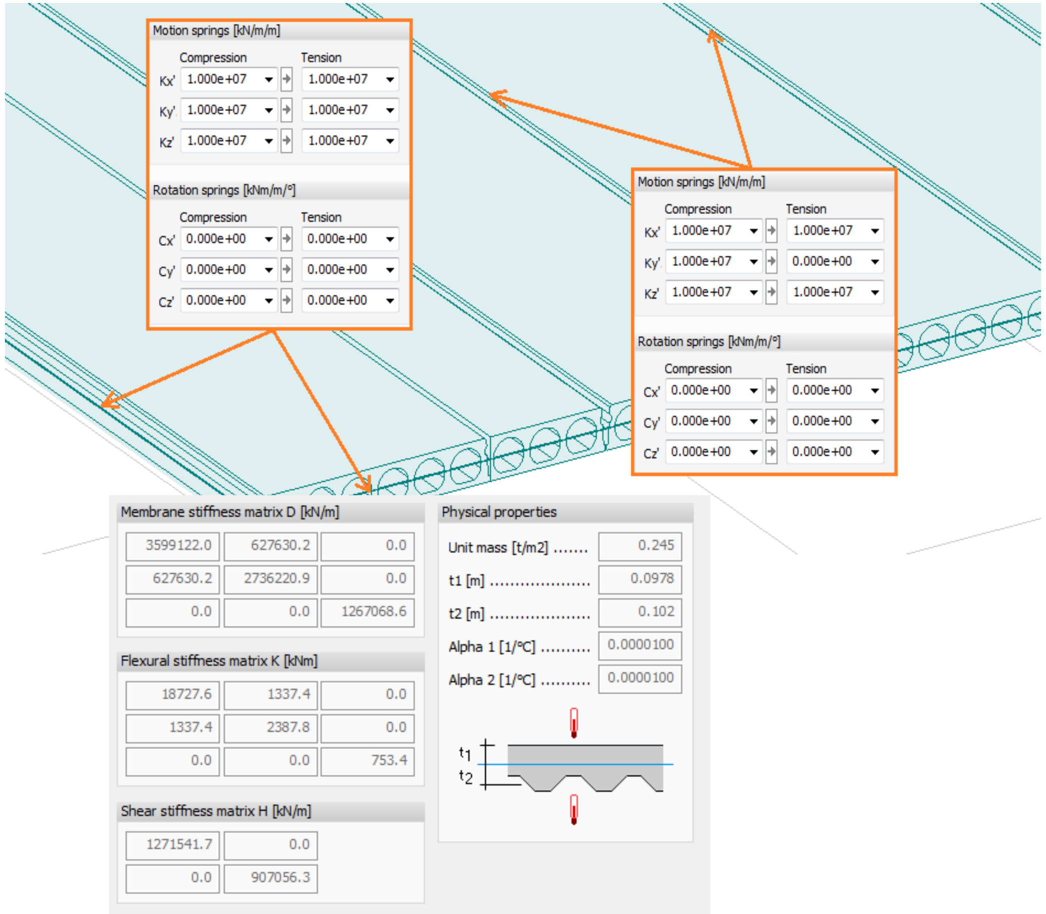
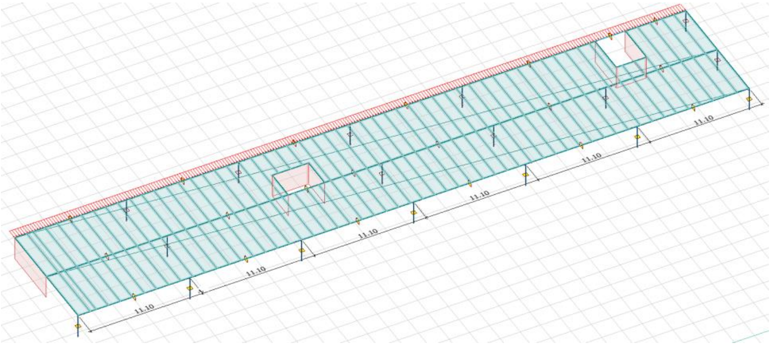
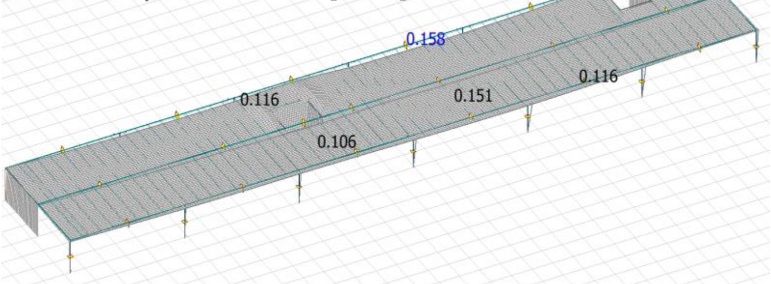


Plate stiffness matrix

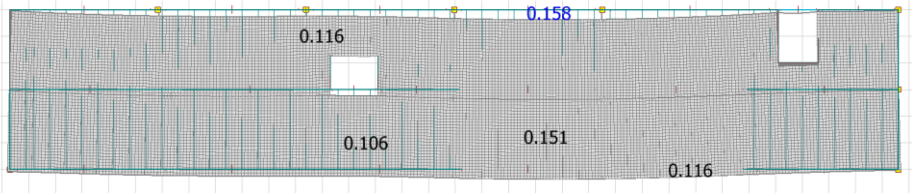
FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    C. FEM-Design analysis – flexible diaphragm hypothesis
      - Lateral displacements
      - Lateral force distribution
      - Internal force distribution in the HC slab

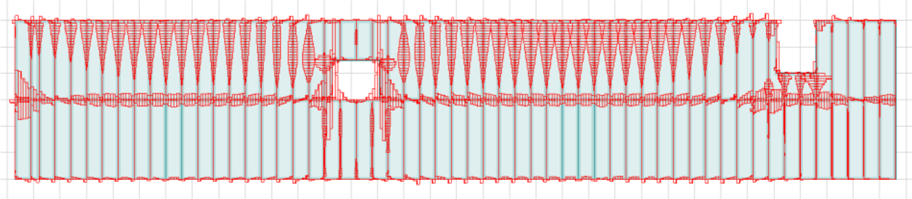




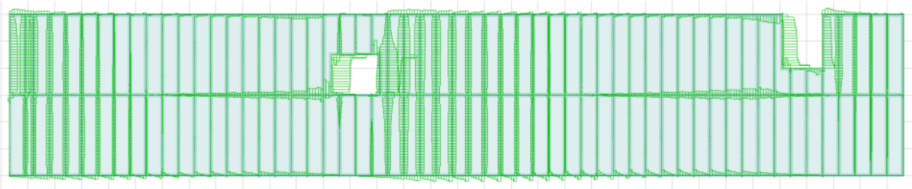
Lateral displacements [mm]



Lateral displacements [mm]



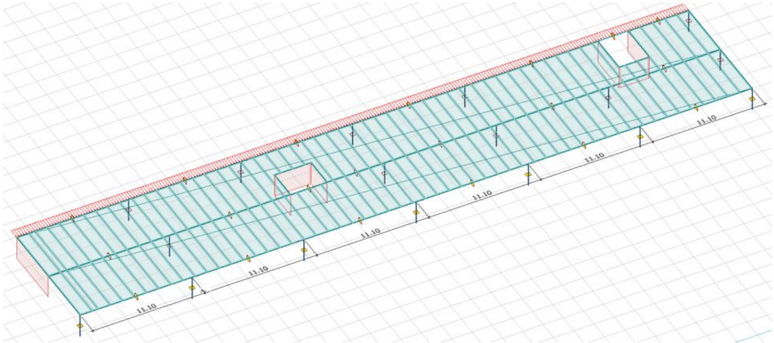
Compression in slab connections

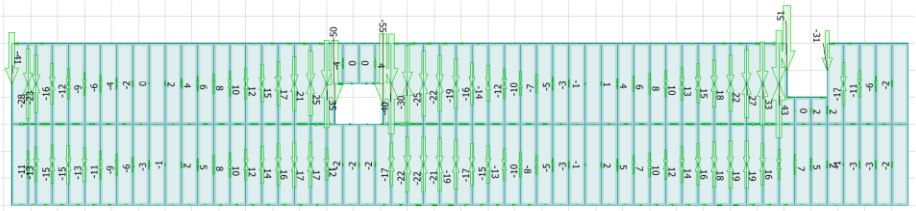


Shear in slab connections

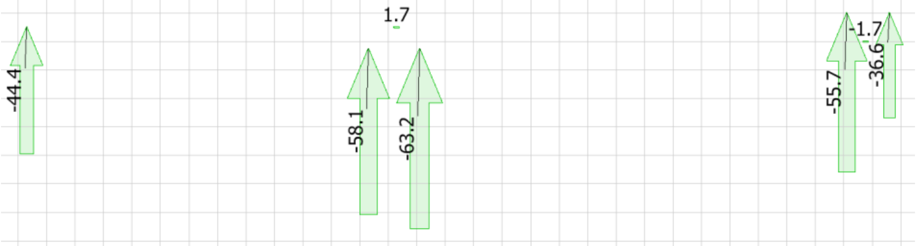
FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    - C. FEM-Design analysis – flexible diaphragm hypothesis
      - Lateral displacements
      - Lateral force distribution
      - Internal force distribution in the HC slab

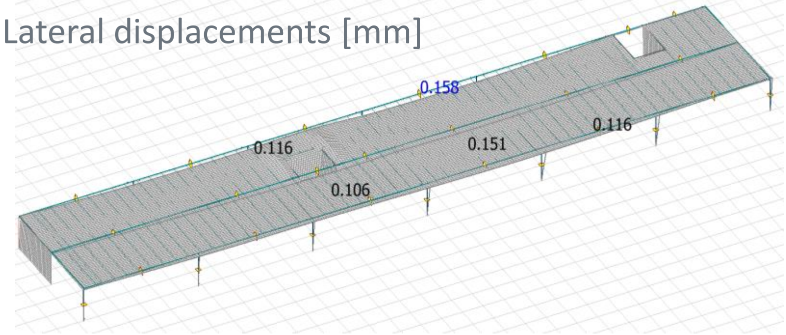




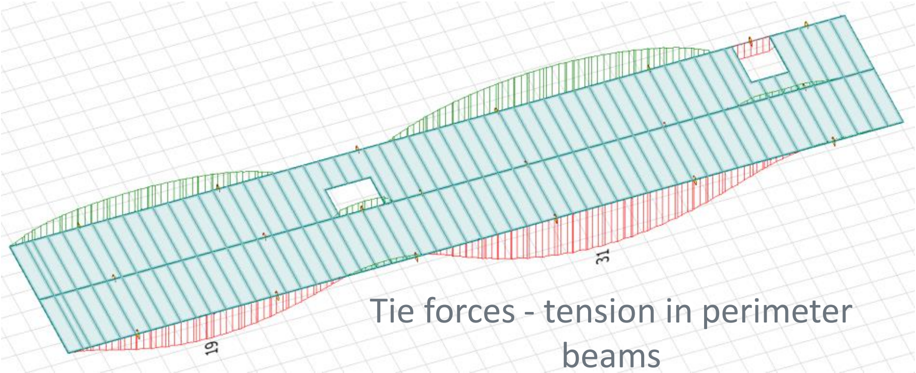
Shear resultants in HC connections



Lateral load distribution to walls [kN]



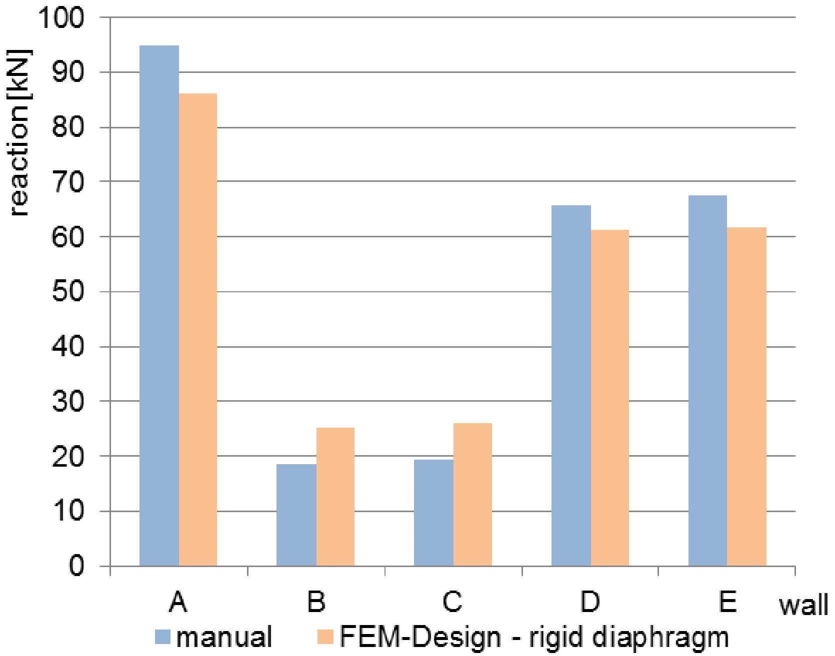
Lateral displacements [mm]



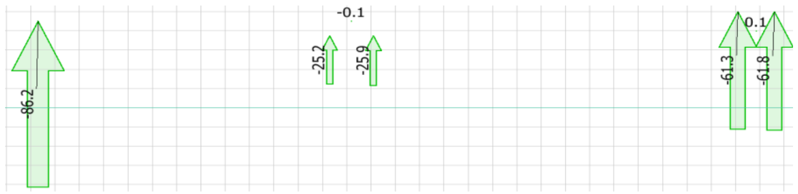
Tie forces - tension in perimeter beams

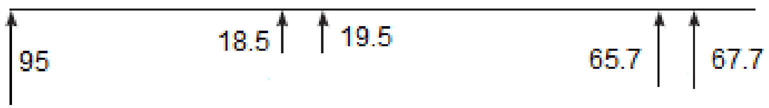
FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    - Structural output comparison
      - Lateral force distribution– rigid diaphragm model: manual vs. FEM-Design



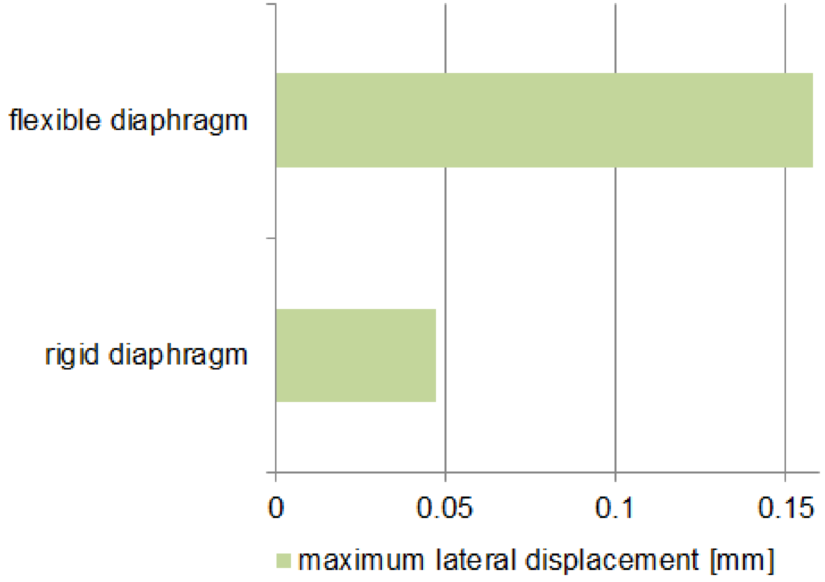
wall	manual	FEM	error*
A	95	86.2	-9
B	18.5	25.2	36
C	19.5	25.9	33
D	65.7	61.3	-7
E	67.7	61.8	-9

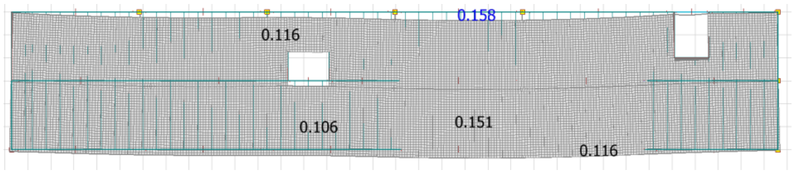




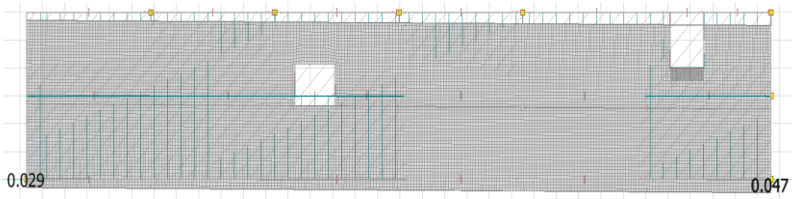
FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    - Structural output comparison
      - Maximum lateral displacement [mm] – rigid vs. flexible diaphragm model





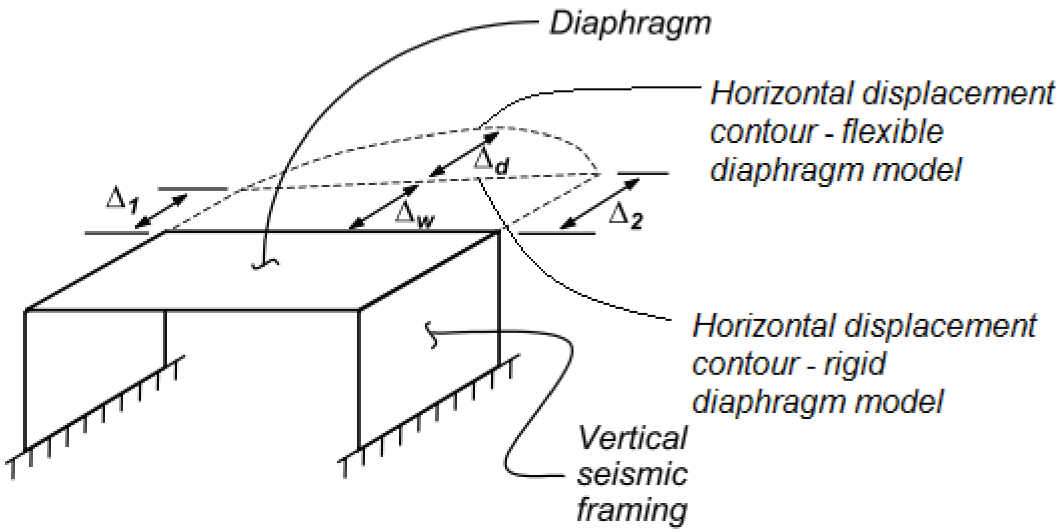
Flexible diaphragm model



Rigid diaphragm model

FEM global structural design

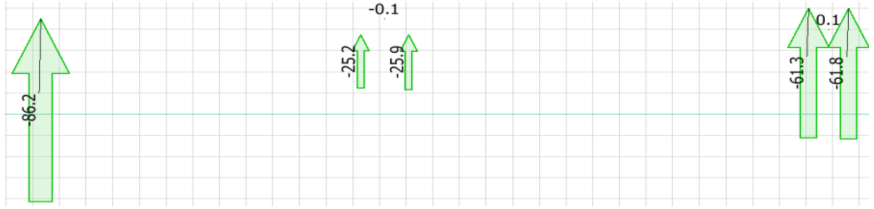
- Precast slabs – diaphragm models
  - Rigid or flexible diaphragm model?
    - Eurocode 2
      - No clear recommendations
      - Rigid diaphragm model seems to be universally suitable
    - Eurocode 8
      - Clear limitations for the rigid diaphragm model
      - Rigid diaphragm can be used when max 10% difference between horizontal displacements of the two models

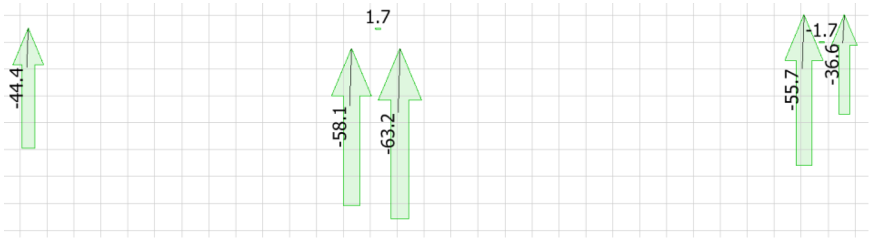


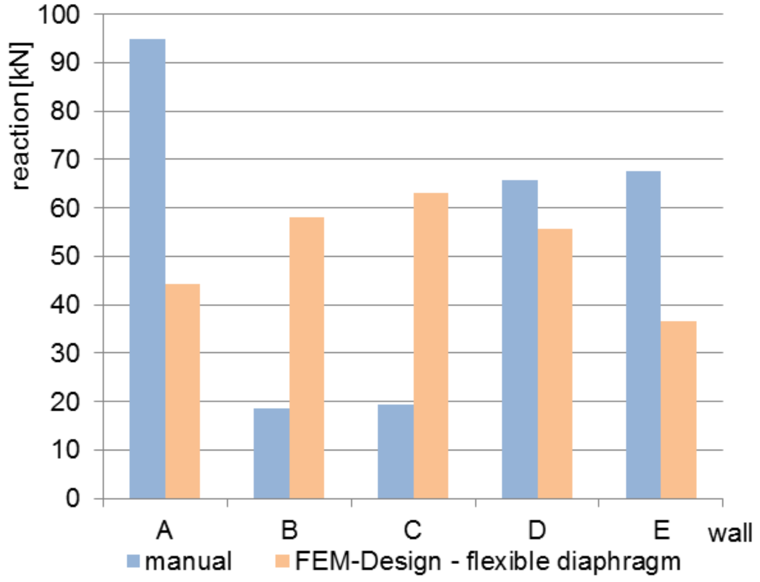
Comparison between rigid and flexible diaphragm maximum lateral displacement

FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    - Structural output comparison
      - Lateral force distribution - rigid vs. flexible diaphragm model



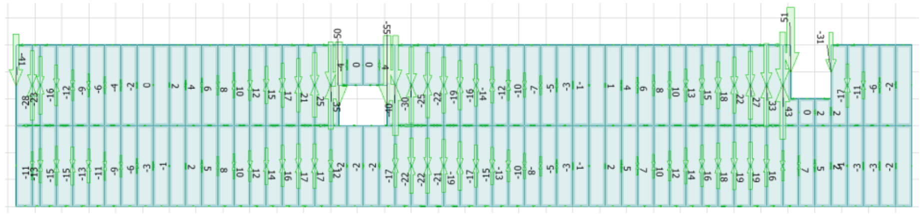


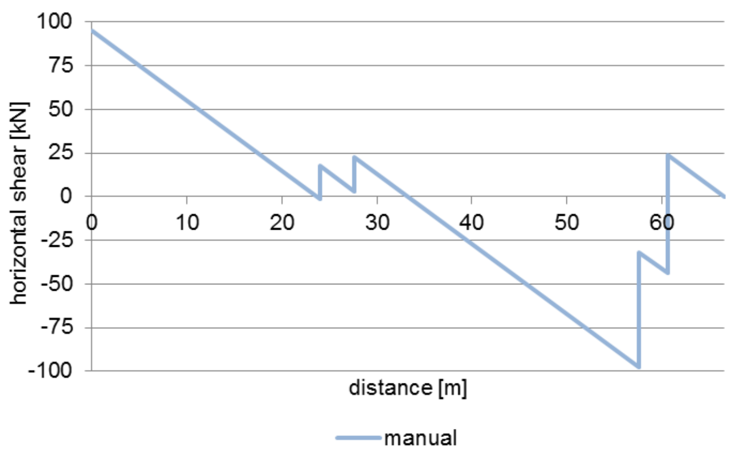


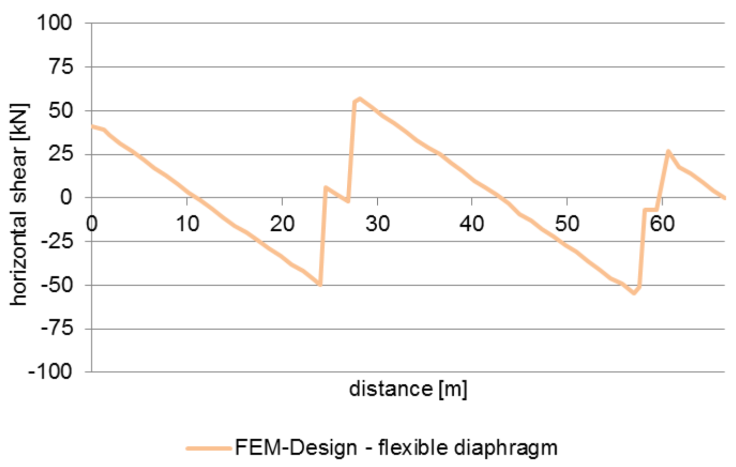
wall	rigid	flexible	error*
A	95	44.4	-53
B	18.5	58.1	214
C	19.5	63.2	224
D	65.7	55.7	-15
E	67.7	36.6	-46

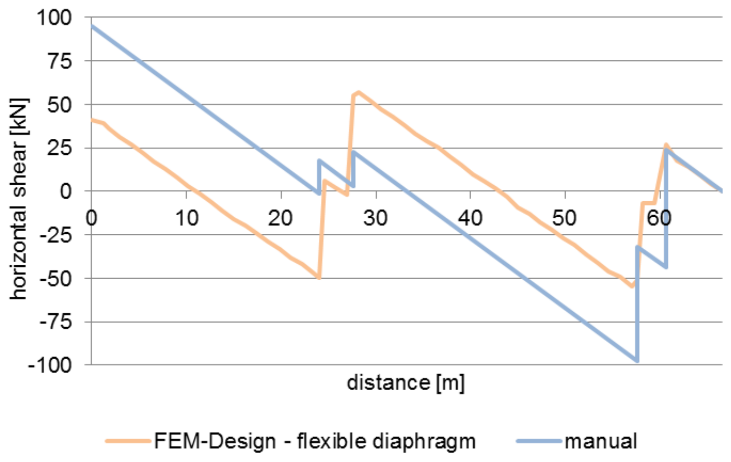
FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    - Structural output comparison:
      - Shear in the HC slab – rigid vs. flexible diaphragm model



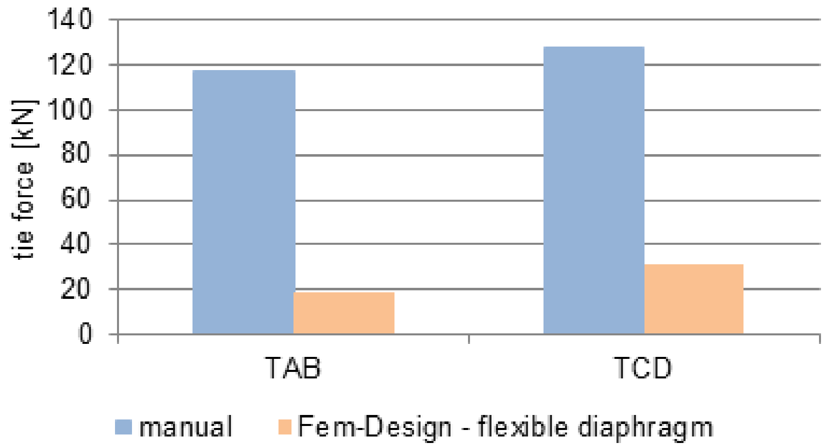


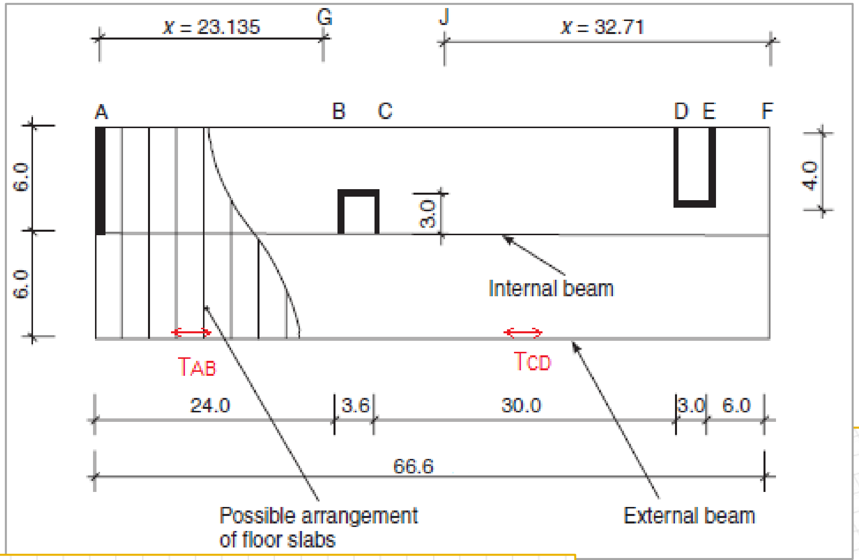


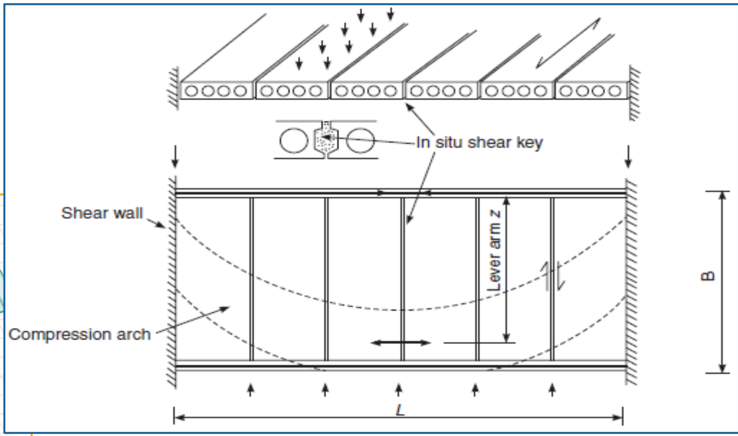


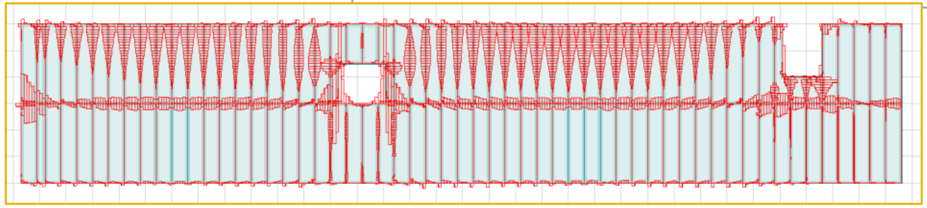
FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    - Structural output comparison:
      - Tie forces – rigid vs. flexible diaphragm model

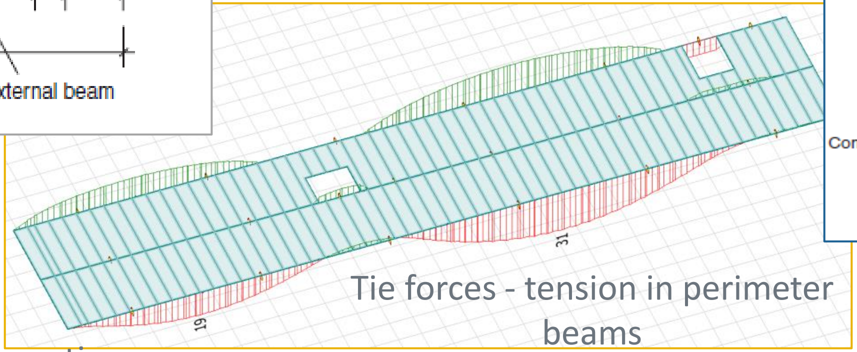








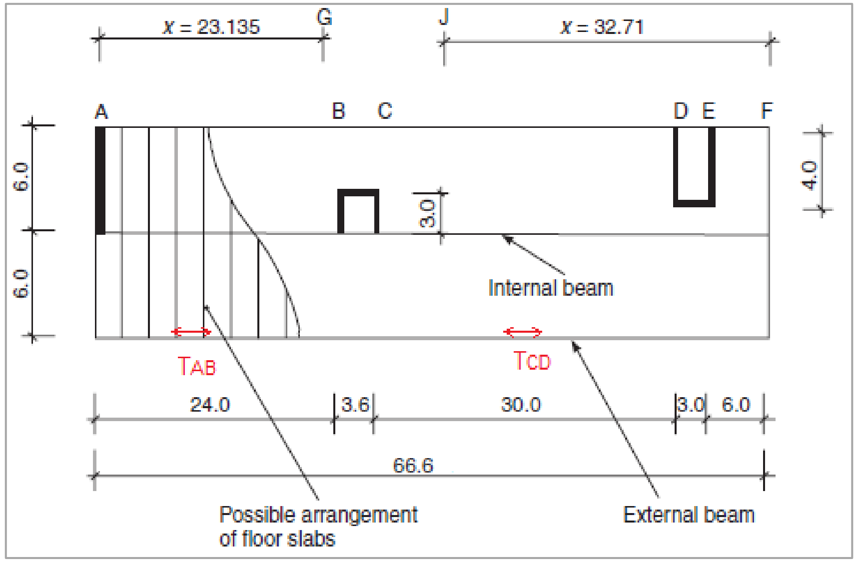
Compression forces distribution in compression only HC slab connections

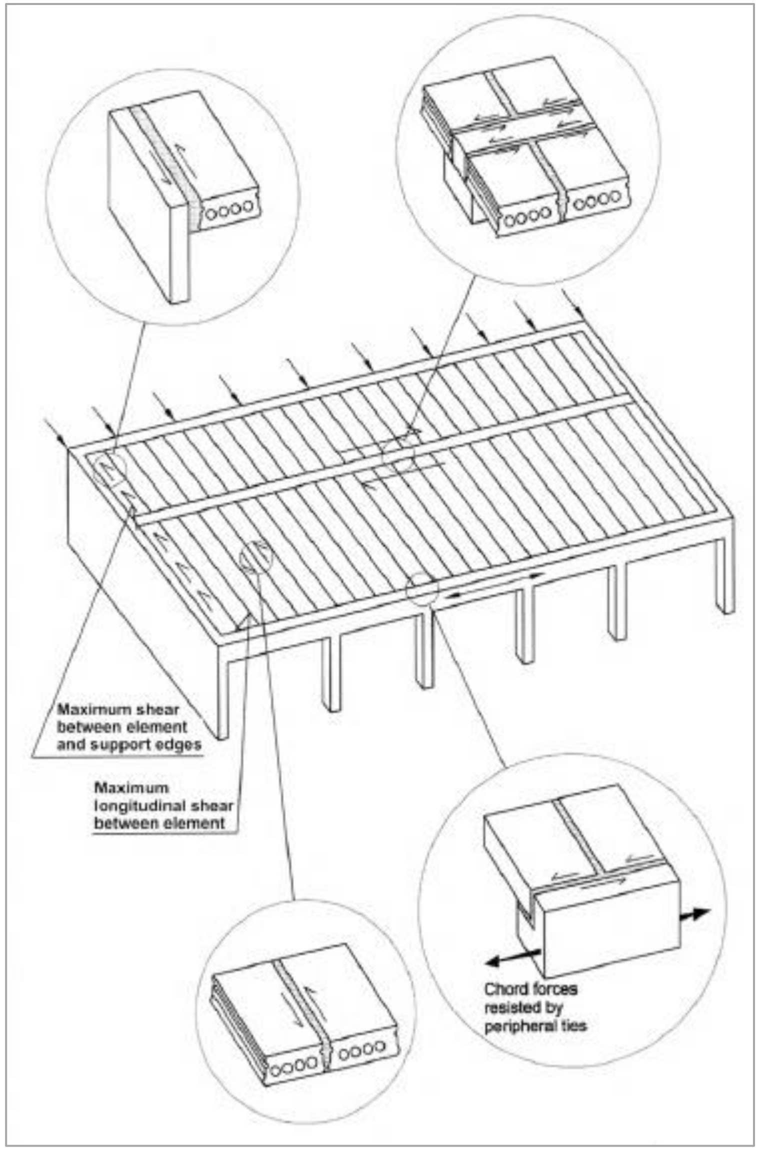


Tie forces - tension in perimeter beams

FEM global structural design

- Precast slabs – examples
  - Example 2: irregular structure
    - Structural output comparison:
      - Tie forces – rigid vs. flexible diaphragm model

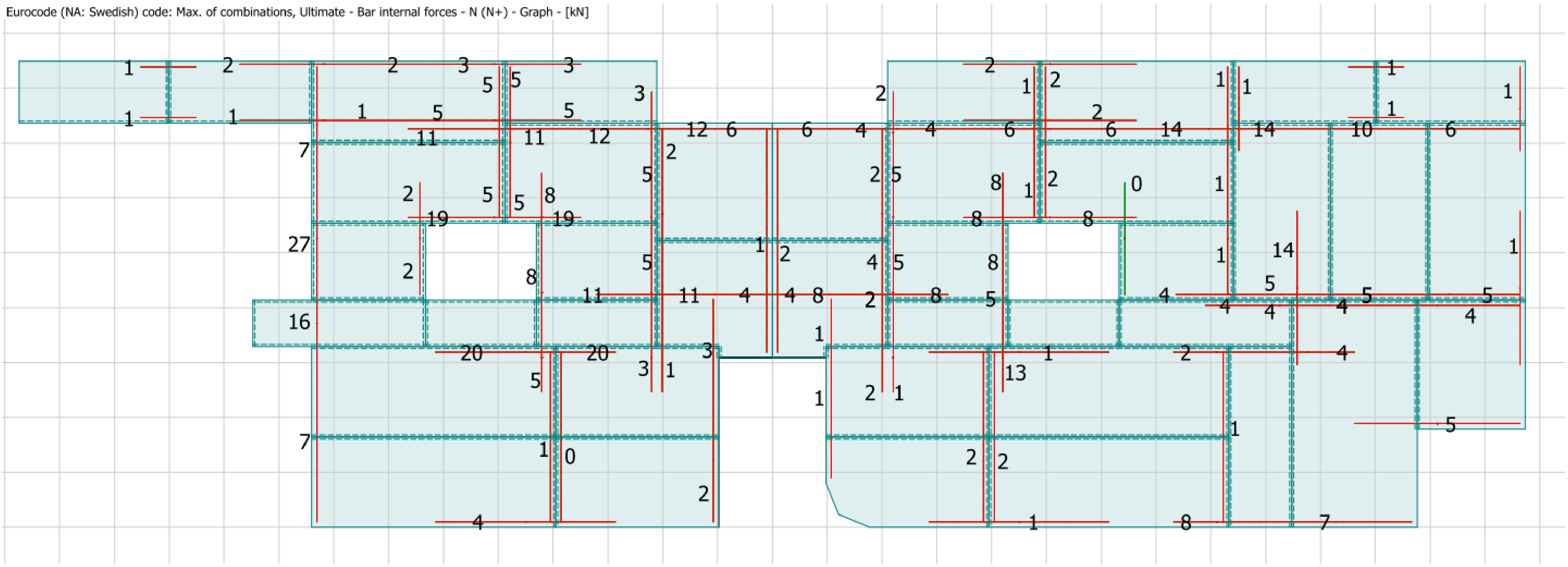




Horizontal force transfer (Structural Precast Concrete Handbook 2nd Edition)

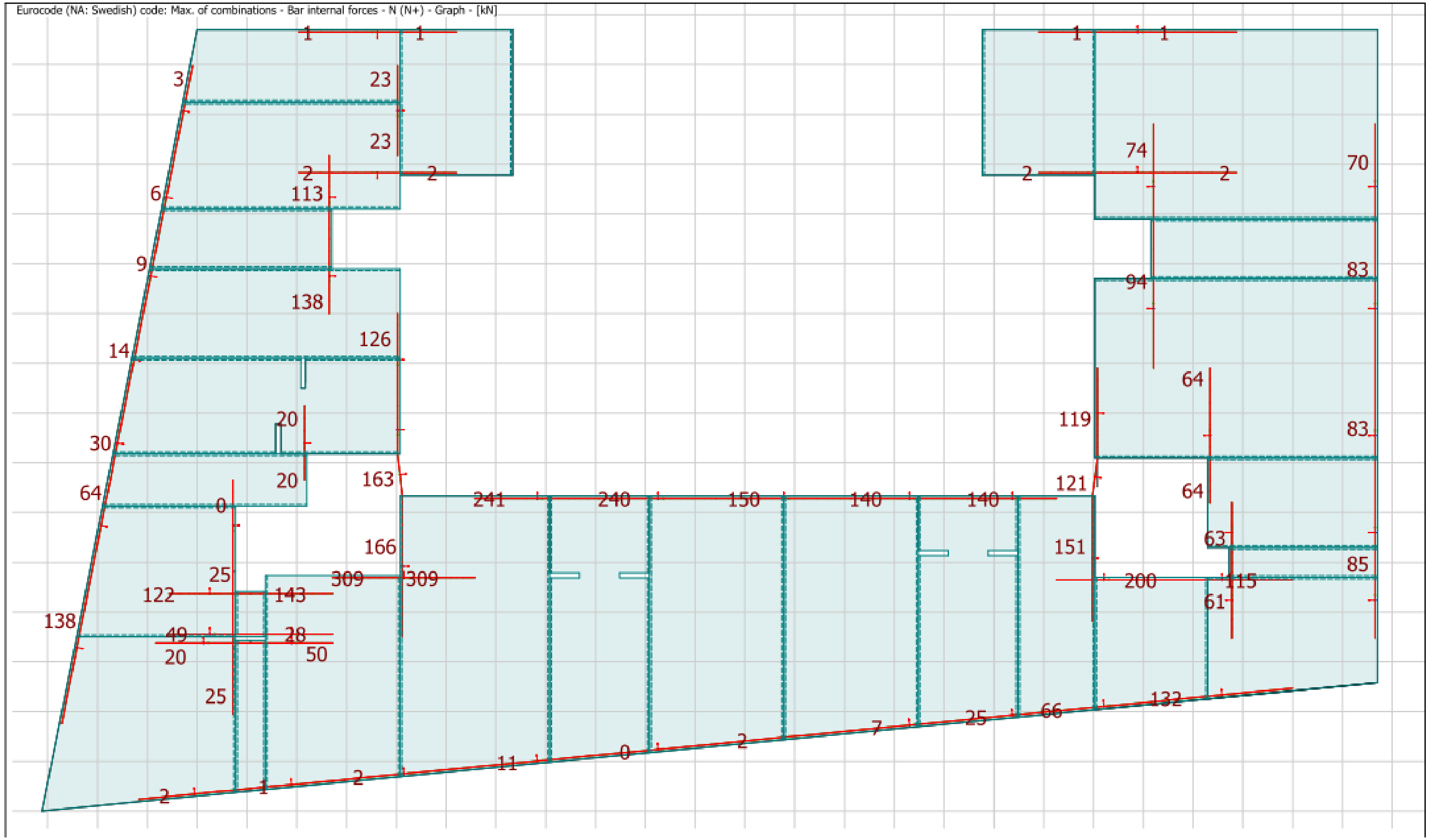
FEM global structural design

- Precast slabs – horizontal tie design example projects

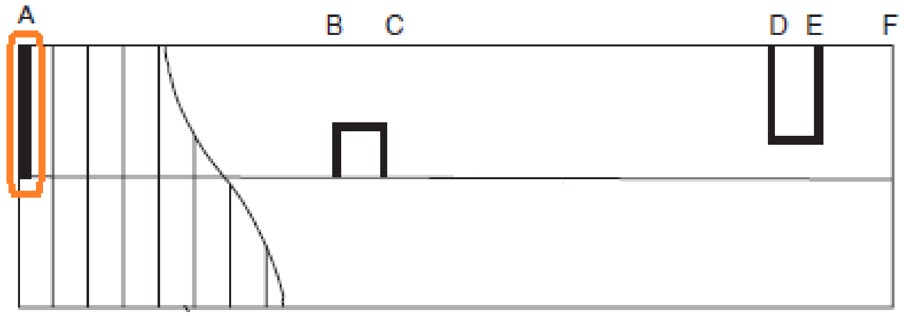


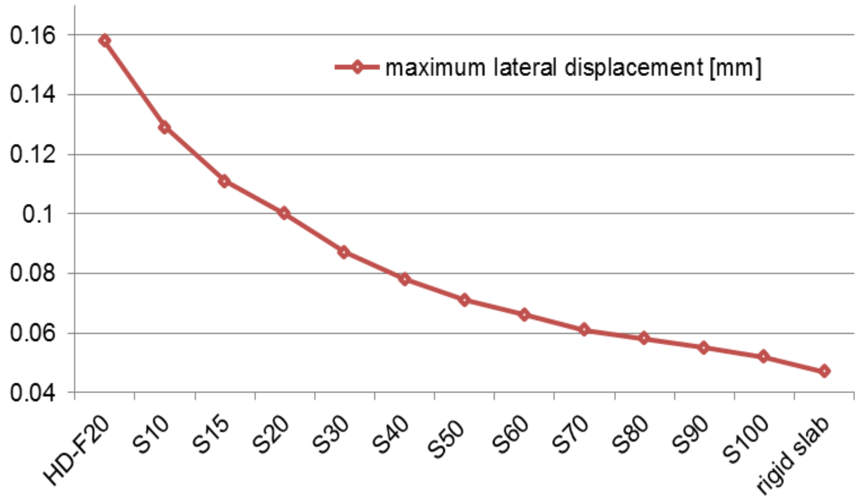
FEM global structural design

- Precast slabs – horizontal tie design example projects

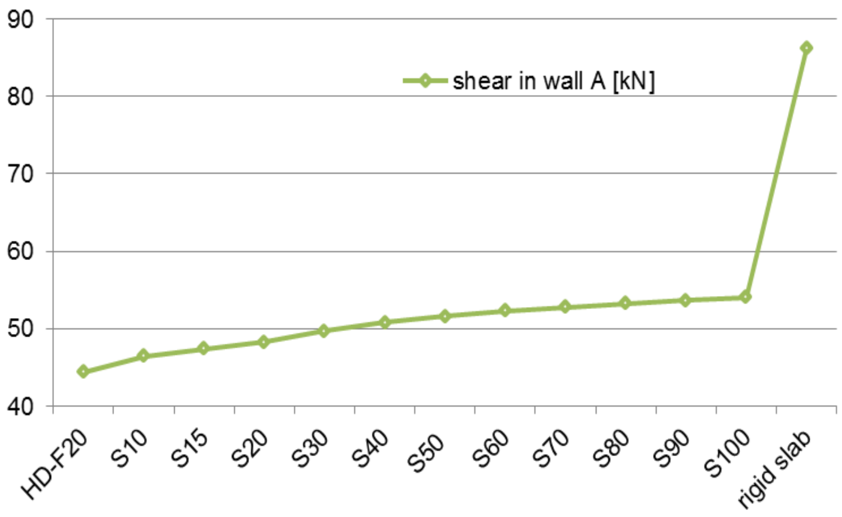


- Precast slabs – examples
  - Example 2: irregular structure
    - Structural output comparison:
      - Structural response variation because of slab stiffness variation





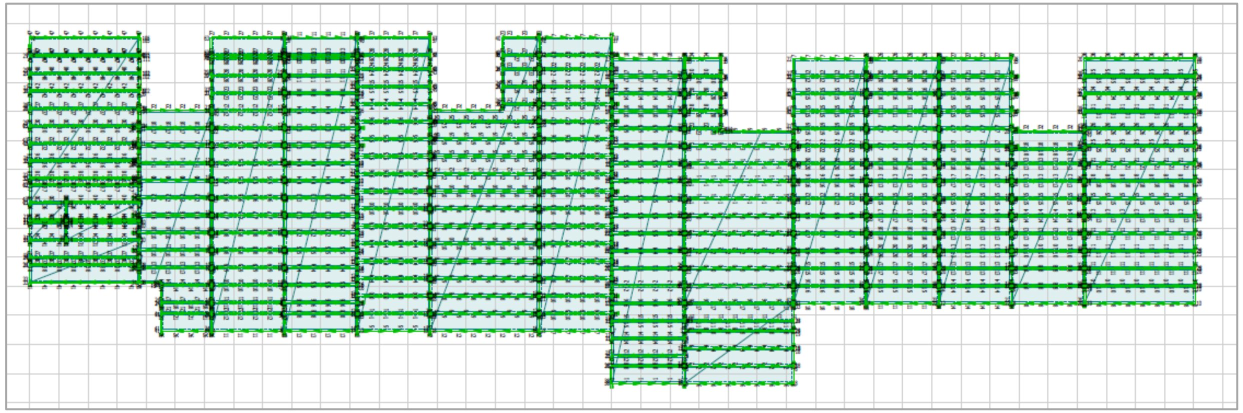
Variation of maximum lateral displacement with the stiffness of the slab



Variation of shear force in wall A with the stiffness of the slab

FEM global structural design

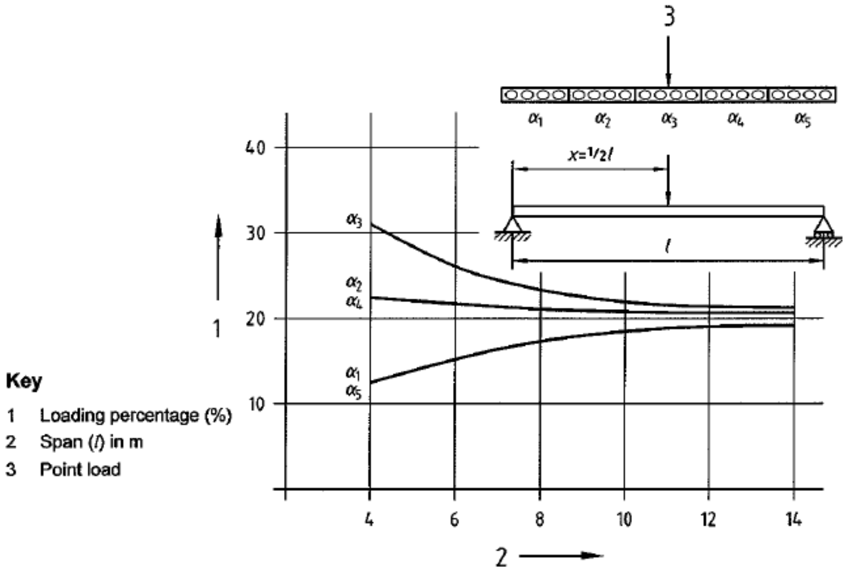
- Precast slabs – design of horizontal diaphragm wrap-up
  - Rigid diaphragm model:
    - Applicable in “manual” and FE models
    - Result can be verified by manual calculations
    - FE models don’t provide structural response for the design of the horizontal diaphragm
  - Flexible diaphragm model:
    - Applicable only with FEM tools
    - Results cannot be verified by manual calculations
    - FE models provide structural response (internal forces) for the design of horizontal diaphragms :
      - Slab to wall/beam connections
      - HC longitudinal connections (e.g. do we need structural toping for the HC slab?)
      - Horizontal ties

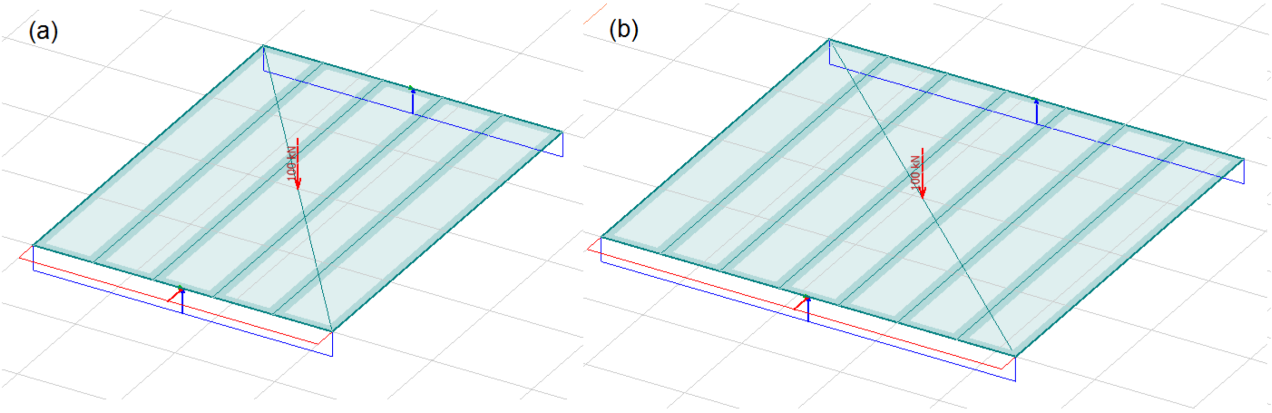


Plastic shear distribution in HC connections

FEM global structural design

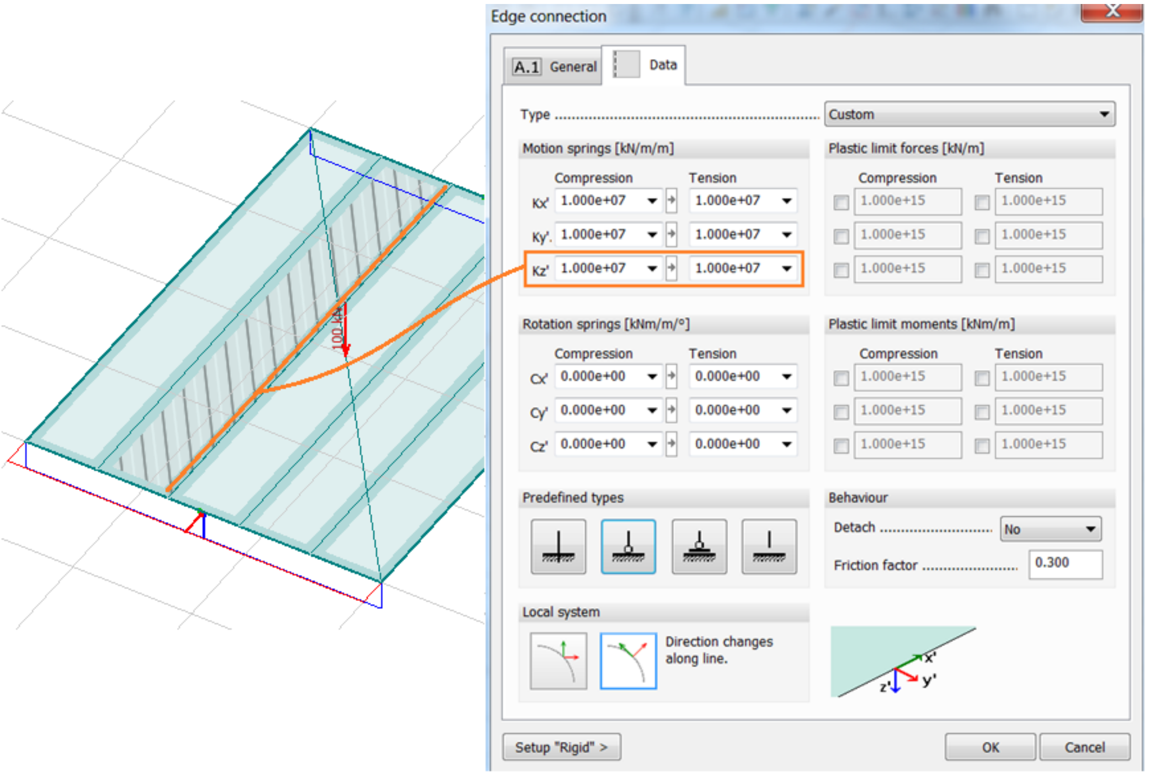
- Precast slabs – transversal load distribution
  - Uniform distribution of gravity loads - **OK** in FE
  - Concentrated loads distribution acc. to EN1168 Annex C – **cannot be emulated** in linear elastic FE

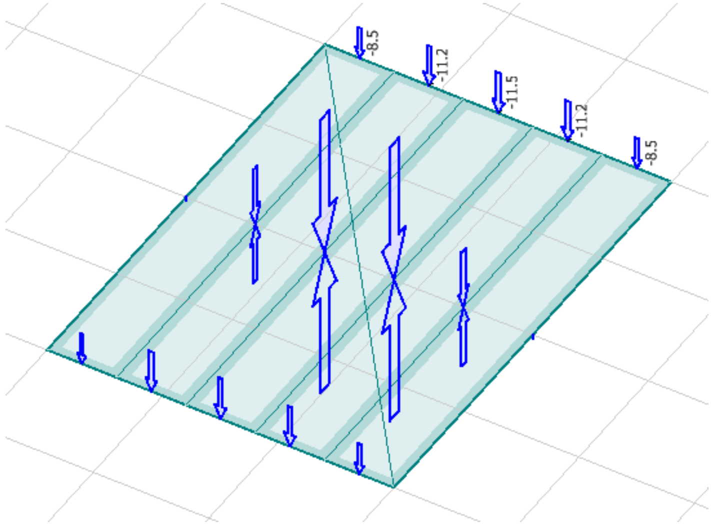




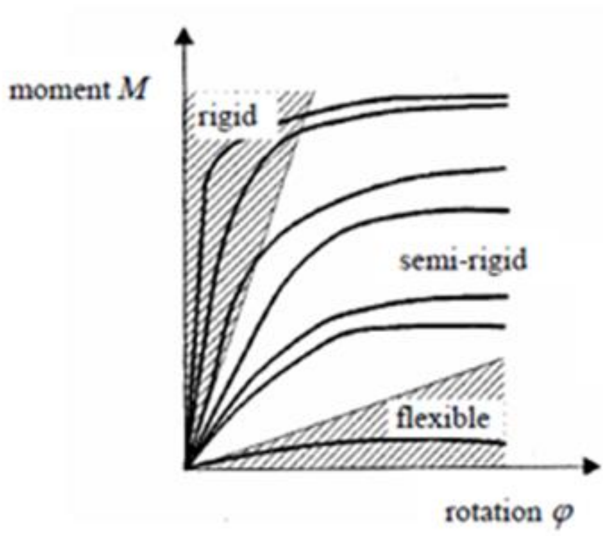
FEM global structural design

- Precast slabs – transversal load distribution
  - Uniform distribution of gravity loads - **OK** in FE
  - Concentrated loads distribution acc. to EN1168 Annex C – **cannot be emulated** in linear elastic FE

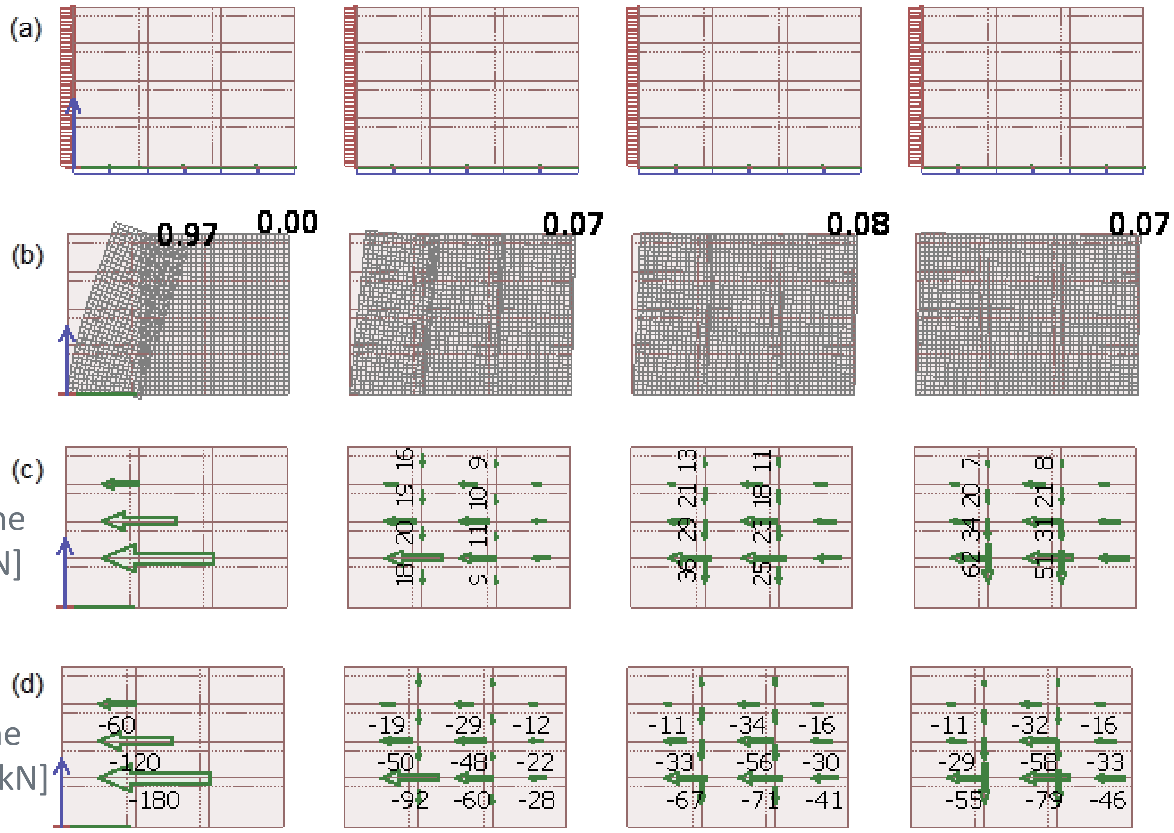




- FEM – Precast connections
  - Structural response variation of precast wall assemblies with edge connection stiffness change



Classification of moment-resisting connections – (Leon, 1998)



(b) top lateral displacement [mm]

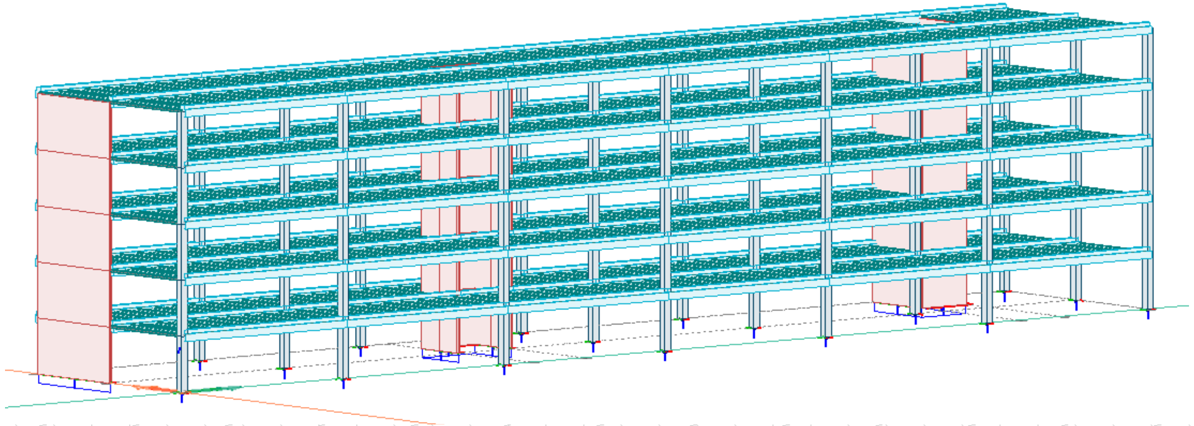
(c) shear resultant in the vertical connections [kN]

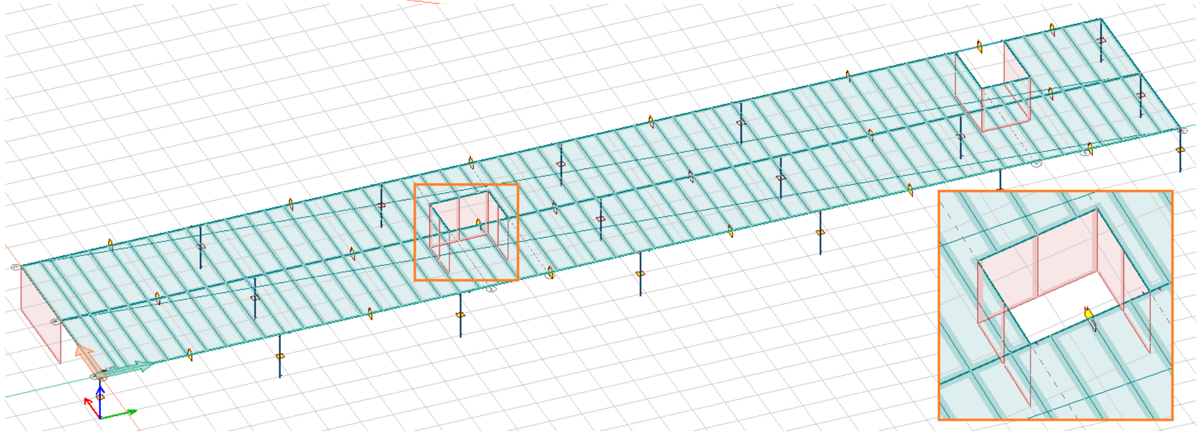
(d) shear resultant in the horizontal connection [kN]

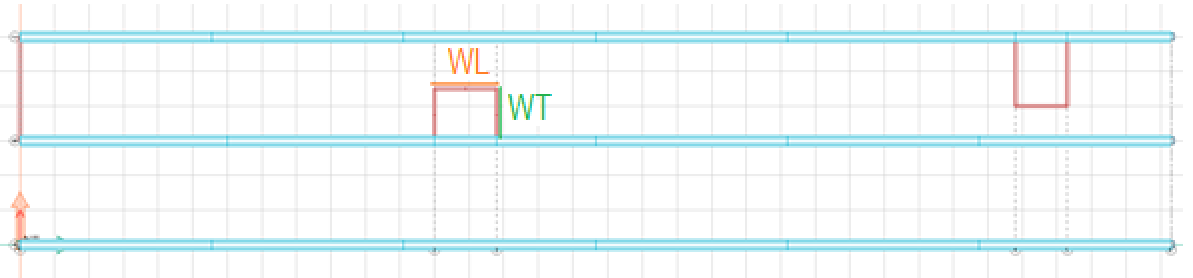
(a) The models consist of (left to right):free edge, 1% of “theoretical” hinge translational stiffness (0.01x1.00e+07), 5% of “theoretical” hinge translational stiffness (0.05x1.00e+07) and “theoretical” hinge

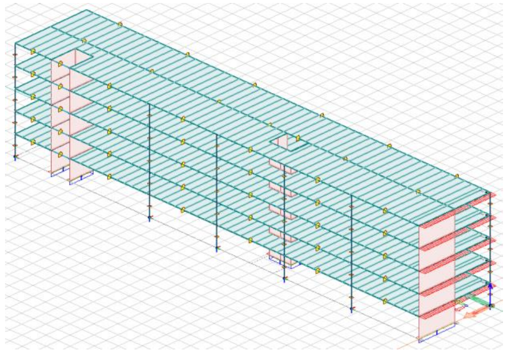
FEM global structural design

- Precast walls – examples
  - Impact of connection stiffness on structural response
  - Example 3:
    - Example 2 general layout with 5 story
    - Flexible diaphragms
    - Two load case: wind X (WX) & wind Y (WY)

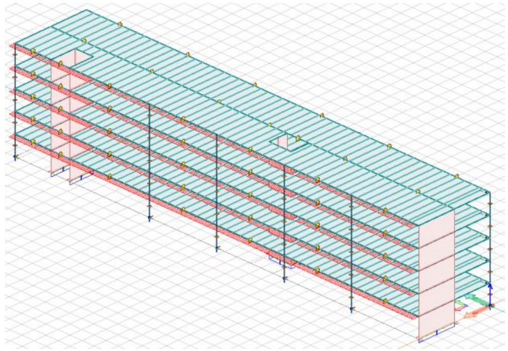








WX: 5 kN/m linear distributed wind load

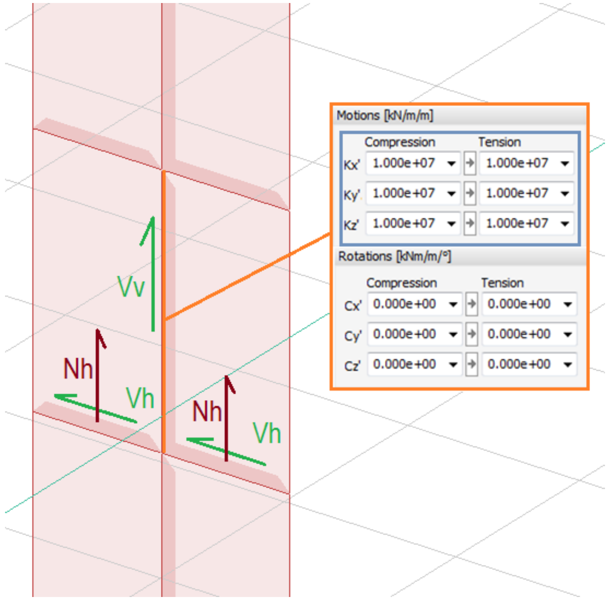


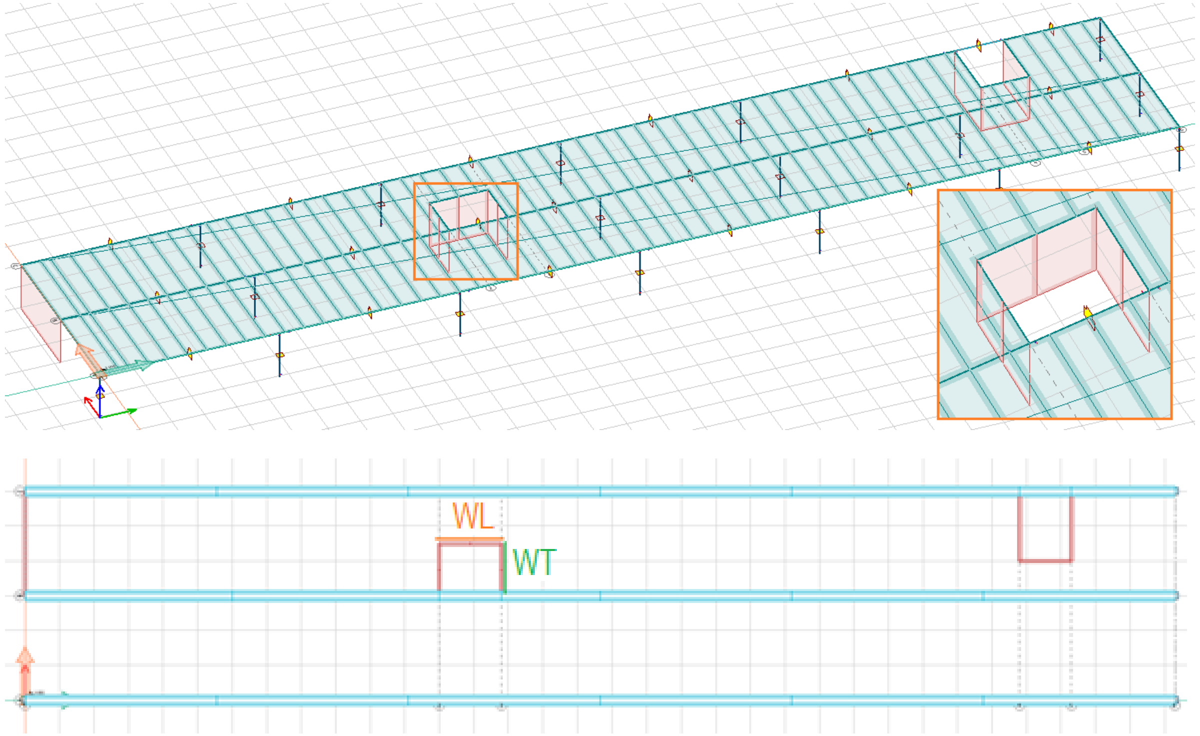
WY: 4 kN/m linear distributed wind load

FEM global structural design

- Precast walls – Example 3
  - Design approaches for wall panel vertical connections stiffness:
    a. free edge
    b. 0.50% of "rigid value"
    c. 5.00% of "rigid value"
    d. "rigid" value
  - Structural output :
    - shear in the vertical connections (Vv)
    - shear in the horizontal connections (Vh)
    - axial force in the horizontal connections (Nh)

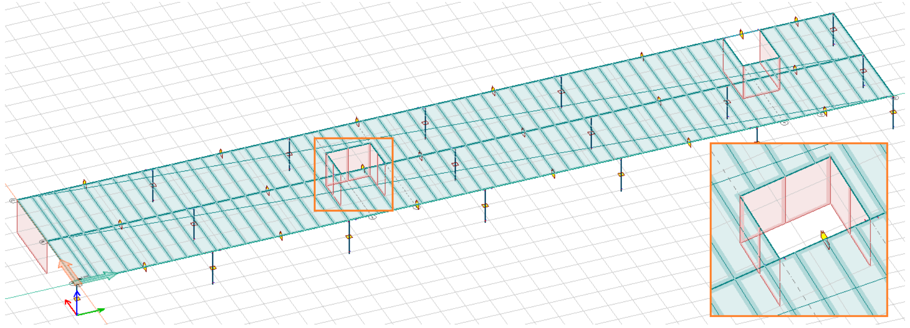
"rigid" values for the wall panel vertical connection

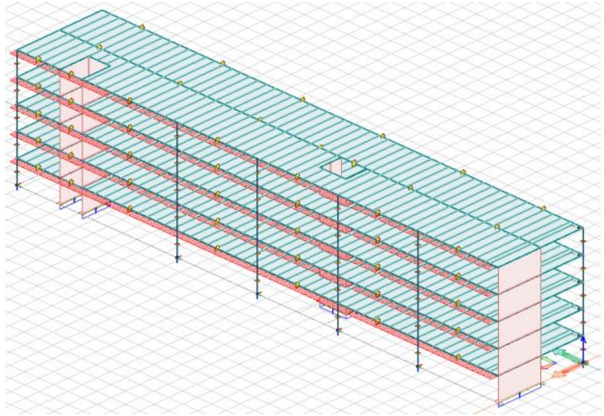


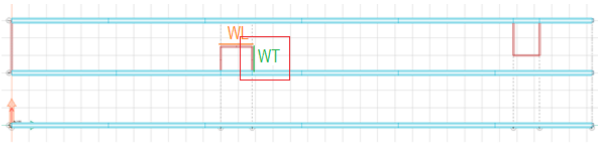


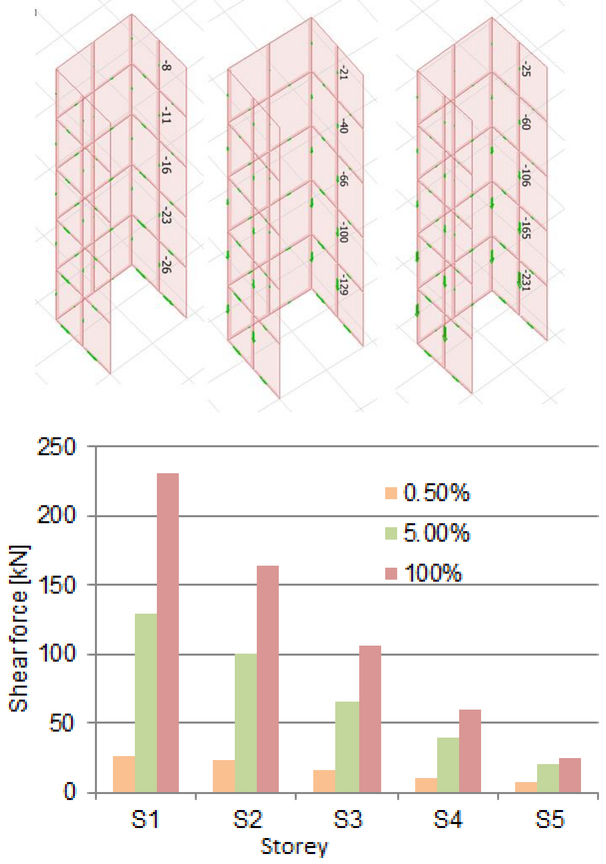
FEM global structural design

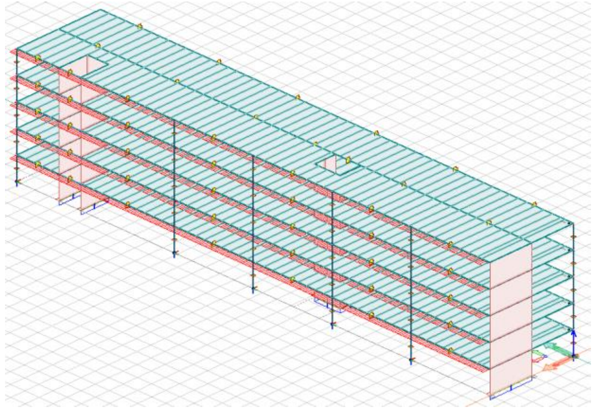
- Precast walls – Example 3
  - Structural output:
    - WY load case
    - WT walls
    - shear in the vertical connections (Vv)



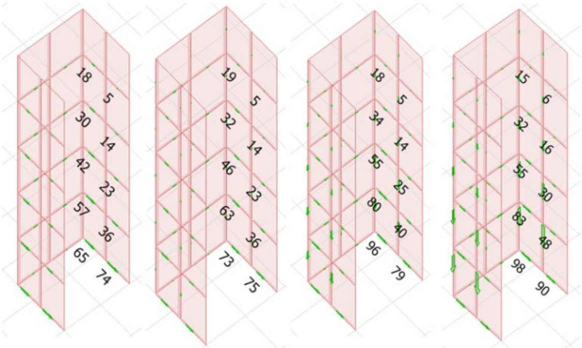


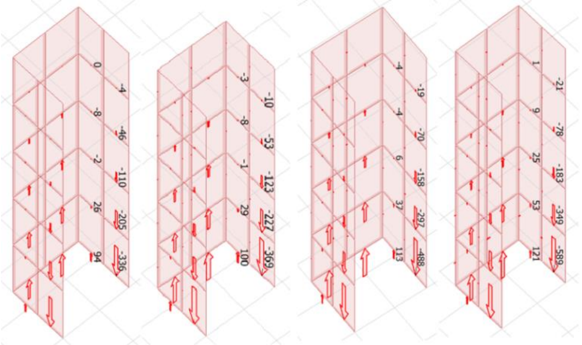


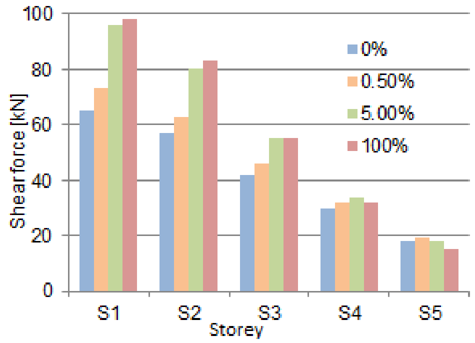


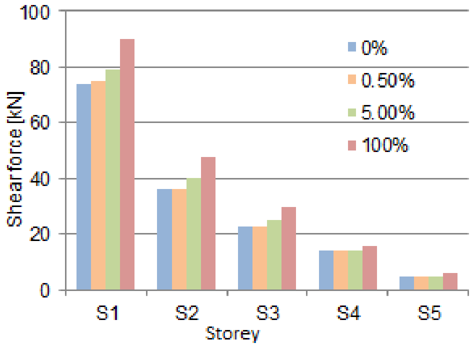


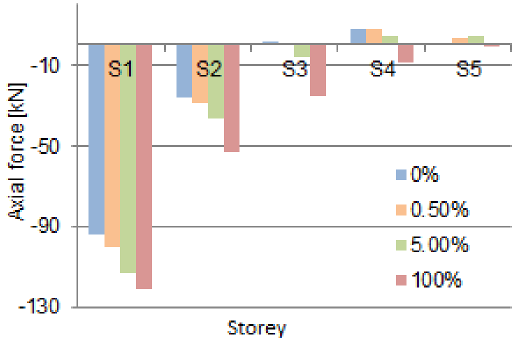
- Precast walls – Example 3
  - Structural output:
    - WY load case
    - WT walls
    - shear in the horizontal connections (Vh)
    - axial force in the horizontal connections (Nh)

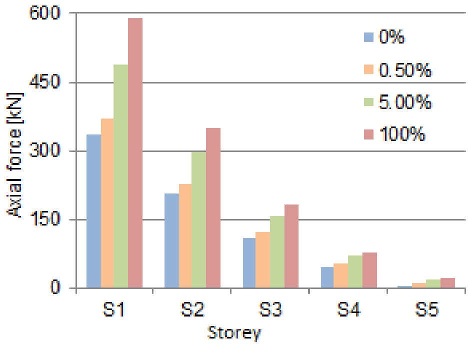


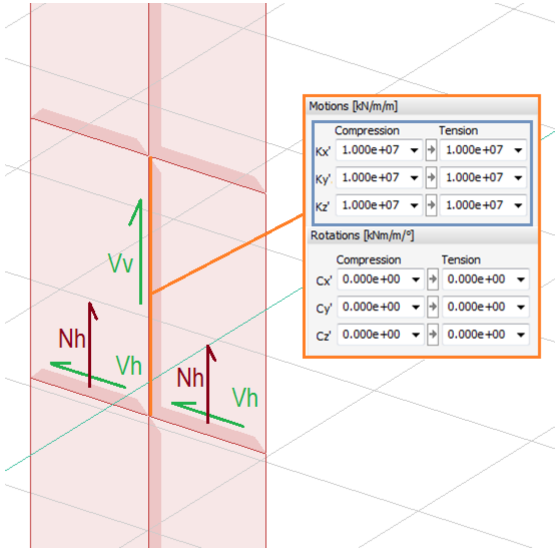






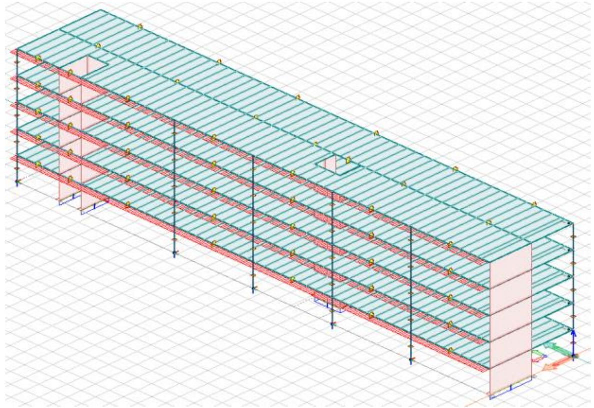


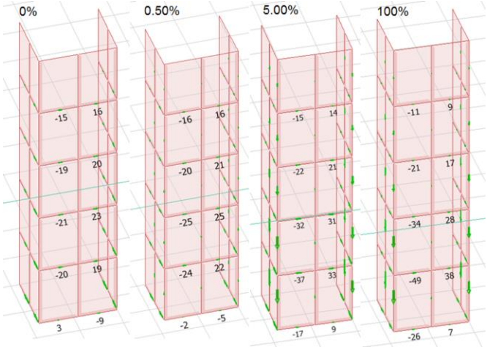


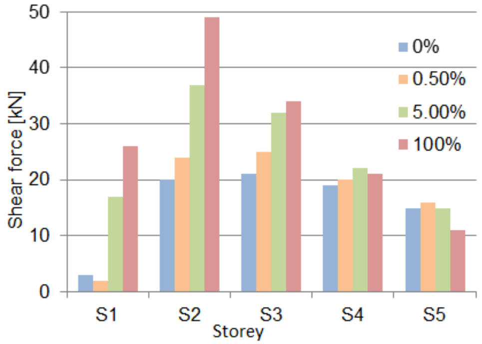


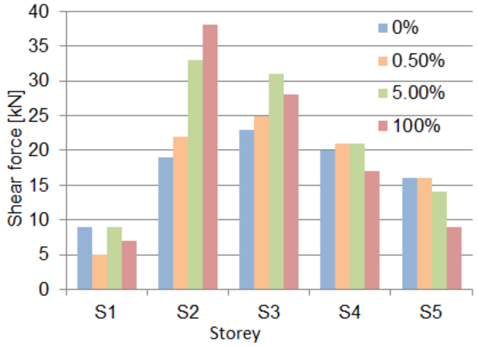
FEM global structural design

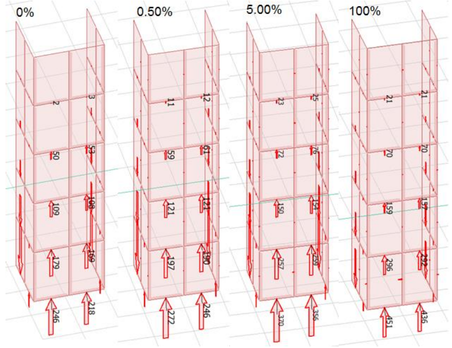
- Precast walls – Example 3
  - Structural output:
    - WY load case
    - WL walls
    - shear in the horizontal connections (Vh)
    - axial force in the horizontal connections (Nh)

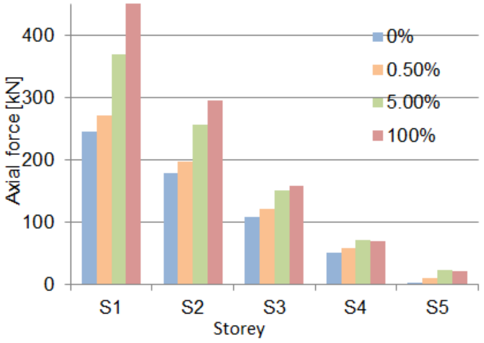


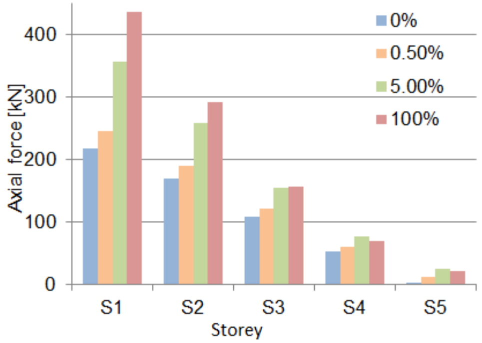






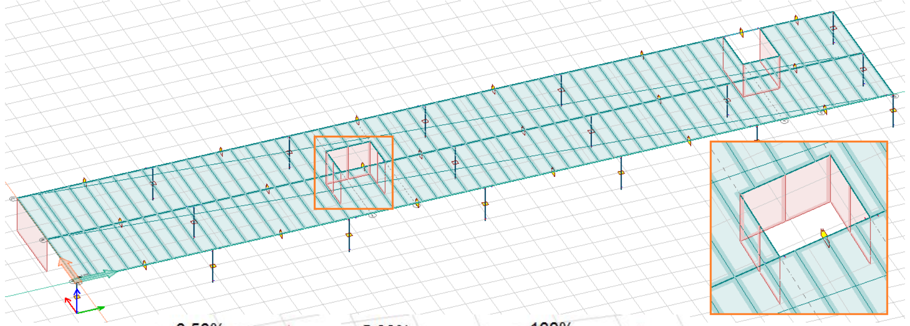


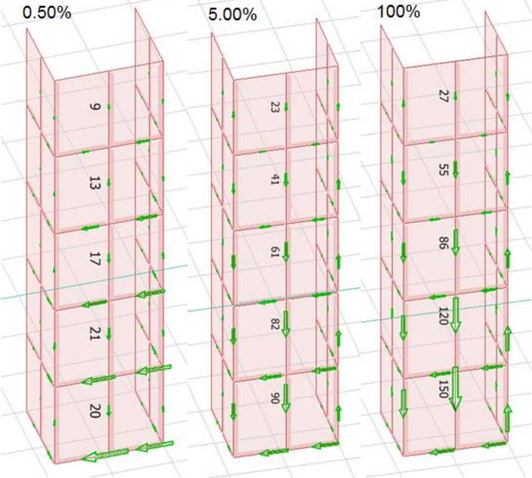


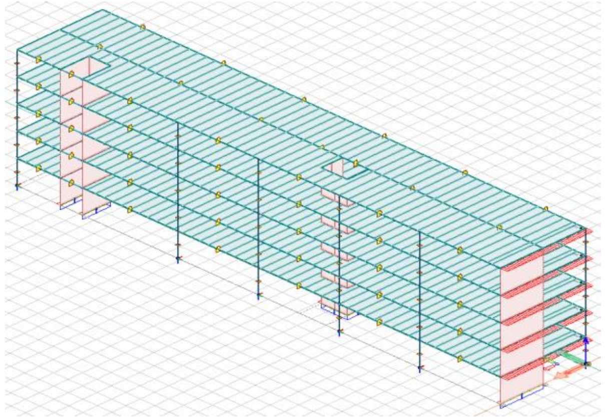


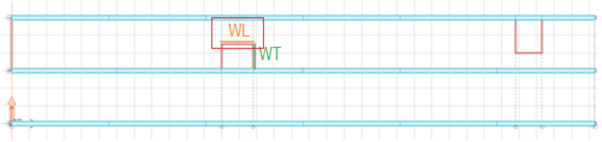
FEM global structural design

- Precast walls – Example 3
  - Structural output:
    - WX load case
    - WL walls
    - shear in the vertical connections (Vv)



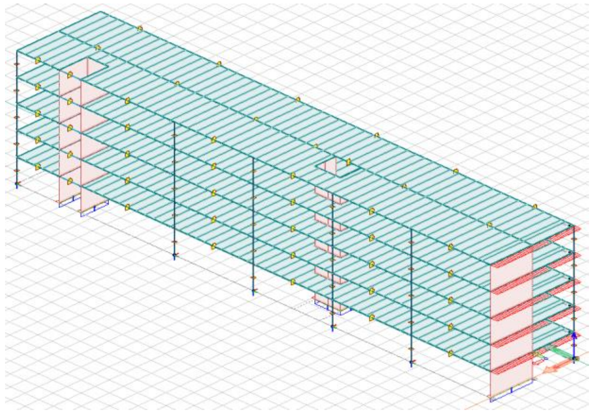


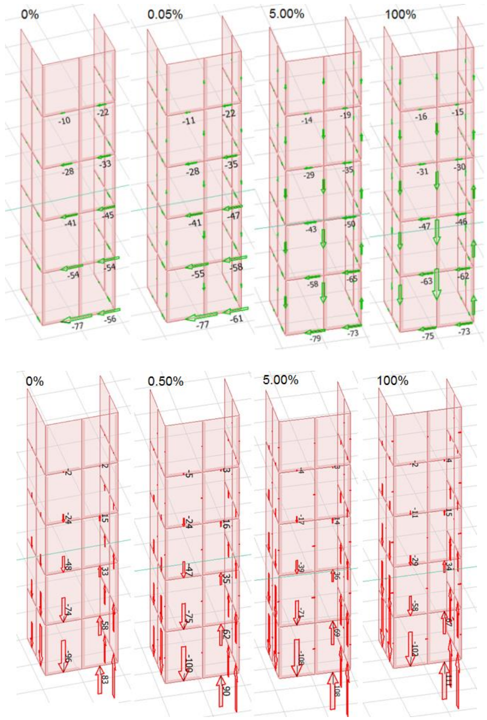


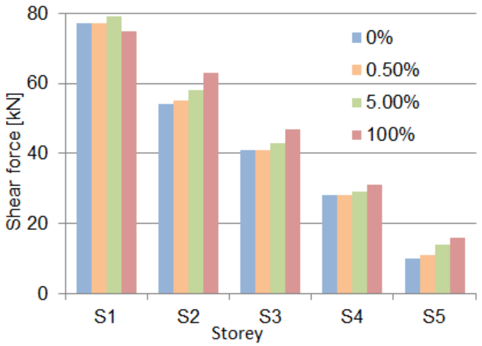


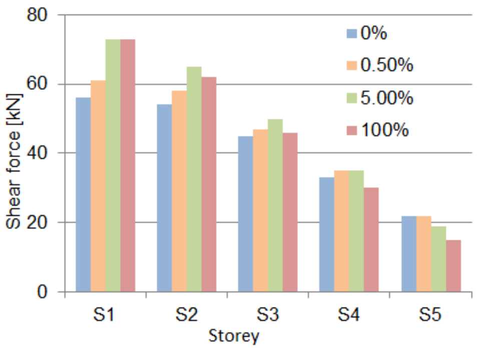
FEM global structural design

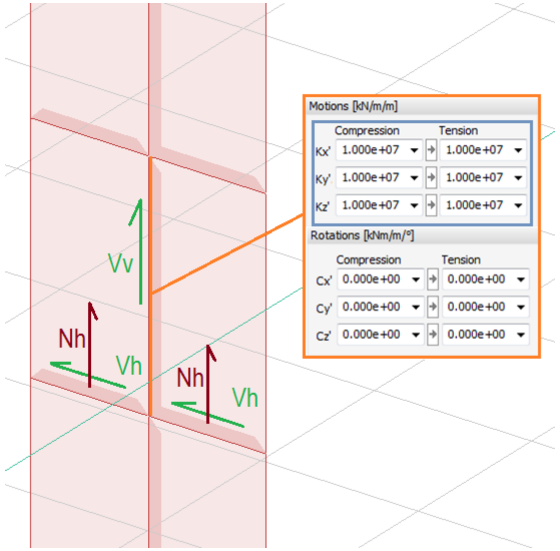
- Precast walls – Example 3
  - Structural output:
    - WX load case
    - WL walls
    - shear in the horizontal connections (Vh)
    - axial force in the horizontal connections (Nh)

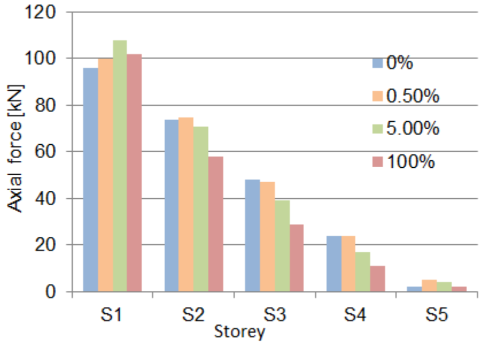


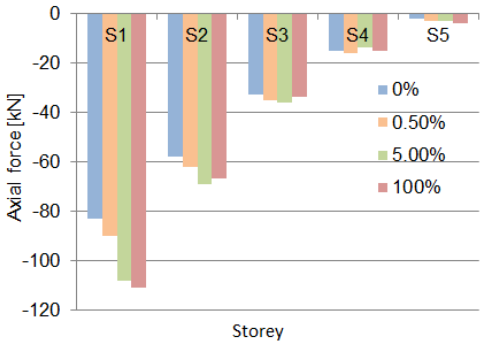












FEM global structural design

- Precast walls – Example 3
  - Structural output:
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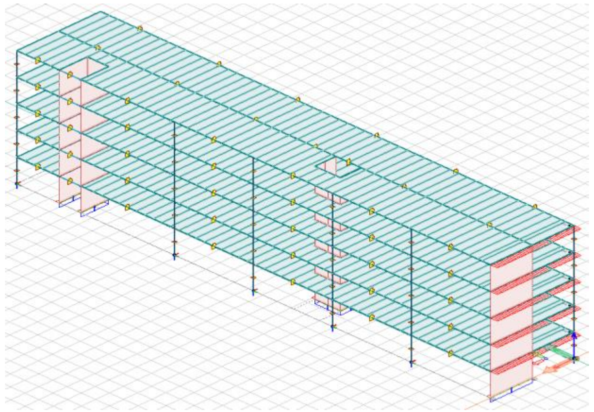
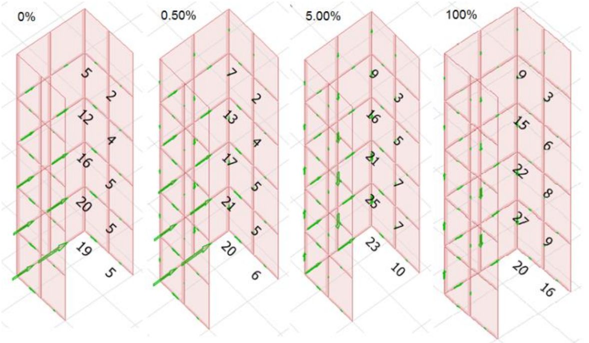
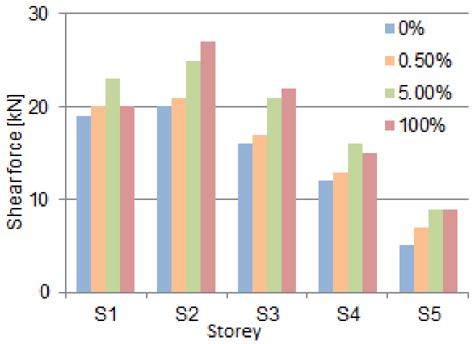
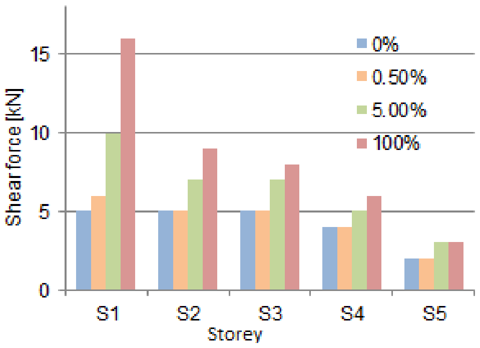
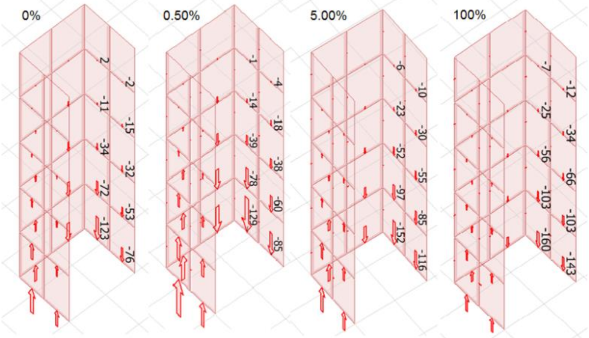
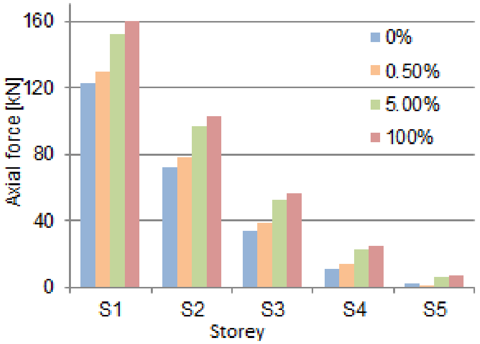


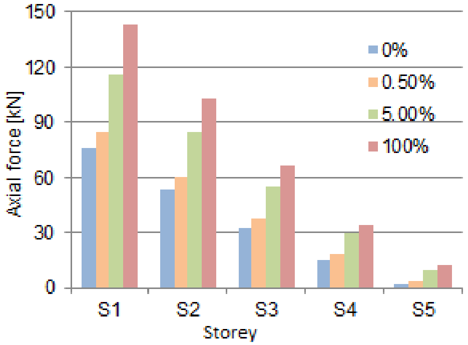












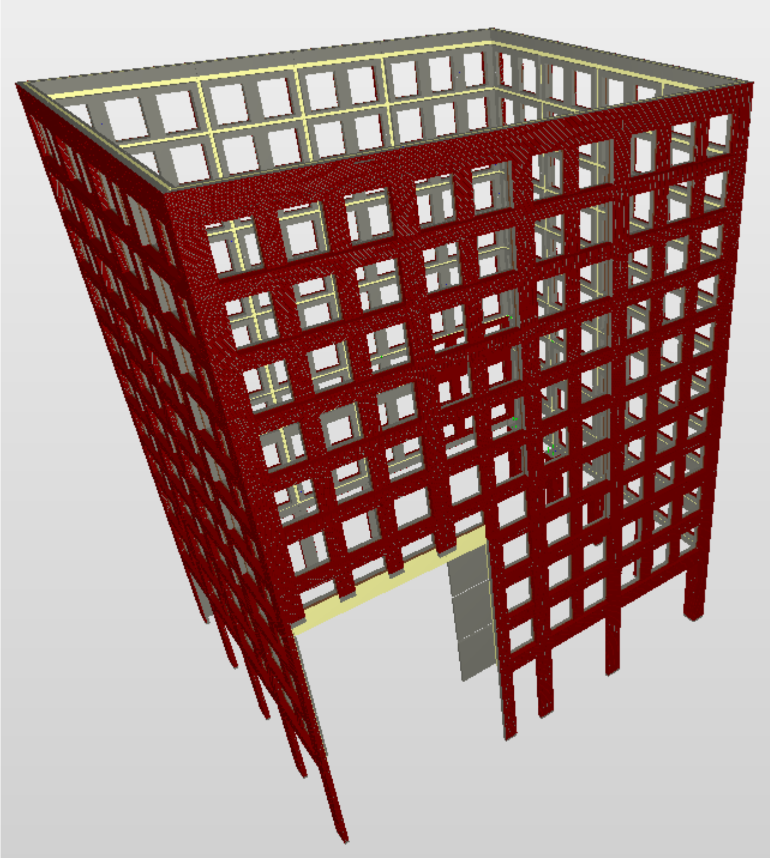
FEM global structural design

- Impact of wall panel connection stiffness on structural response
  - Horizontal precast wall connections:
    - Commonly considered stiff, equivalent monolithic
  - Vertical precast wall connections:
    - Stiffness depends on connection type:
      - Free edges - adjacent walls work independently
      - Semi-rigid flexible: e.g., steel couplers (welded plates) and no grouting
      - Semi-rigid stiff: e.g., steel couplers (welded plates) and grouting with indented edges
      - Rigid, equivalent-monolithic
    - Stiffness variations has, as a main consequence, variation of shear demand in the connection
    - Increased stiffness decreases demand on the horizontal connection and vice-versa

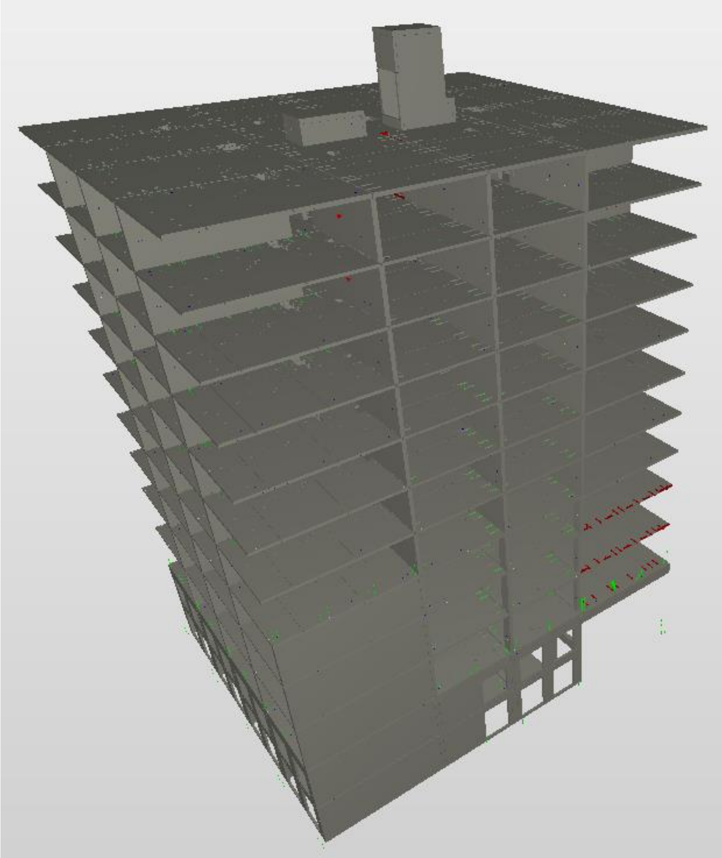
FEM modeling and analysis

Project: Spot X designed by Ingenieursstudio DCK:

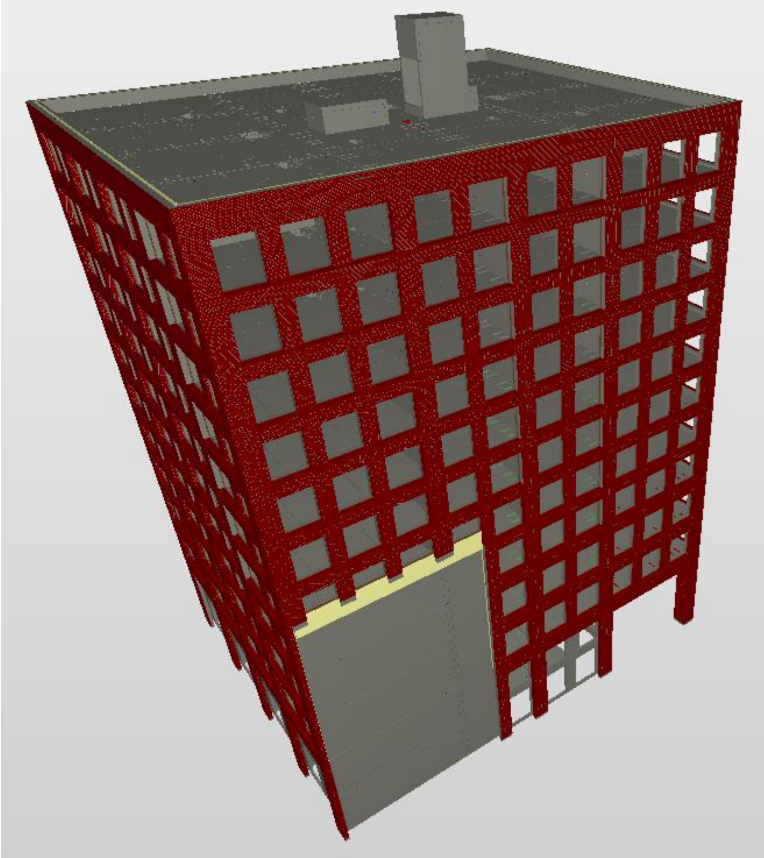
- Sandwich façade and self-supporting pre-tensioned slabs



Façade model



Slabs and inner walls

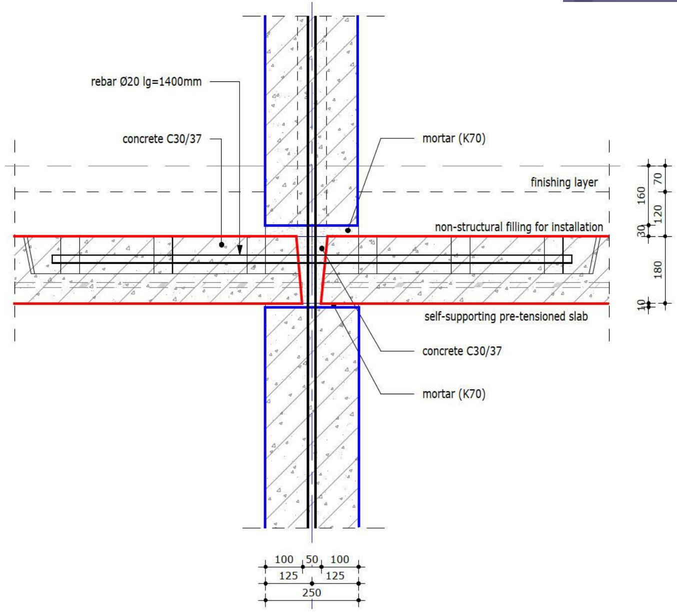


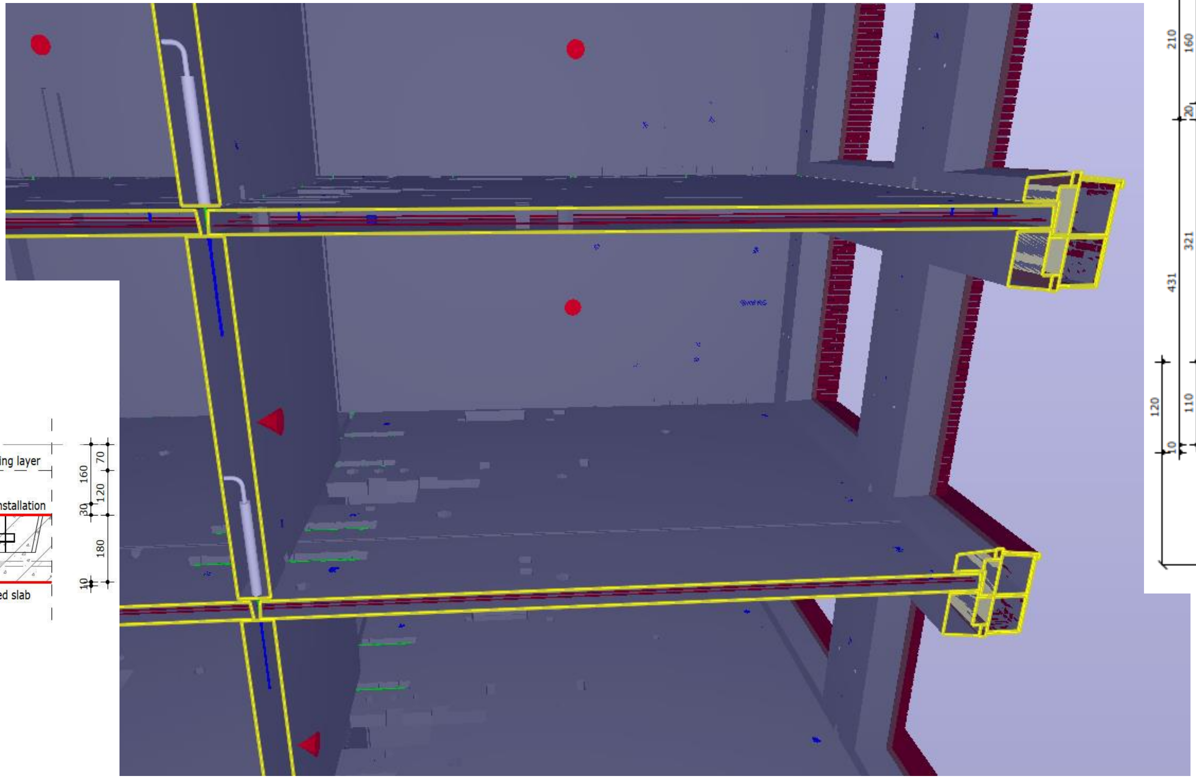
Combined model

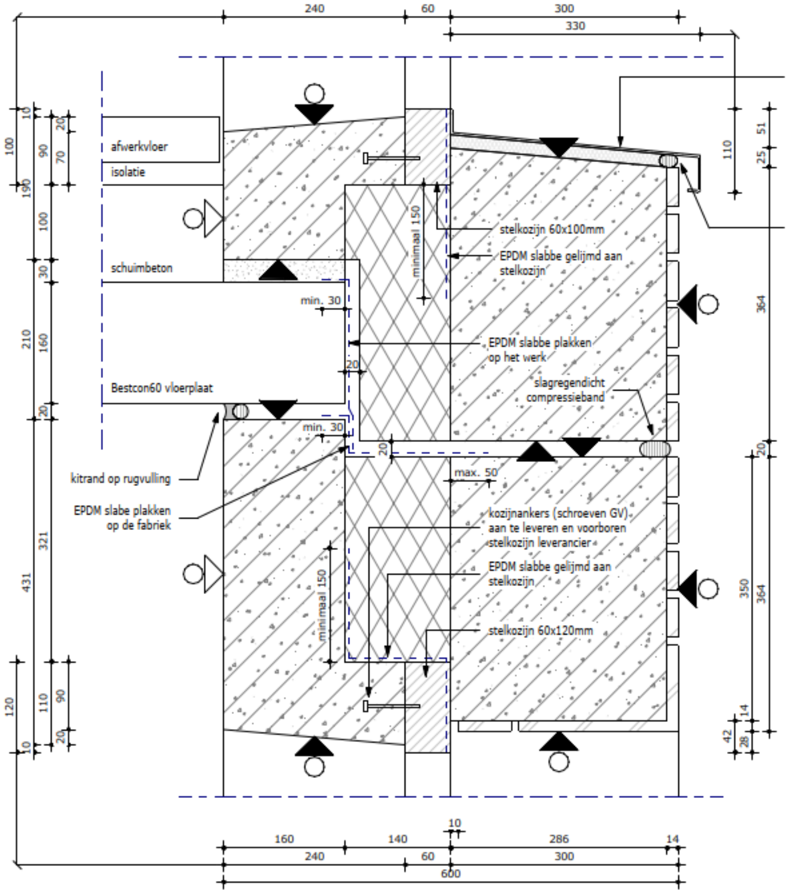
FEM modeling and analysis

Project: Spot X designed by Ingenieursstudio DCK:

- Sandwich façade and self-supporting pre-tensioned slabs



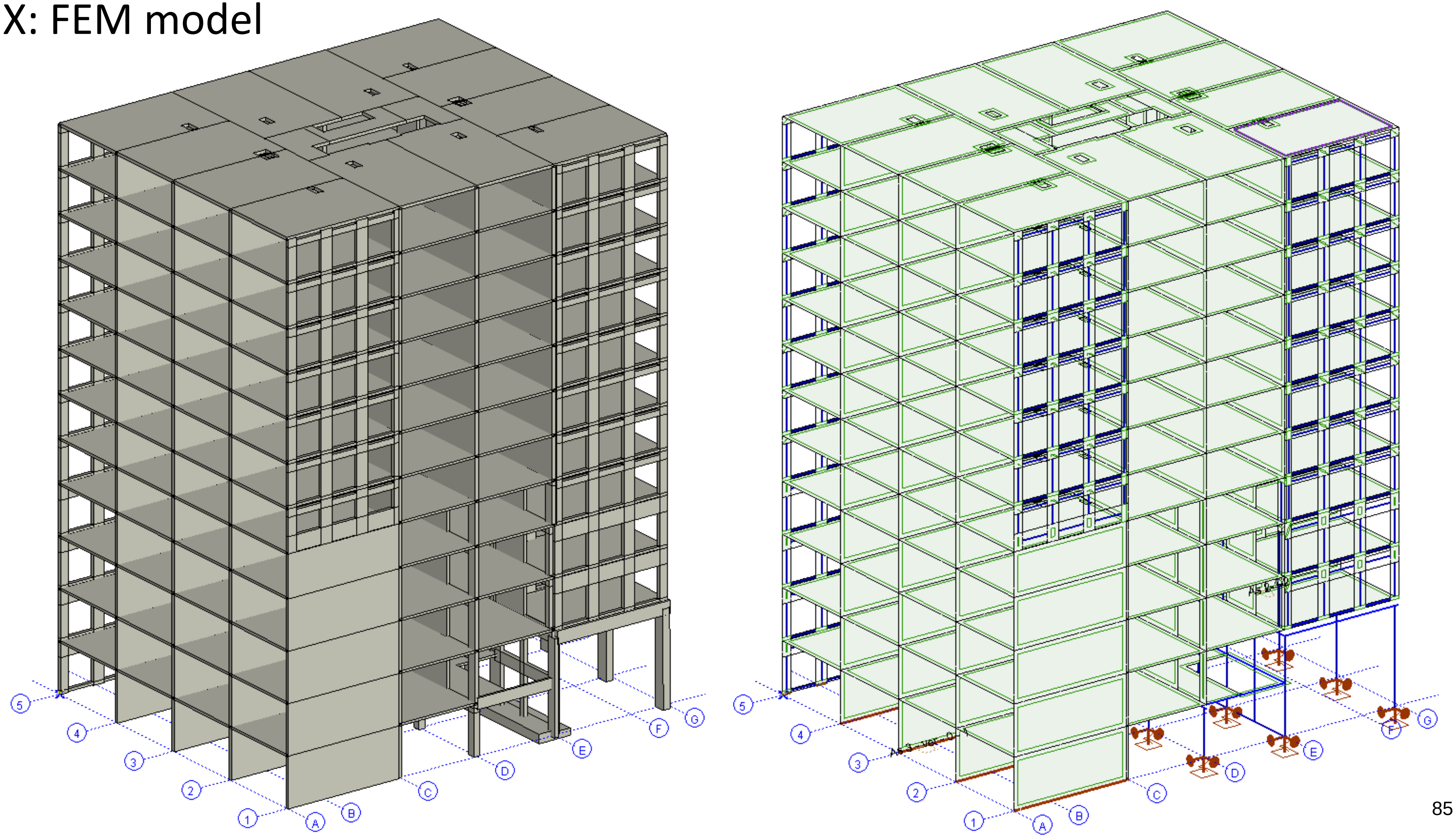




FEM modeling and analysis

Project: Spot X: FEM model

- AxisVM

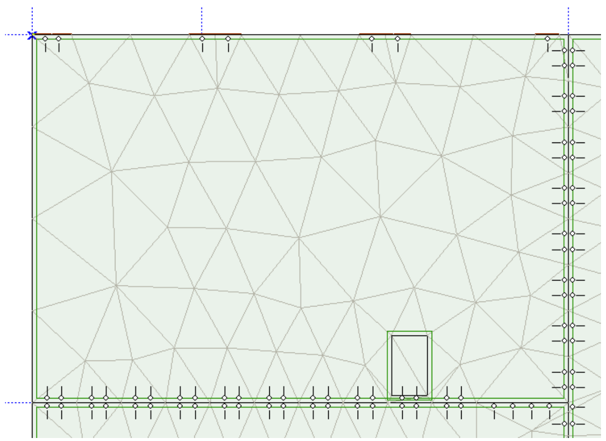


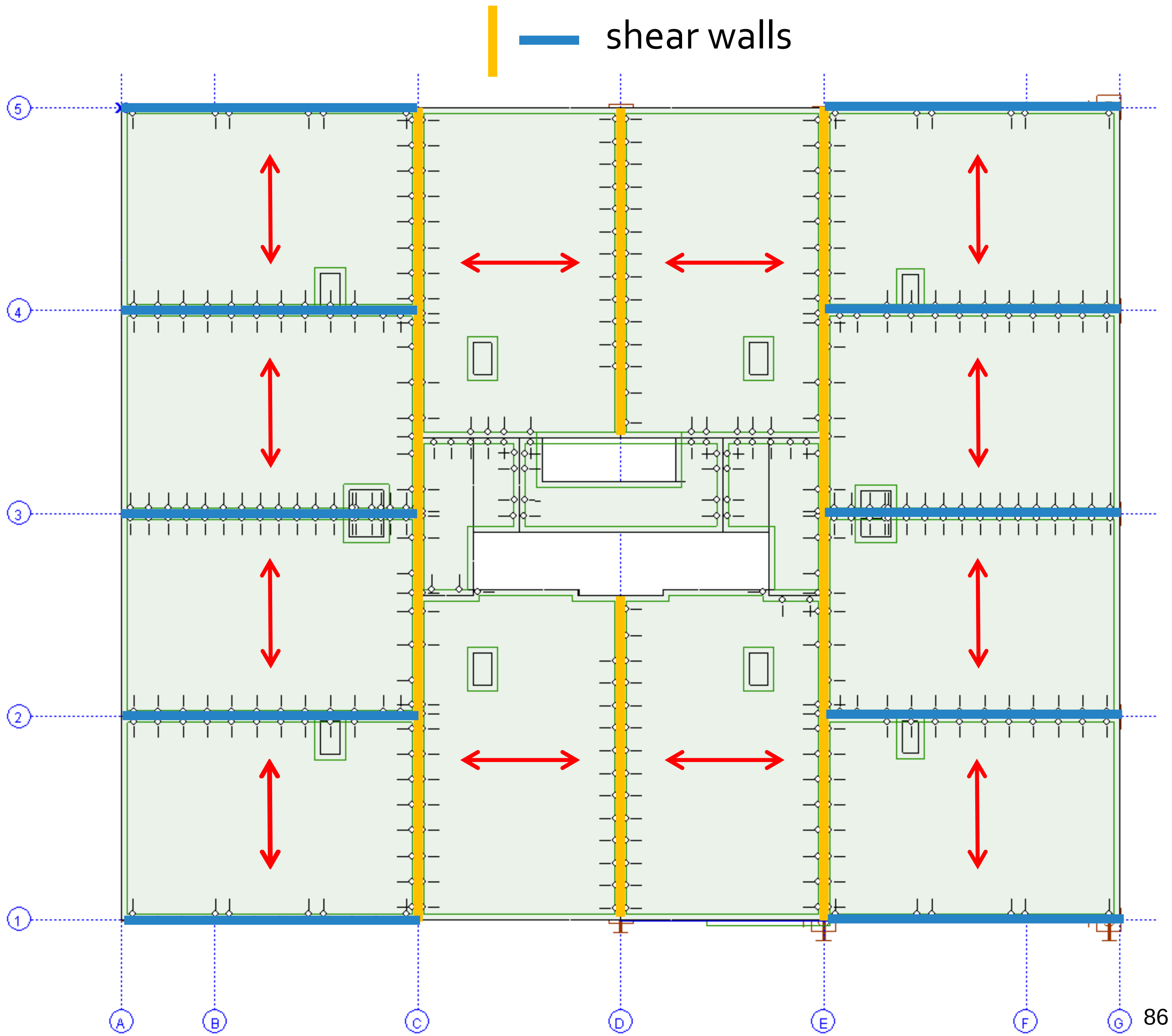
FEM modeling and analysis

Project: Spot X: FEM model

Modeling slabs:

- Shell elements, thickness = 220 mm
- Stiffness: $E_x = E_y = 12.730\ N/mm^2$
  - Including stiffness reduction →
  - k,torsion = 0,1 / k,shear = 0,1 / k,bending = 0,1
- Triangular surface elements mesh (1,5 m)





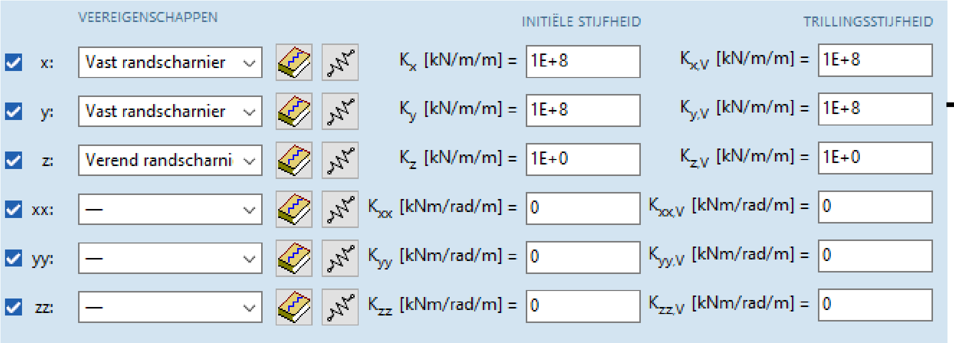
FEM modeling and analysis

Project: Spot X: FEM model

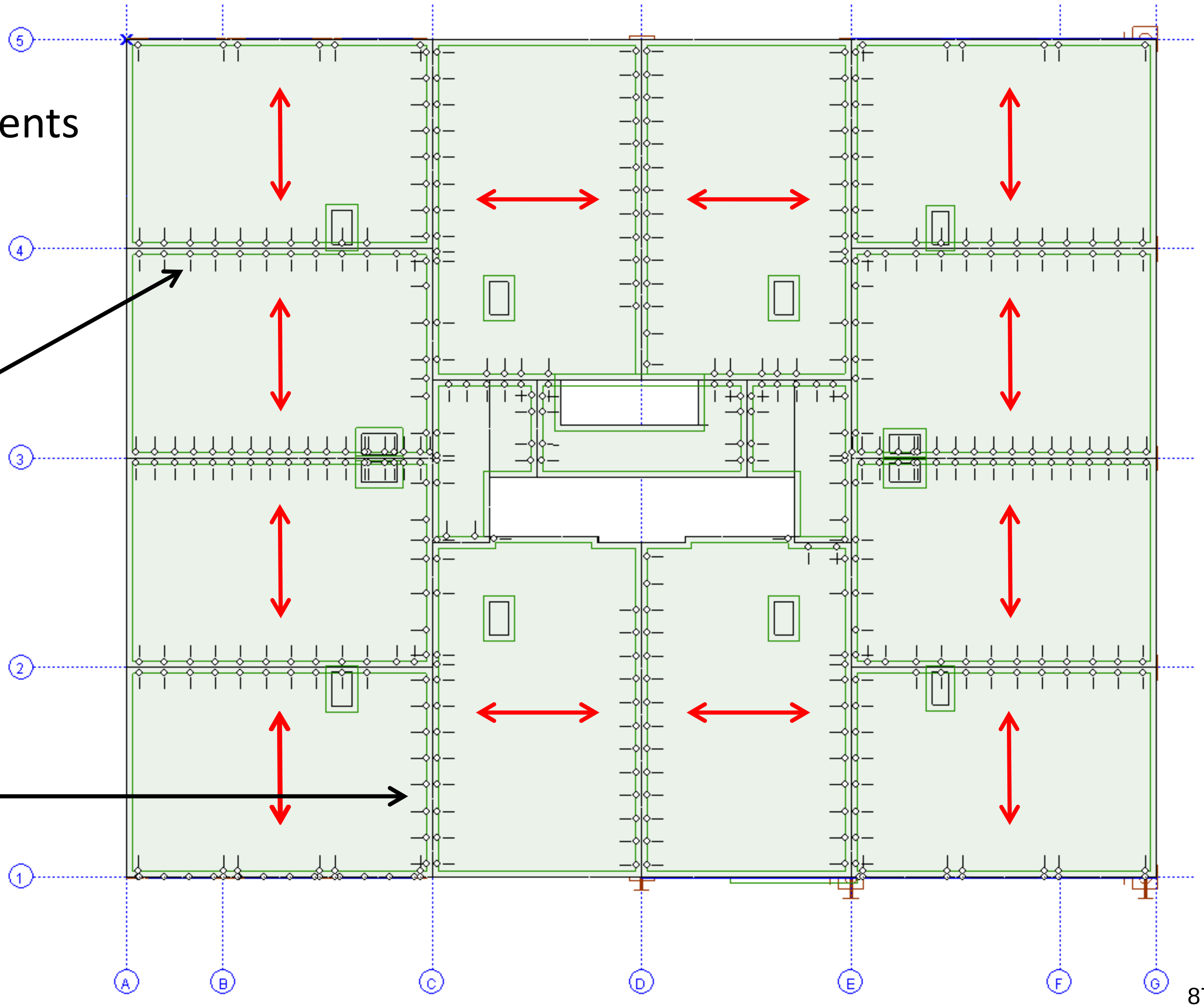
- Slab-Wall connections with interface elements

Veer

	VEEREIGENSCHAPPEN	INITIËLE STIJFHEID		TRILLINGSSTIJFHEID	
x:	Vast randscharnier	K_x [kN/m/m] =	1E+8	$K_{x,V}$ [kN/m/m] =	1E+8
y:	Vast randscharnier	K_y [kN/m/m] =	1E+8	$K_{y,V}$ [kN/m/m] =	1E+8
z:	Vast randscharnier	K_z [kN/m/m] =	1E+8	$K_{z,V}$ [kN/m/m] =	1E+8
xx:	—	K_{xx} [kNm/rad/m] =	0	$K_{xx,V}$ [kNm/rad/m] =	0
yy:	—	K_{yy} [kNm/rad/m] =	0	$K_{yy,V}$ [kNm/rad/m] =	0
zz:	—	K_{zz} [kNm/rad/m] =	0	$K_{zz,V}$ [kNm/rad/m] =	0



	VEEREIGENSCHAPPEN	INITIËLE STIJFHEID		TRILLINGSSTIJFHEID	
x:	Vast randscharnier	K_x [kN/m/m] =	1E+8	$K_{x,V}$ [kN/m/m] =	1E+8
y:	Vast randscharnier	K_y [kN/m/m] =	1E+8	$K_{y,V}$ [kN/m/m] =	1E+8
z:	Verend randscharni	K_z [kN/m/m] =	1E+0	$K_{z,V}$ [kN/m/m] =	1E+0
xx:	—	K_{xx} [kNm/rad/m] =	0	$K_{xx,V}$ [kNm/rad/m] =	0
yy:	—	K_{yy} [kNm/rad/m] =	0	$K_{yy,V}$ [kNm/rad/m] =	0
zz:	—	K_{zz} [kNm/rad/m] =	0	$K_{zz,V}$ [kNm/rad/m] =	0

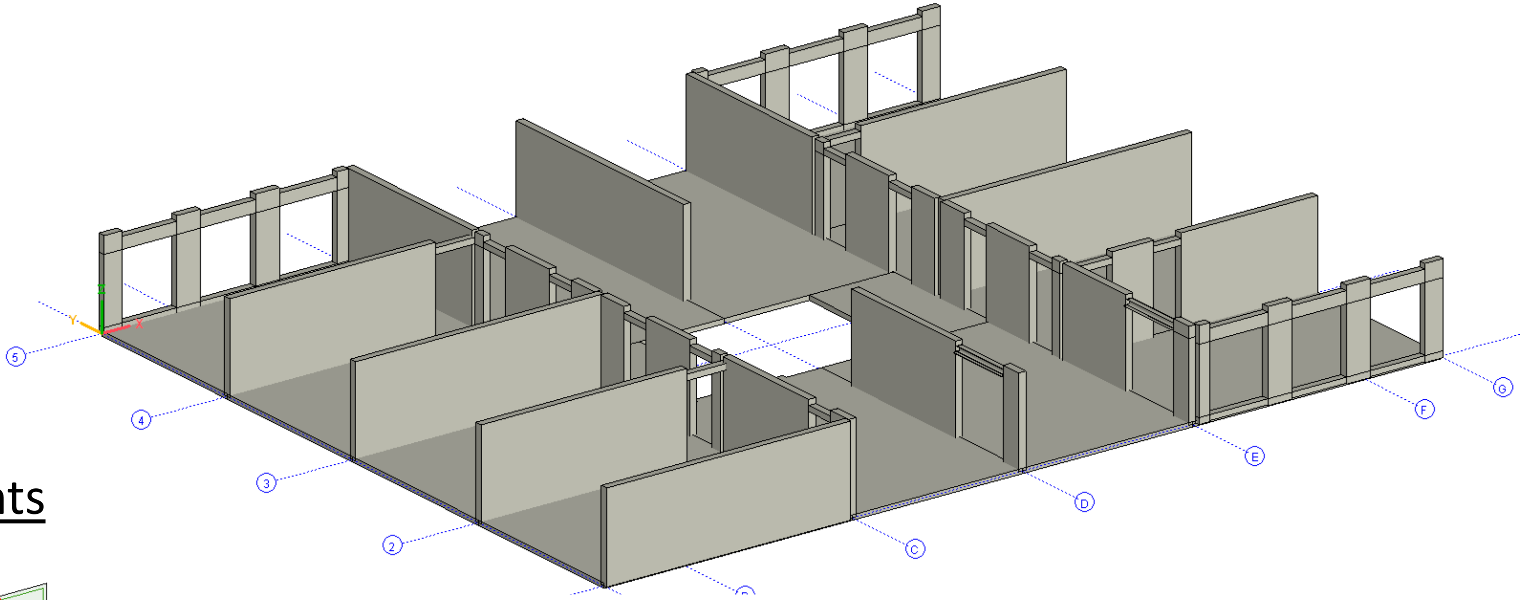


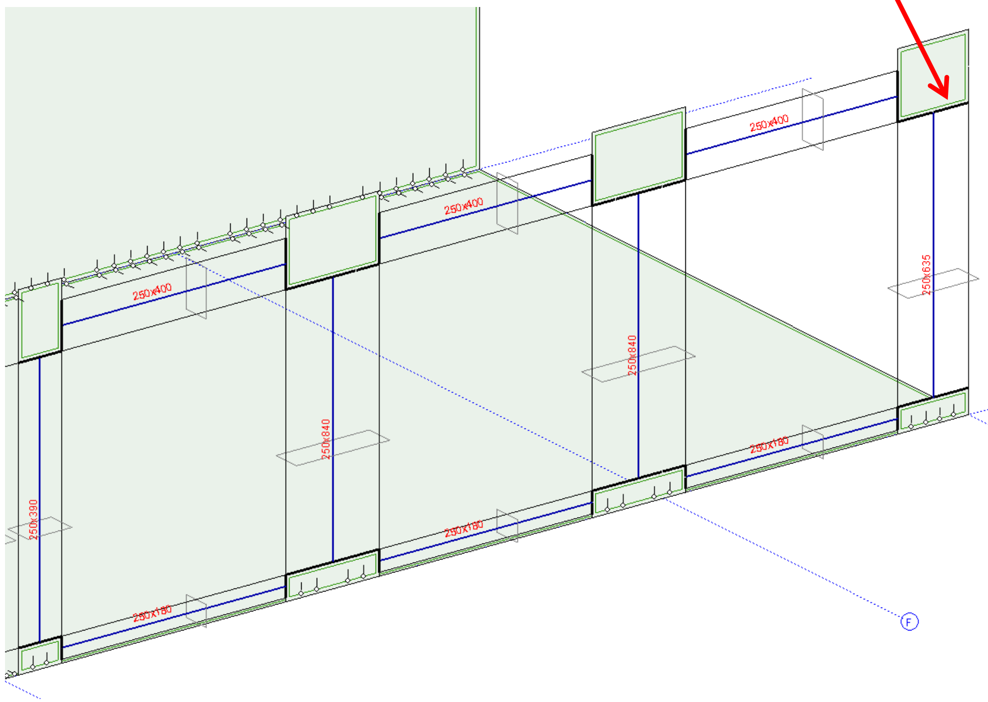
FEM modeling and analysis

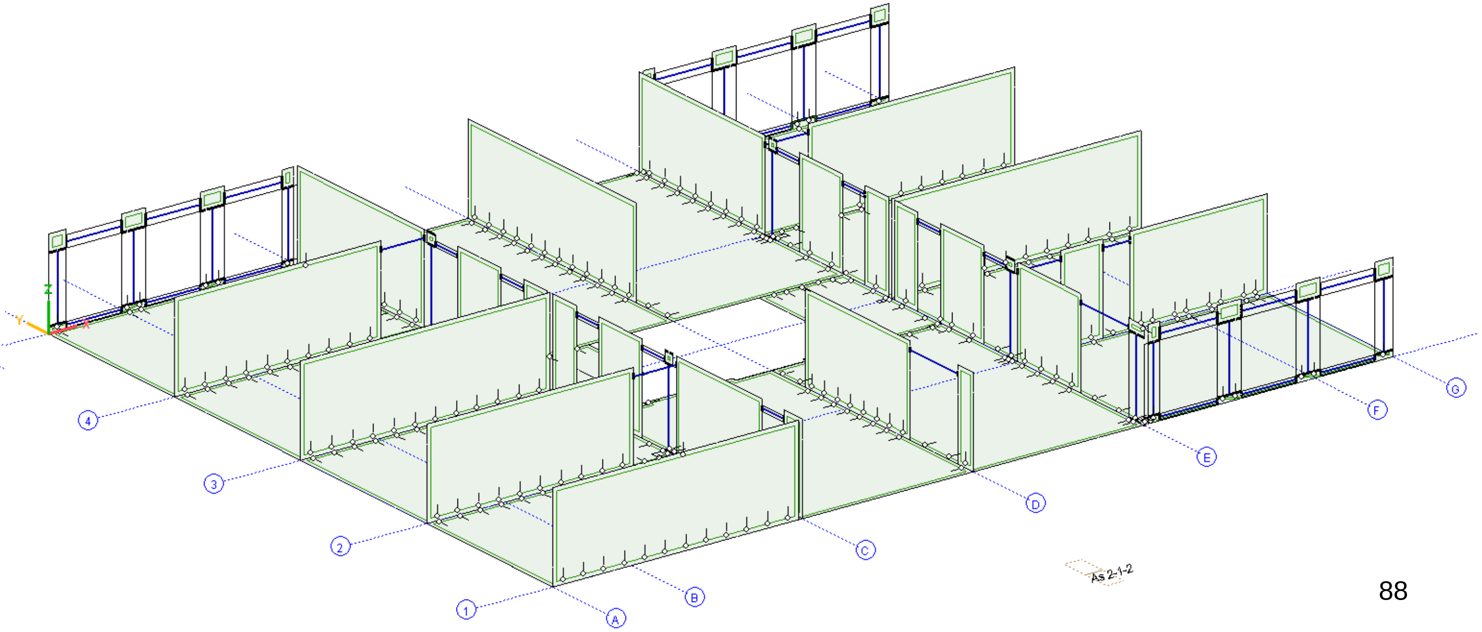
Project: Spot X: FEM model

Framed façade element:

- Lintels and Columns
- Composition Shell + Beam elements
- Mutually connected with stiff elements



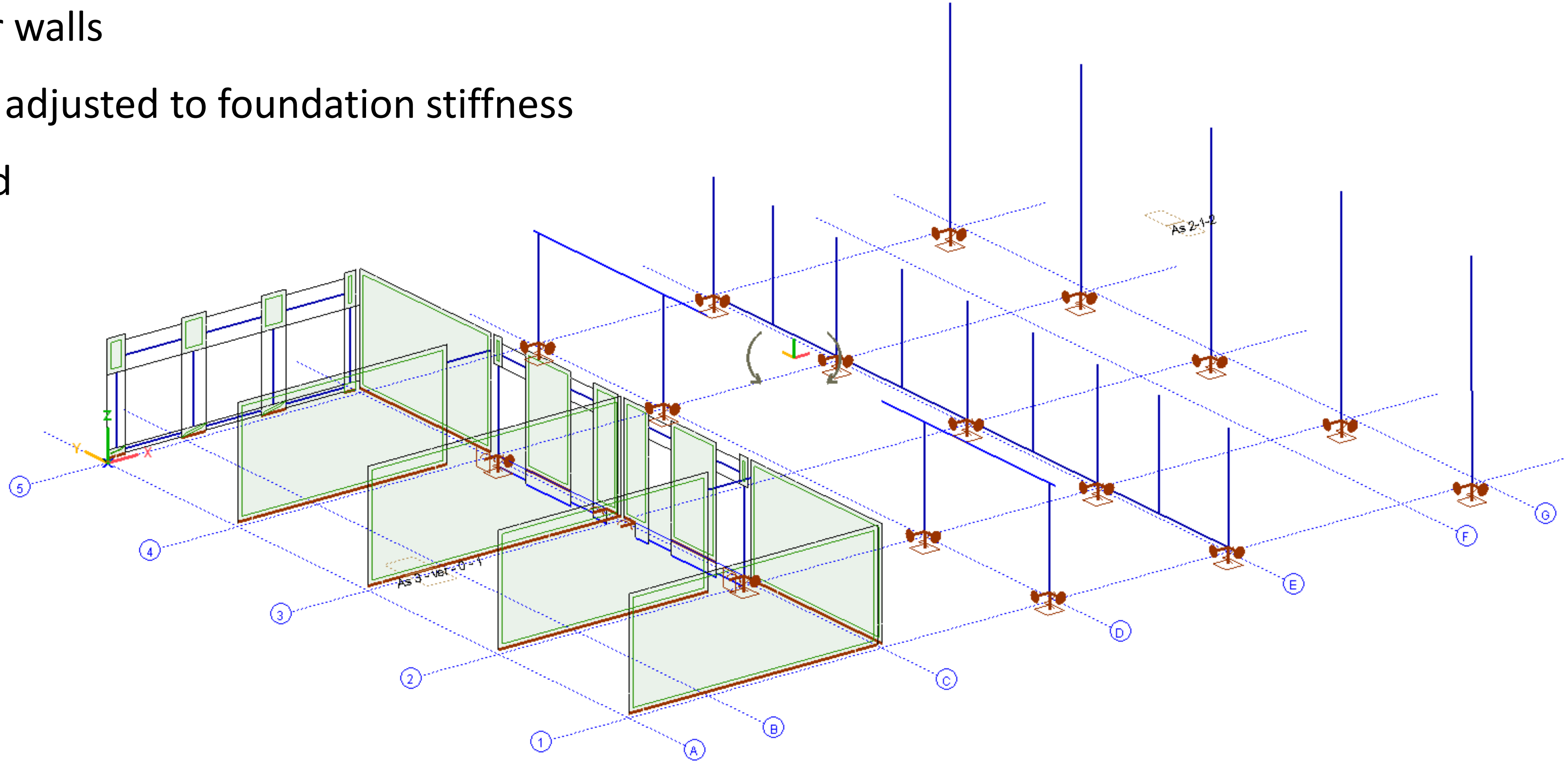




FEM modeling and analysis

Project: Spot X: FEM model

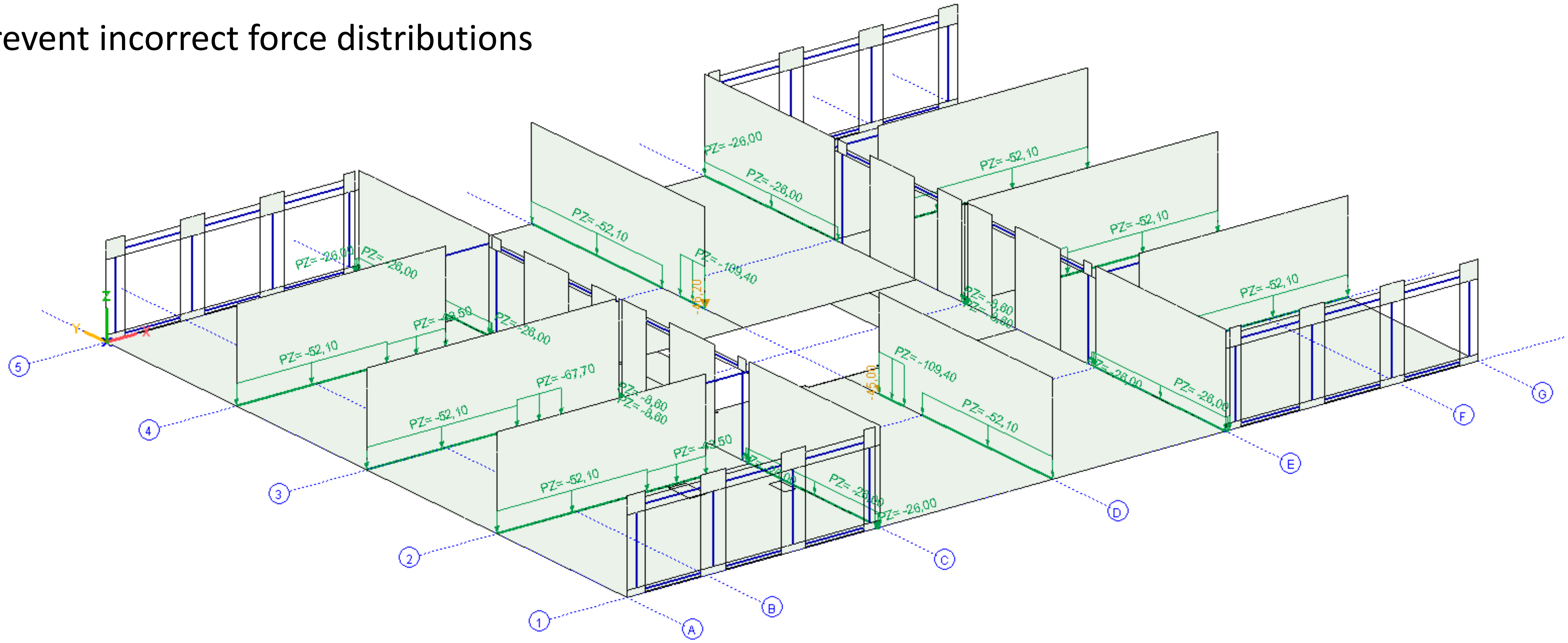
- Point supports for columns
- Line supports for walls
- Vertical stiffness adjusted to foundation stiffness
- Horizontally fixed



FEM modeling and analysis

Project: Spot X: FEM model

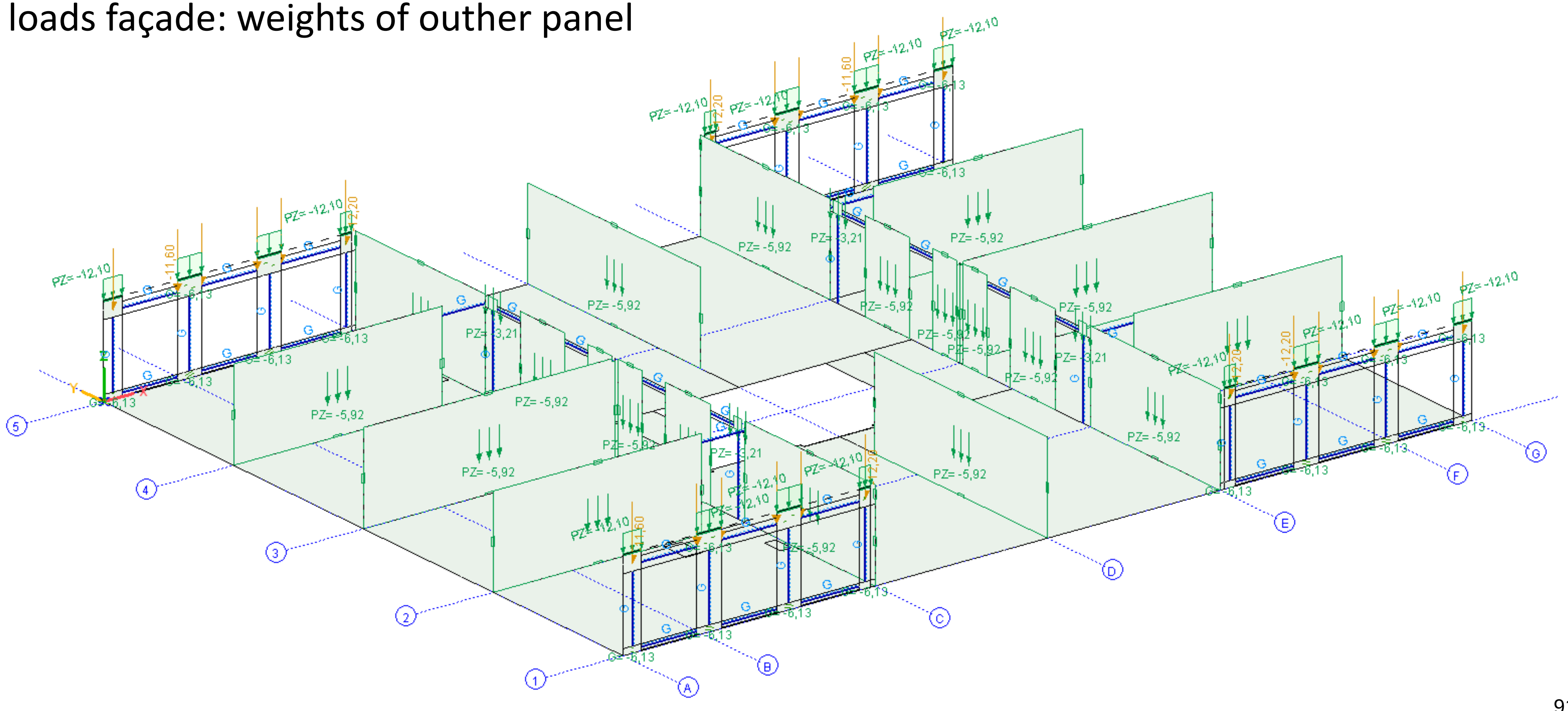
- Vertical slab loads straight to the walls
- To prevent incorrect force distributions



FEM modeling and analysis

Project: Spot X: FEM model

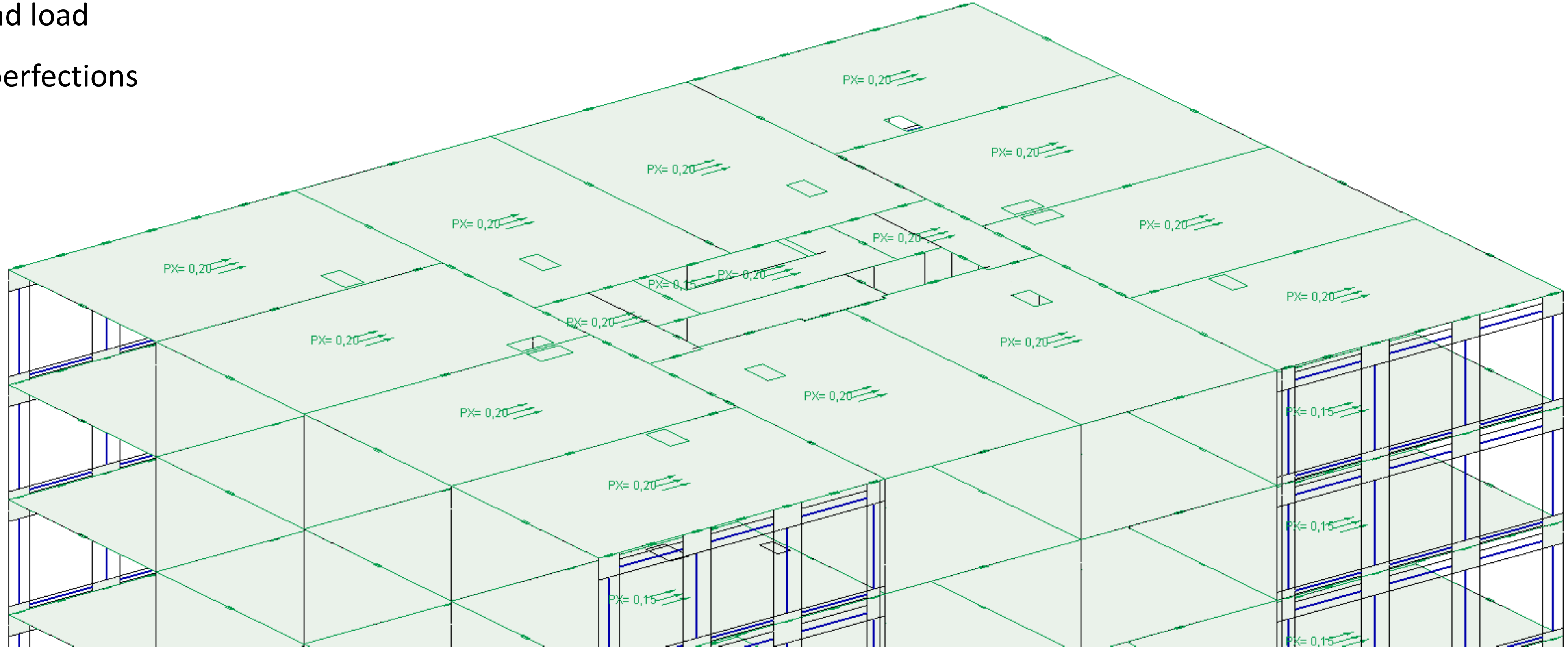
- Dead loads Walls (vertical surface load)
- Line /point loads façade: weights of outher panel



FEM modeling and analysis

Project: Spot X: FEM model

- Lateral in-plane surface loads
  - Wind load
  - Imperfections



Agenda/ 10:00 – 11:40

- Introduction
- Design and calculations for precast concrete building structures:
  - Specifics for element lay-out design
  - Requirements in current and next generation of Eurocodes
  - FEM modeling and analysis of structures composed of HCS and Shear Walls
- Q&A session

Break / 11:40 – 12:00

Evaluation of shear wall behavior

- Design and modeling of precast concrete shear walls:
  - Overview of wall panel connections
  - Element lay-outs with connections
  - Connection stiffnesses for FEM
- Evaluation of shear wall behavior:
  - Connection stiffnesses
  - Overall shear wall behavior
  - Simplified control calculations
  - Verification of bearing capacities
- Q&A session

Evaluation of shear wall behavior

- Design and modeling of precast concrete shear walls:
  - FEM modeling of shear walls
  - Precast wall panels connections experimental study

Design and modeling of precast concrete shear walls

- FEM commercial software

midas Gen

ETABS®

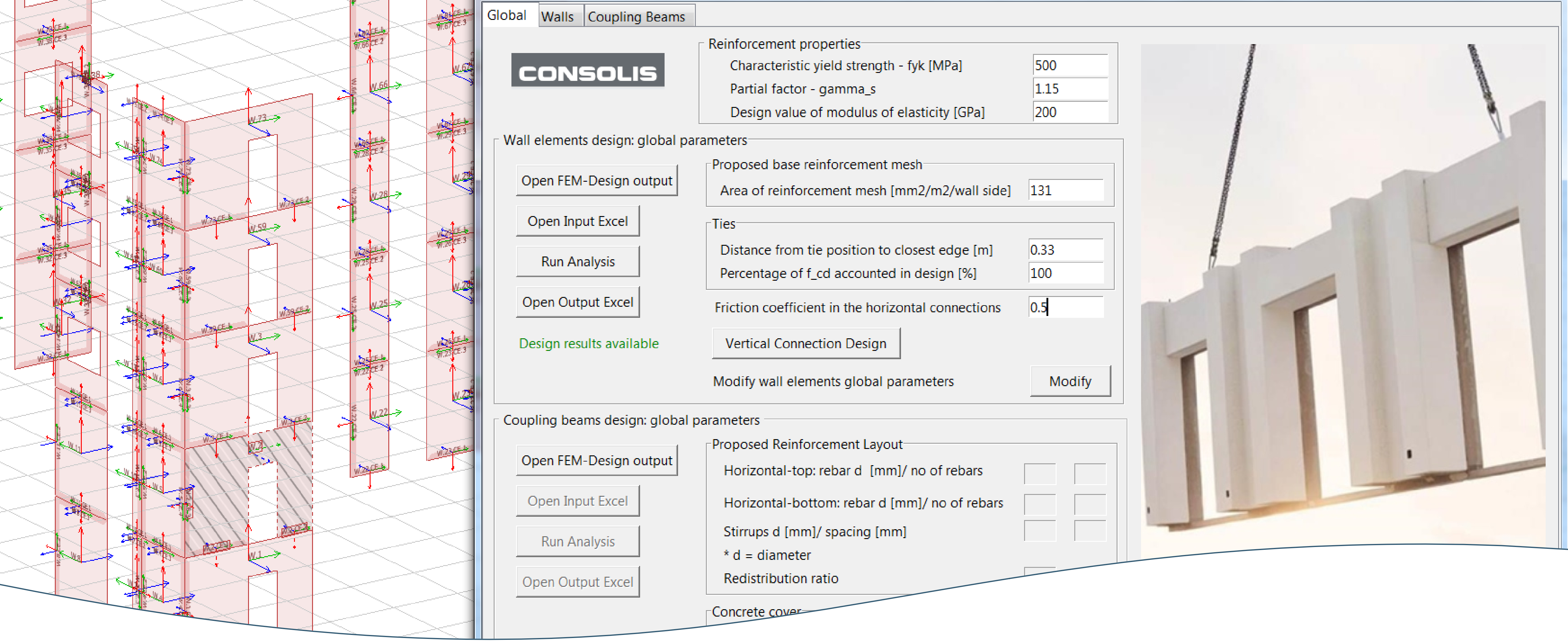
StruSoft FEM-Design

Dlubal

AUTODESK Robot Structural Analysis Professional

AXISVM

SAP2000

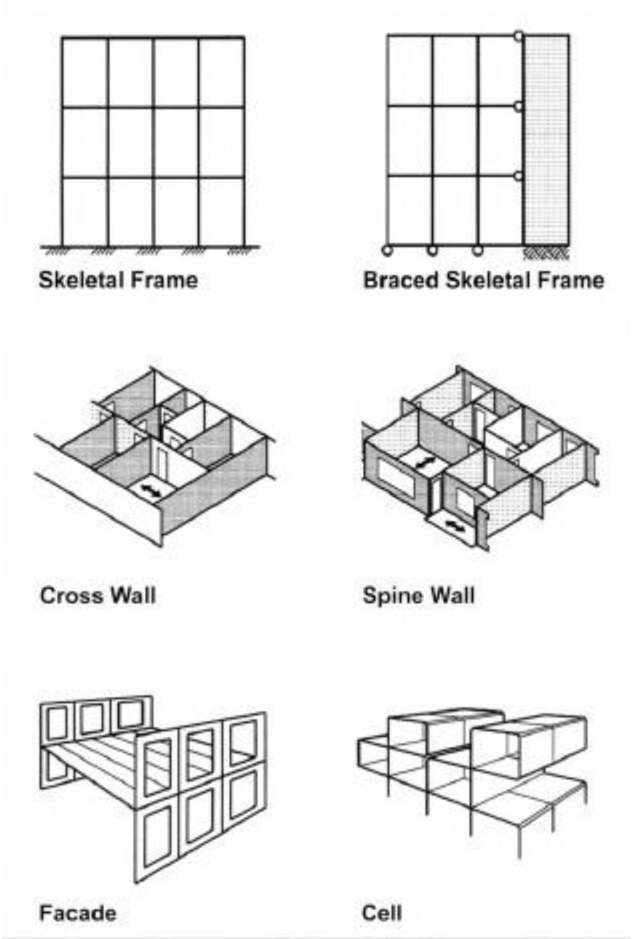


CEN-Brace

automated design of bracing walls

Design and modeling of precast concrete shear walls

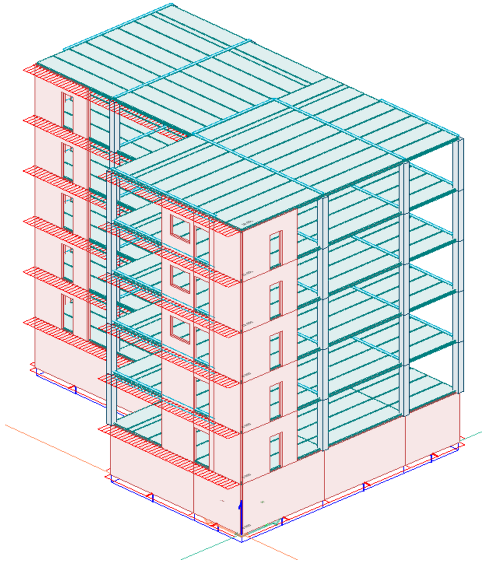
- Precast concrete shear walls



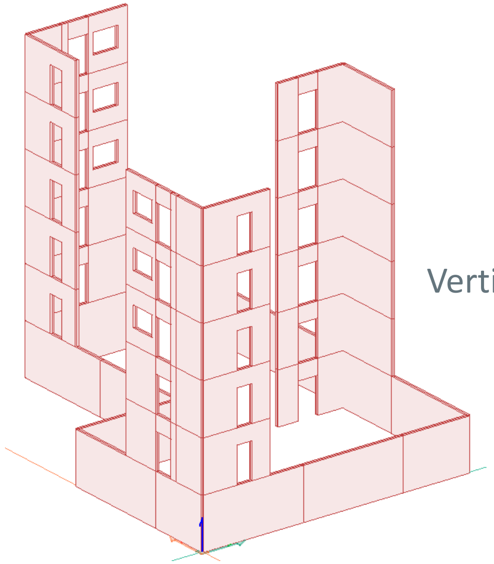
Precast concrete building systems (Structural Precast Concrete Handbook)

Design and modeling of precast concrete shear walls

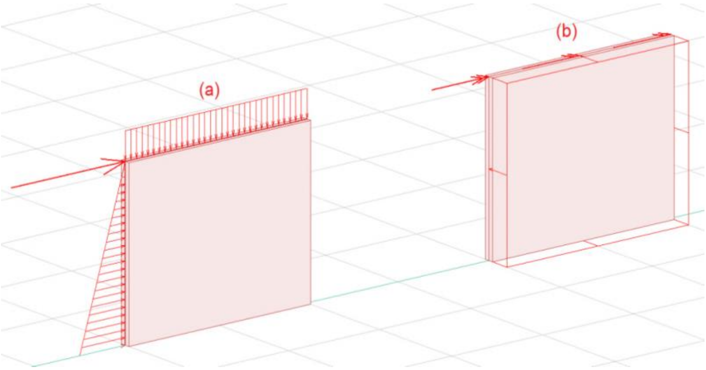
- Shear walls
  - Purpose
    - Resists lateral loads
    - Precast wall panels resist lateral loads as in-plane loads
    - Together with the slabs → lateral resisting system
    - Global structural response needed to get local element design forces
  - Design needs to be consistent
    - global analysis should match element design assumptions:
      - are lintels behaving as coupling beams?
      - do the selected wall connection stiffness match connection type?



FEM model of structural system subjected to wind loads



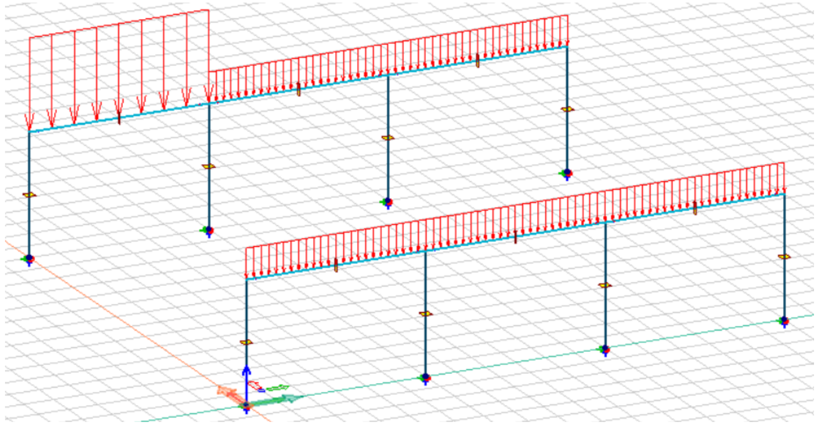
Vertical bracing system made of precast bracing walls



(a) In plane and (b) out of plane loads acting on a wall panel

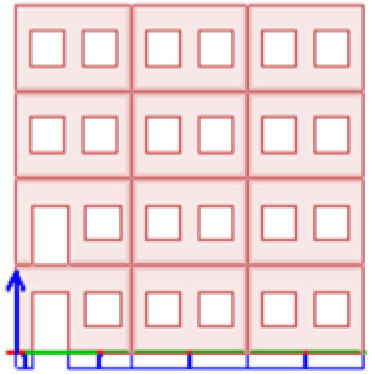
Design and modeling of precast concrete shear walls

- Shear walls
  - Why is global design needed for the design of bracing walls?
  - Differentiation can be made between:
    - Structural elements that can be designed as stand-alone elements
    - Structural elements that need to account for neighboring conditions

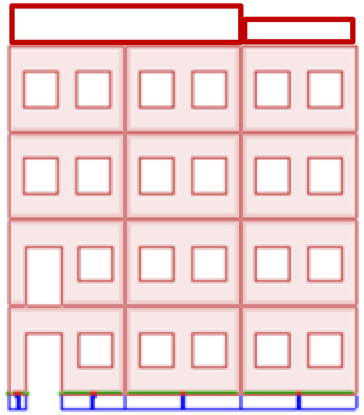


Simply supported beams on cantilever columns

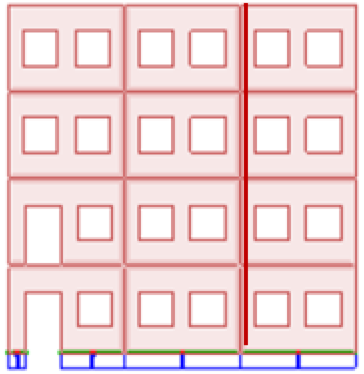
a. Initial model



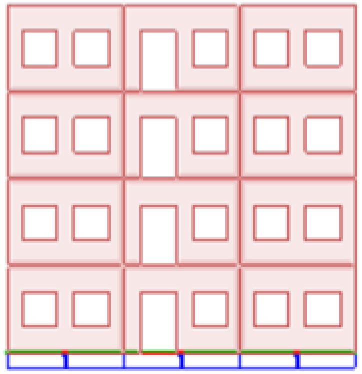
b.1 local modification of loads



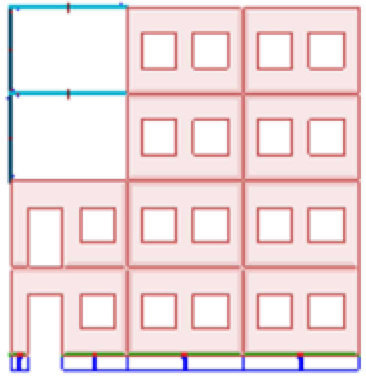
b.2 vertical connection stiffness modification



b.3 position of openings modification

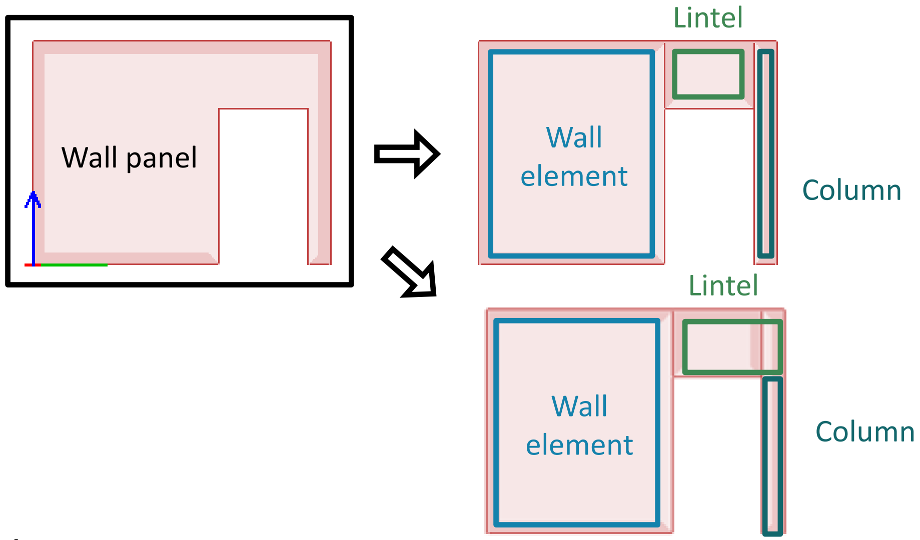


b.4 modification of overall bracing system



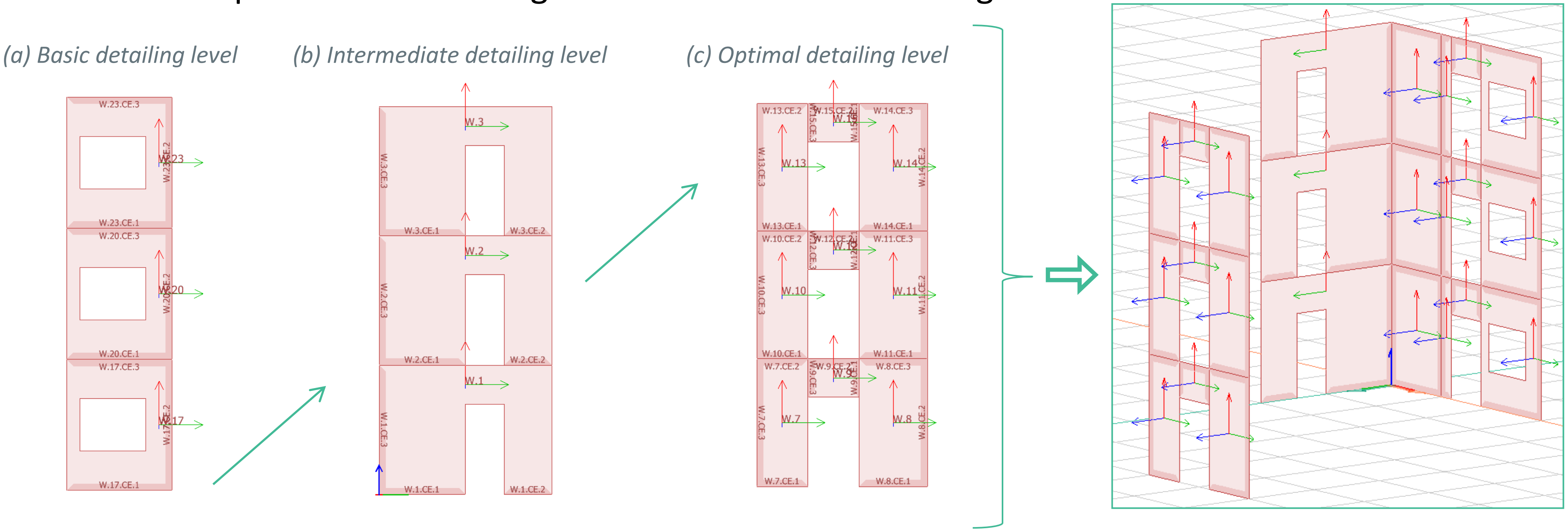
Design and modeling of precast concrete shear walls

- Shear walls
  - Precast wall panels
    - Can be considered as assemblies of:
      - Wall elements
      - Coupling beams/Lintels
      - Columns
    - Output is linked to the global model level of detailing



Different approaches to model the subassemblies of a precast panel

(a) Basic detailing level *(b) Intermediate detailing level* *(c) Optimal detailing level*

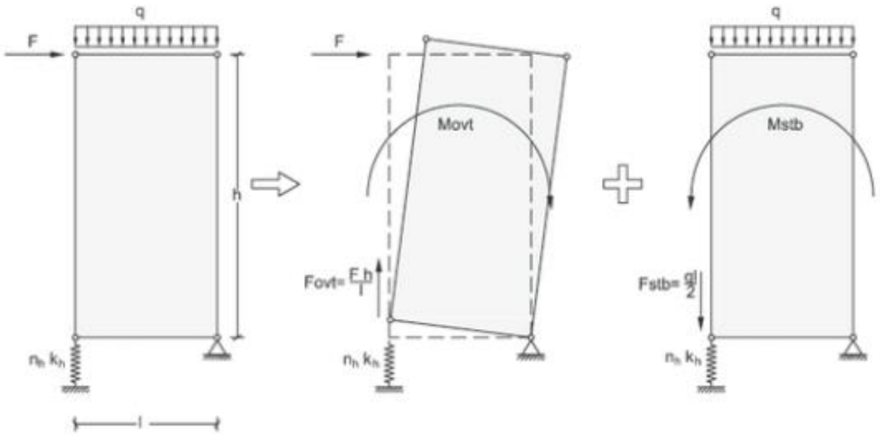


All detailing levels can coexist in the same FEM model

Design and modeling of precast concrete shear walls

- Shear walls
  - Rigid body or deformable body?
  - Rigid body:
    - Does not deform when subjected to loads
    - Overturning verification
  - When can we consider rigid body behavior of wall panels?
    - No limits provided in Eurocode
    - It depends on:
      - Geometry of the wall panel
      - Density of walls
      - Intensity of lateral loads

Rigid body *Deformable body*

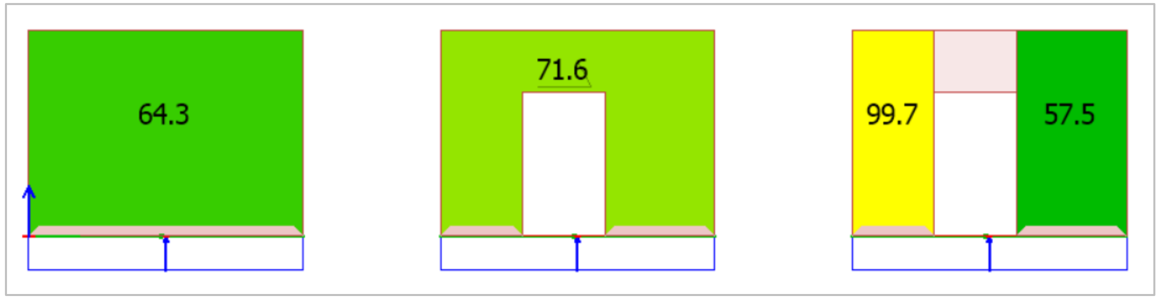


Principle of overturning verification

Design and modeling of precast concrete shear walls

- Shear walls
  - Overturing verification

- Overturning verification variation depending on wall model



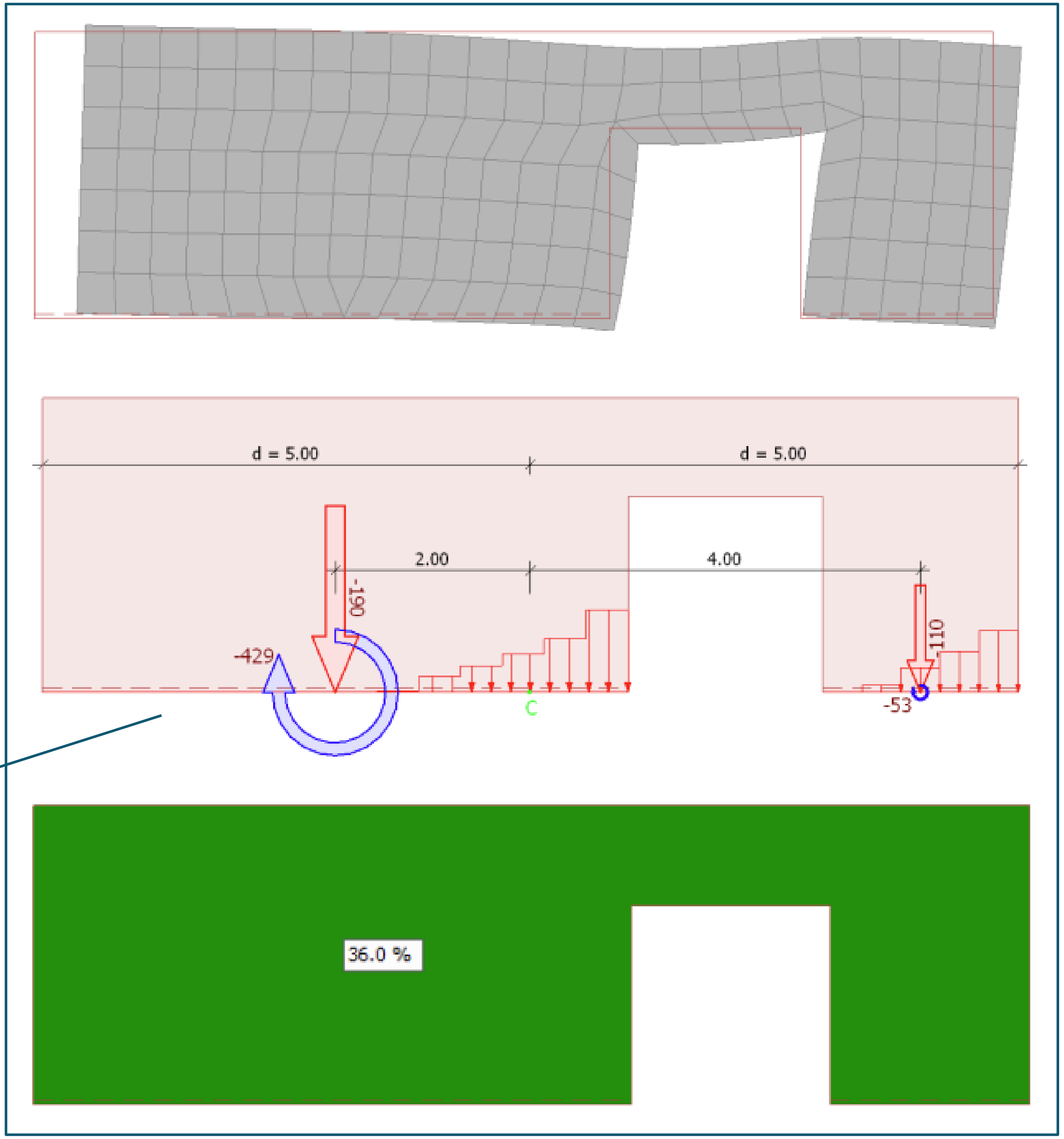
- Example of overturning verification (FEM-Design manual)

$$F = \sum_{i=1}^{n} F_i = -190 - 110 = -300\ kN$$

$$M^{(C)} = \sum_{i=1}^{n} M_i + F_i * v_i = -429 + 190 * 2 - 53 - 110 * 4 = -542\ kNm$$

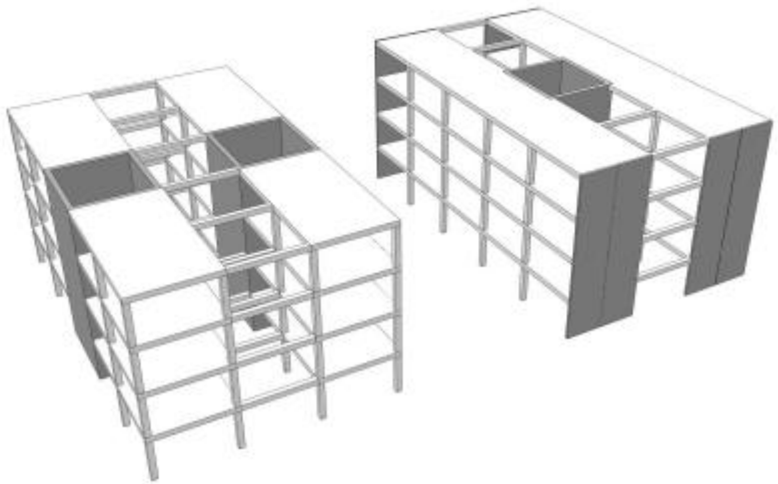
$$e = \frac{M^{(C)}}{F} = \frac{-542}{-300} = 1.81\ m$$

$$Utilization = \frac{e}{d} * 100\ \% = \frac{1.81}{5.00} * 100 = 36\%$$



Design and modeling of precast concrete shear walls

- Shear walls
  - Overturing verification?

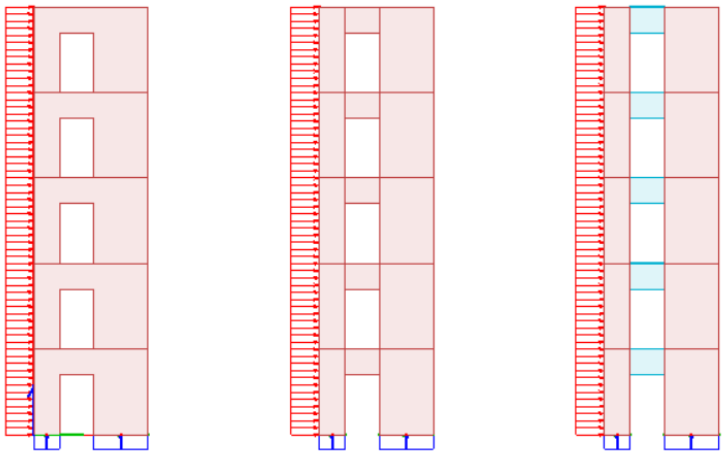


Dual frame-wall structural systems (fib 74)

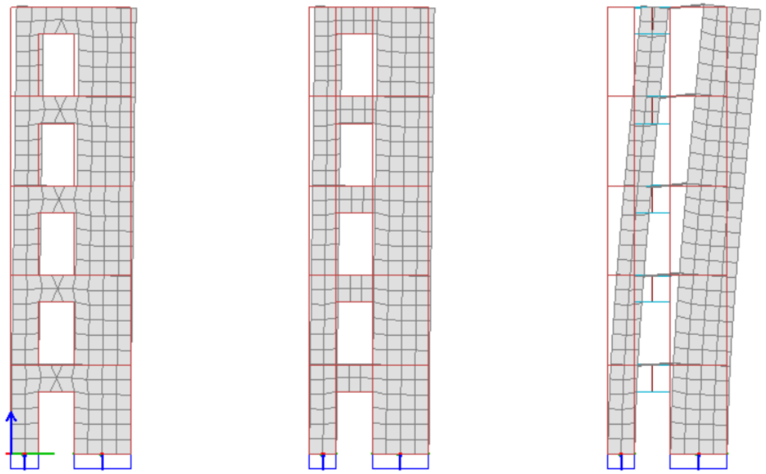
Illustration of load bearing wall structure (fib 74)

Design and modeling of precast concrete shear walls

- Coupled and uncoupled walls
  - Coupled walls:
    - Lintels behave as coupling beams
    - Sharp increase of internal forces in lintels
    - Lower internal forces in wall elements
    - Small lateral displacement
  - Uncoupled walls:
    - Lintels behave as hinged beams
    - Limited internal forces in lintels
    - Increased internal forces in wall elements
    - Large lateral displacements
  - Pros and cons?



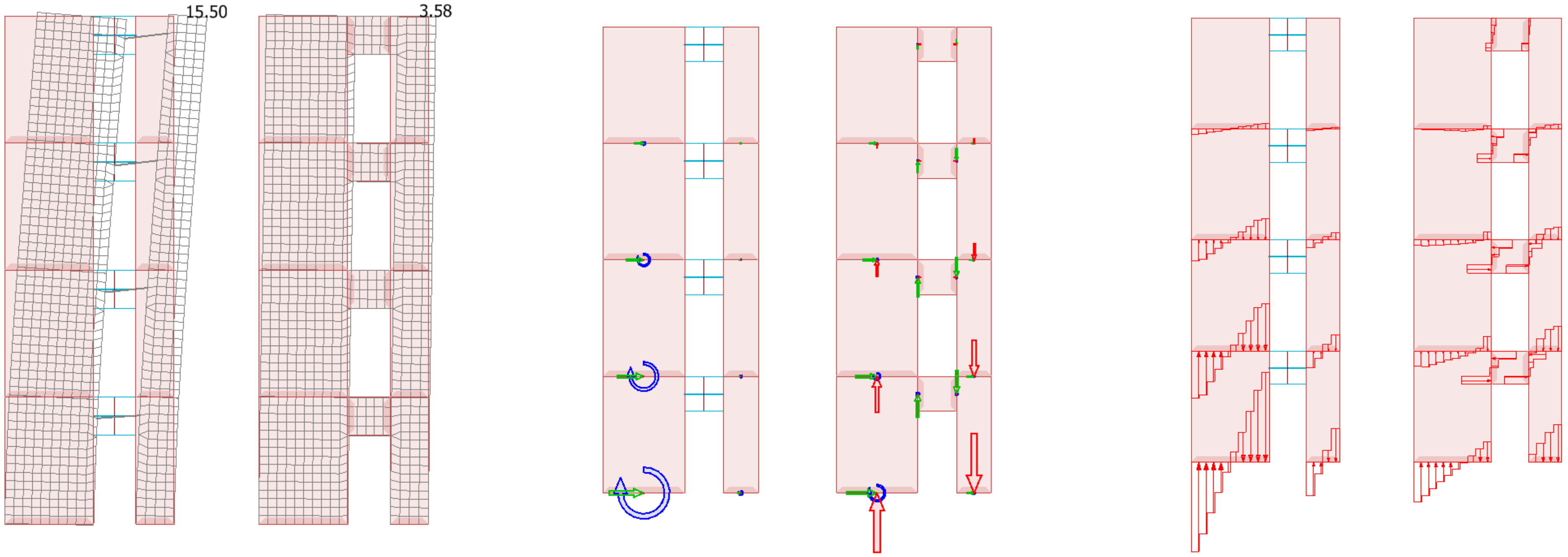
Coupled and uncoupled walls models in FEM-Design



Lateral displacement of coupled and uncoupled walls

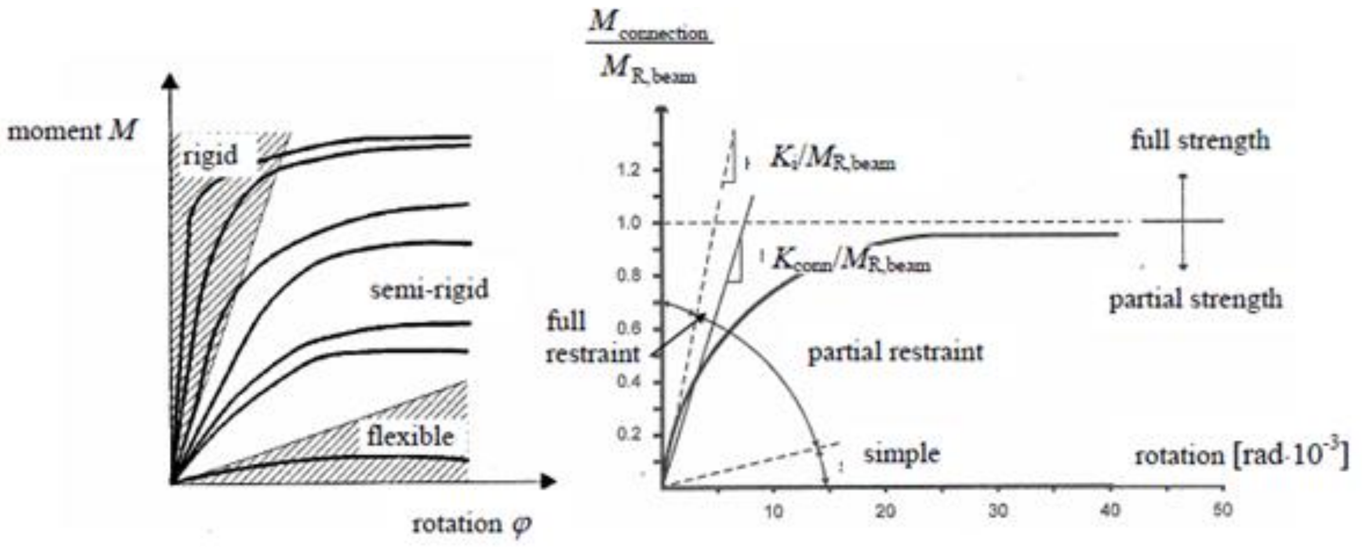
Design and modeling of precast concrete shear walls

- Coupled and uncoupled walls
  - Coupled vs. uncoupled walls



Lateral displacements and internal forces distribution in coupled/uncoupled walls subjected to lateral load

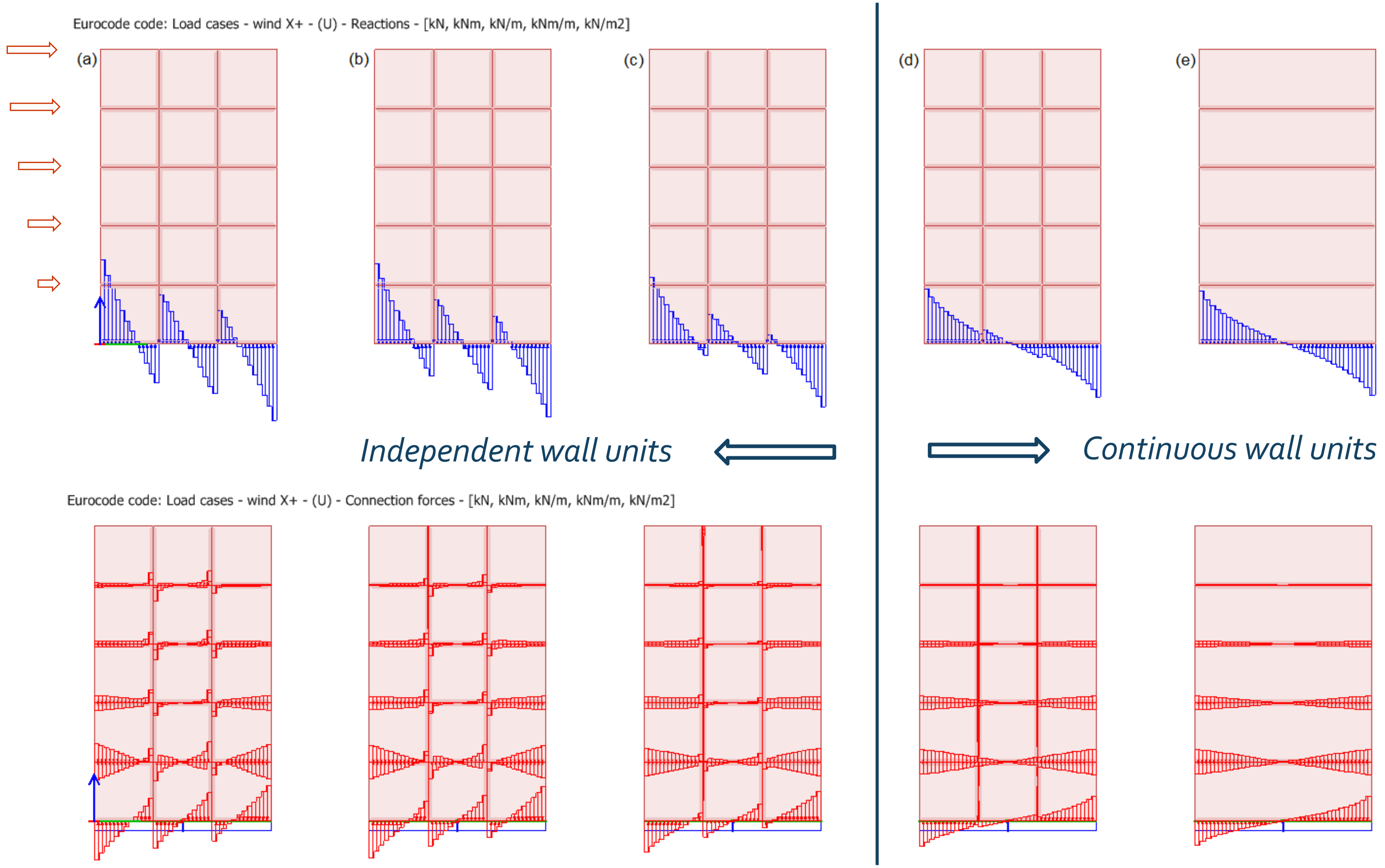
- Influence of connection stiffness on structural response
  - Theory: “pure” hinges and fixed connections
  - Precast beams and columns:
    - Bending moment vs rotation defines the stiffness of the connection/section
  - Precast wall connections:
    - Shear versus vertical displacement defines the stiffness of wall panel connections

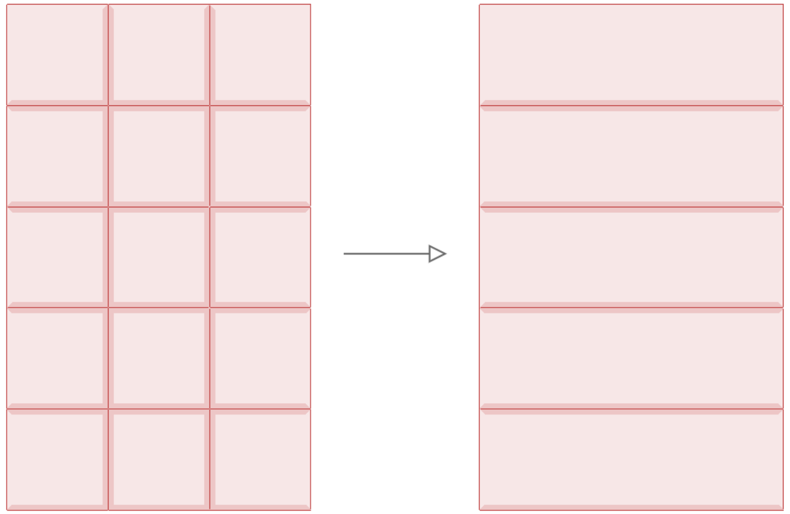


Classification of moment-resisting connections – (Leon, 1998)

Design and modeling of precast concrete shear walls

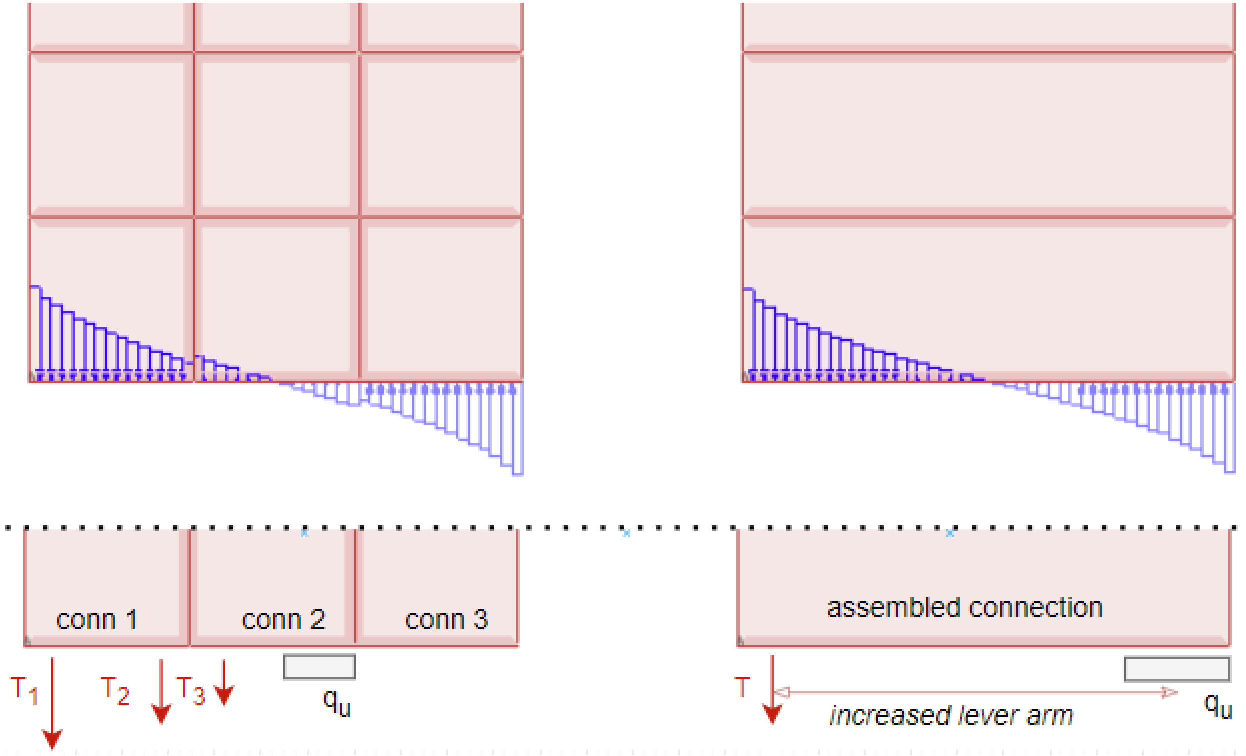
- Influence of connection stiffness on design
  - Independent vs. continuous units



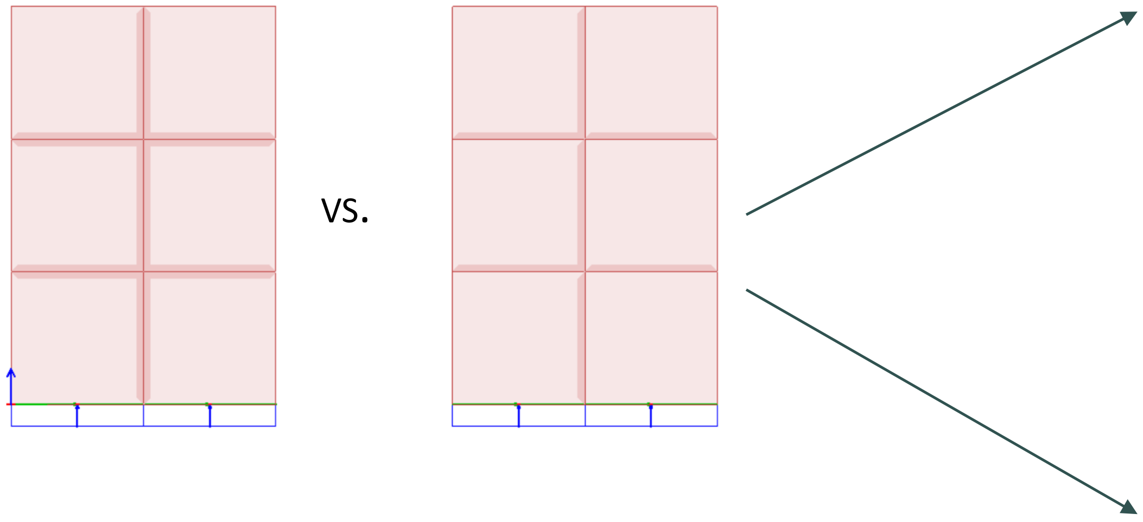


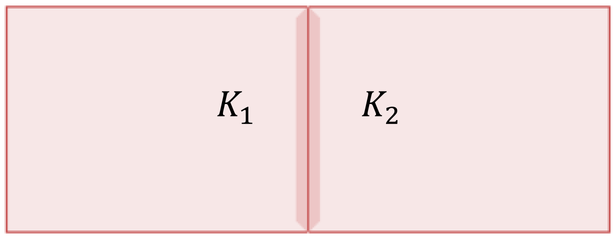
Design and modeling of precast concrete shear walls

- Influence of connection stiffness on design
  - Independent vs. continuous units:
    - Influence ULS design of horizontal connections
    - When vertical connections are relatively stiff, continuous units provide a more economic design

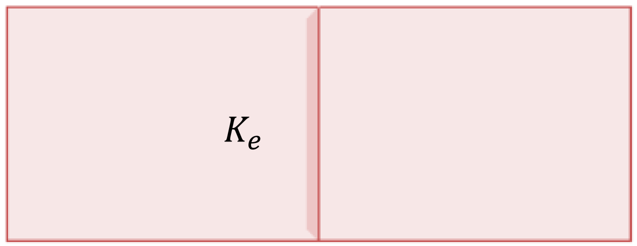


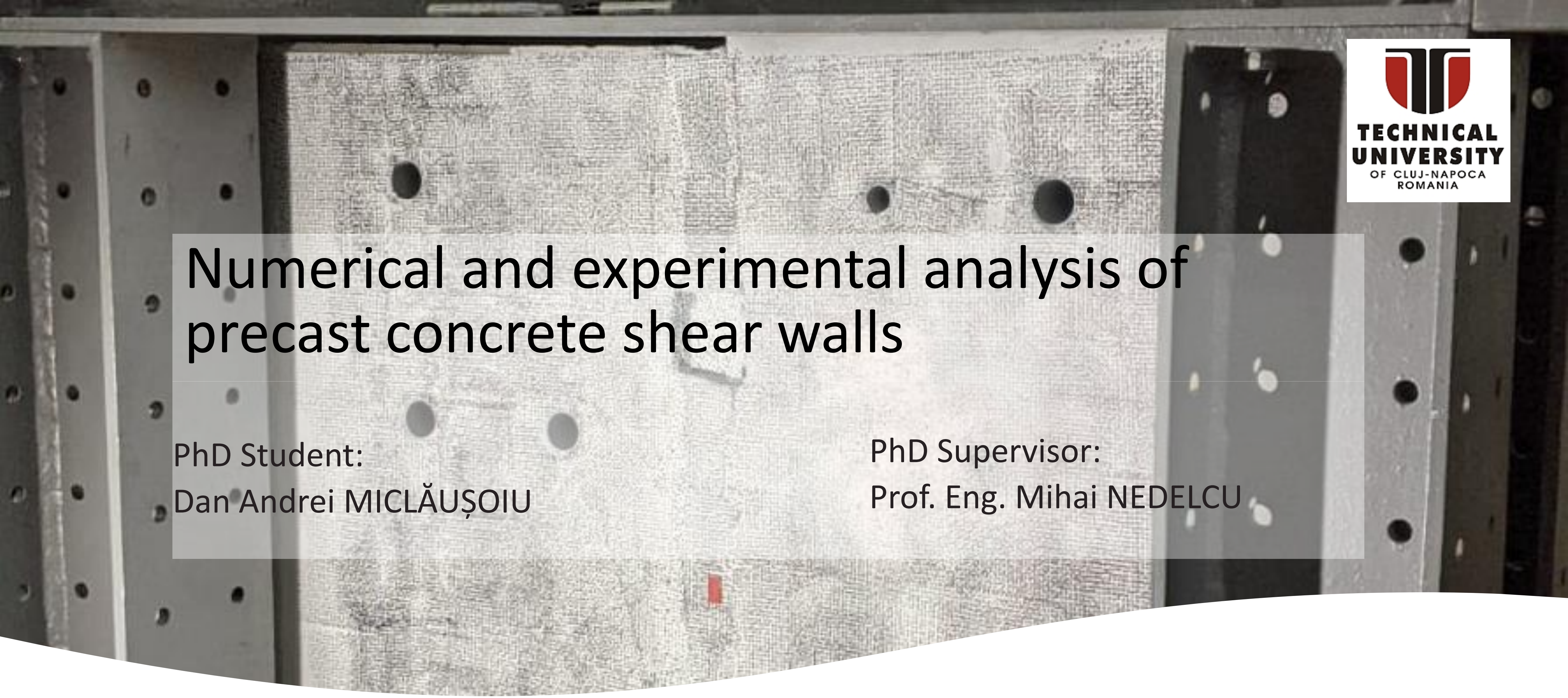
- Influence of connection stiffness on design
  - Positioning of connections





$$\frac{1}{K_e} = \frac{1}{K_1} + \frac{1}{K_2} \qquad if\ K_1 = K_2, \qquad K_e = \frac{K_1}{2}$$





Numerical and experimental analysis of precast concrete shear walls

PhD Student:
Dan Andrei MICLĂUȘOIU

PhD Supervisor:
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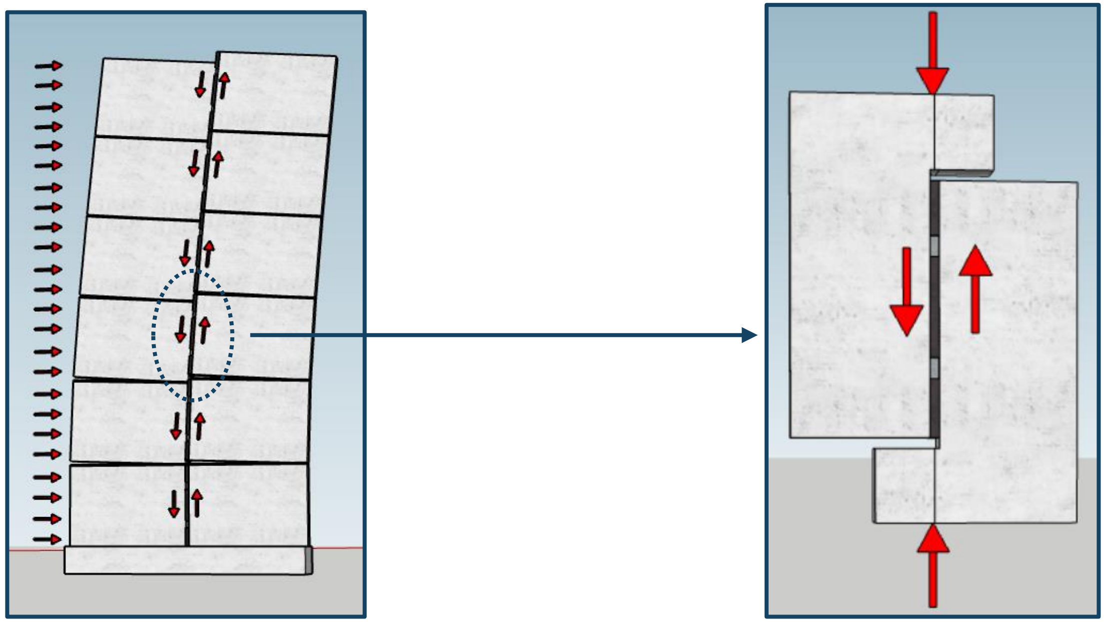
CONSOLIS

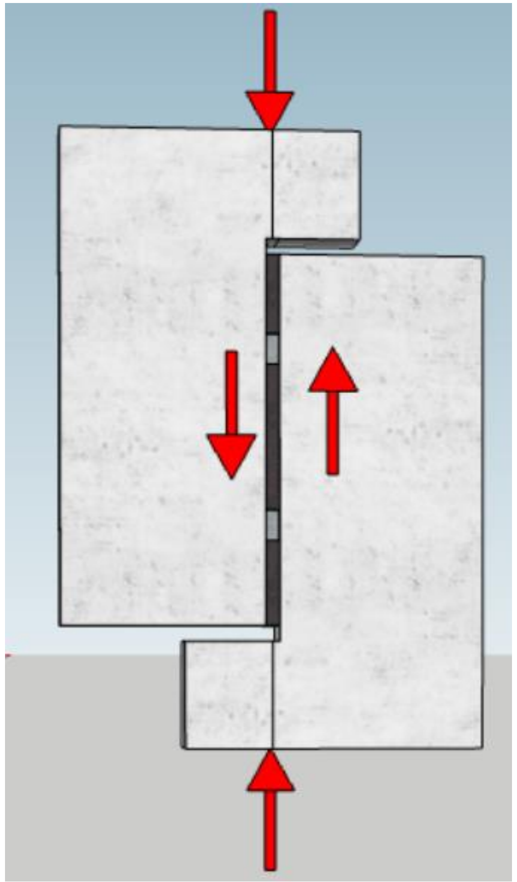
Design and modeling of precast concrete shear walls

- Precast concrete connections – Consolis experimental study
  - Objectives
    - Better understand behavior and capacity of vertical wall to wall connections
    - Assess the stiffness of the connection
      - Connections stiffness heavily impacts the distribution of internal forces throughout the shear wall assemblies and, consequently, in the whole structure
    - Verify the design methods for different connection types
      - EN 1992-1-1 capacity provided by rel. 6.25 vs experiment failure values

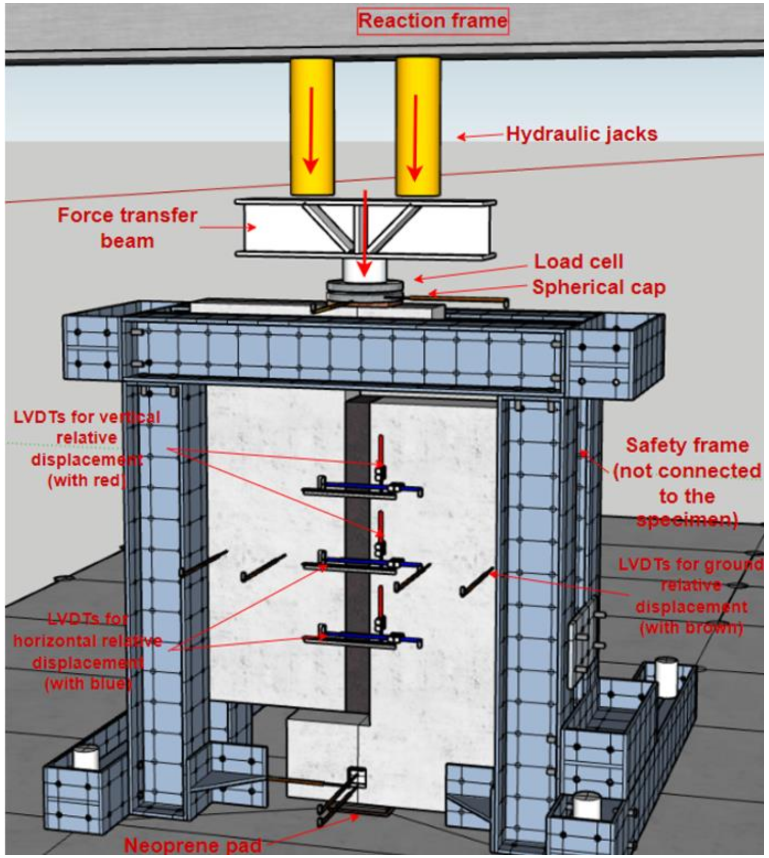


Design and modeling of precast concrete shear walls

- Precast concrete connections – Consolis experimental study



Concept

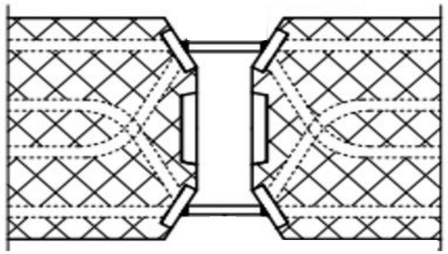


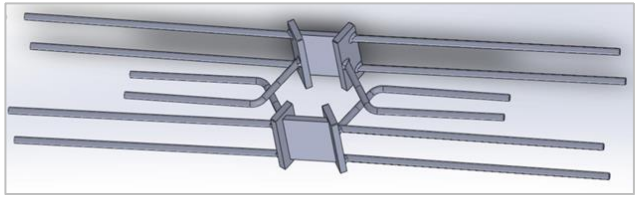
Planning

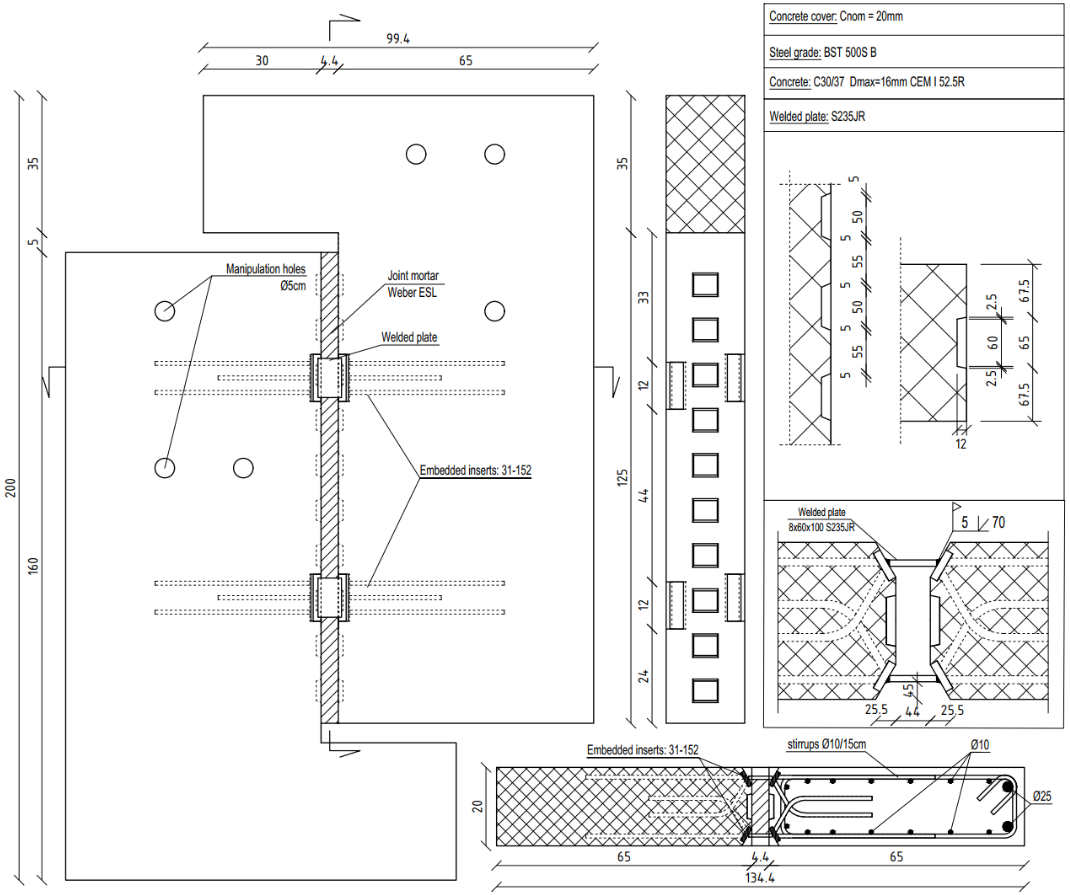
Reality

Design and modeling of precast concrete shear walls

- Precast concrete connections – Consolis experimental study
  - Welded plate connection with shear keys
    - 3 identical specimens have been tested
    - On-site welded required for the welded plates
    - Connection grouted after welding of the steel plates
    - Indentations/recesses on 1/3 of the thickness
    - Commonly steel inserts are placed only on one side of the connection

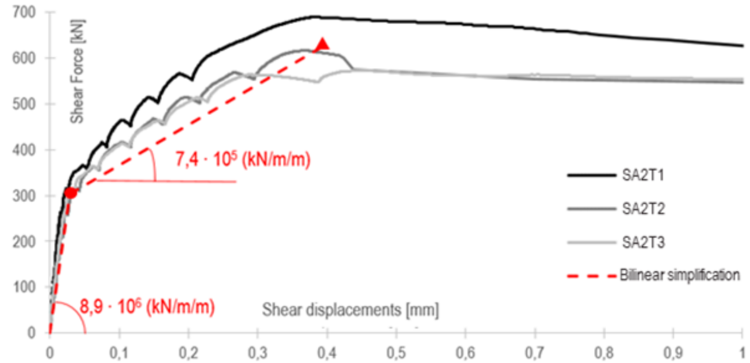




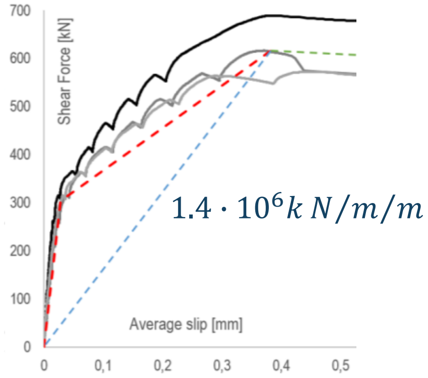


Design and modeling of precast concrete shear walls

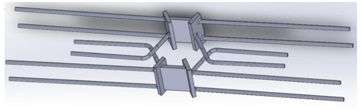
- Precast concrete connections – Consolis experimental study
  - Welded plate connection with shear keys
    - Equivalent monolithic behavior before cracking
    - More than 90% shear stiffness reduction after cracking

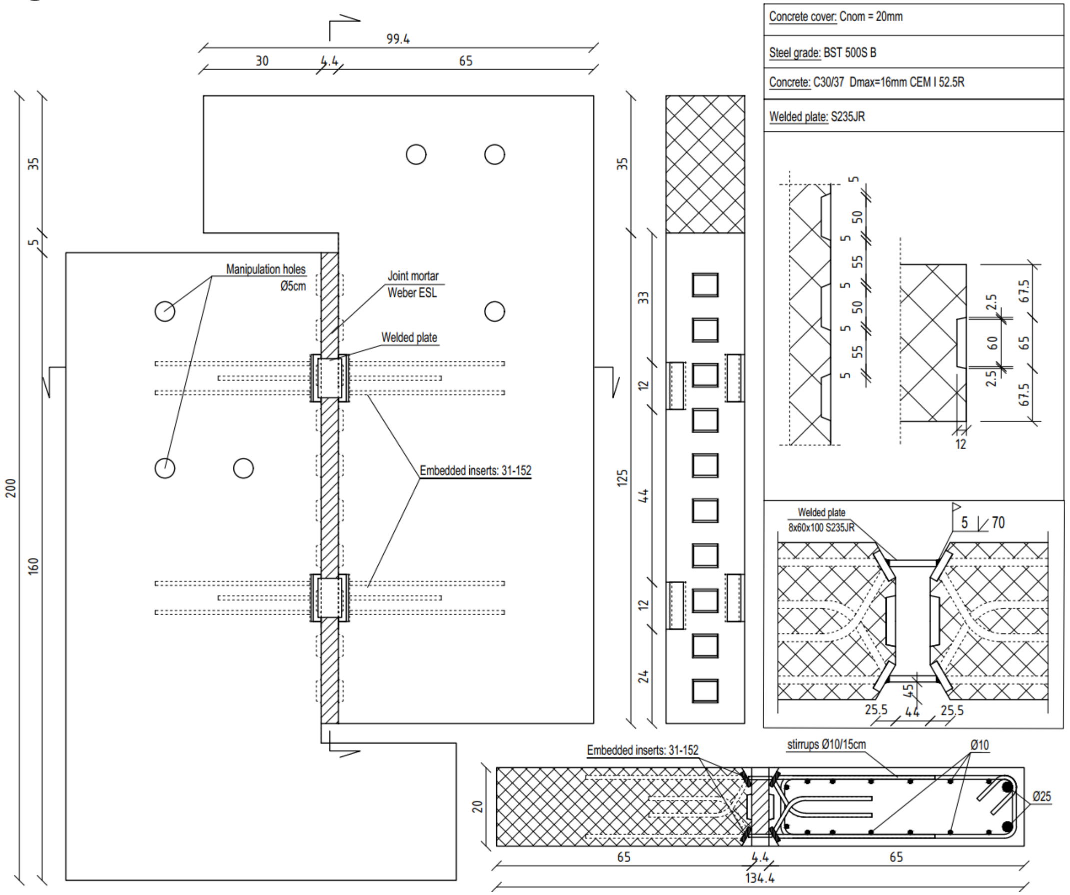


Shear force vs. shear displacement capacity curve



Proposed secant stiffness value for day-to-day design





Design and modeling of precast concrete shear walls

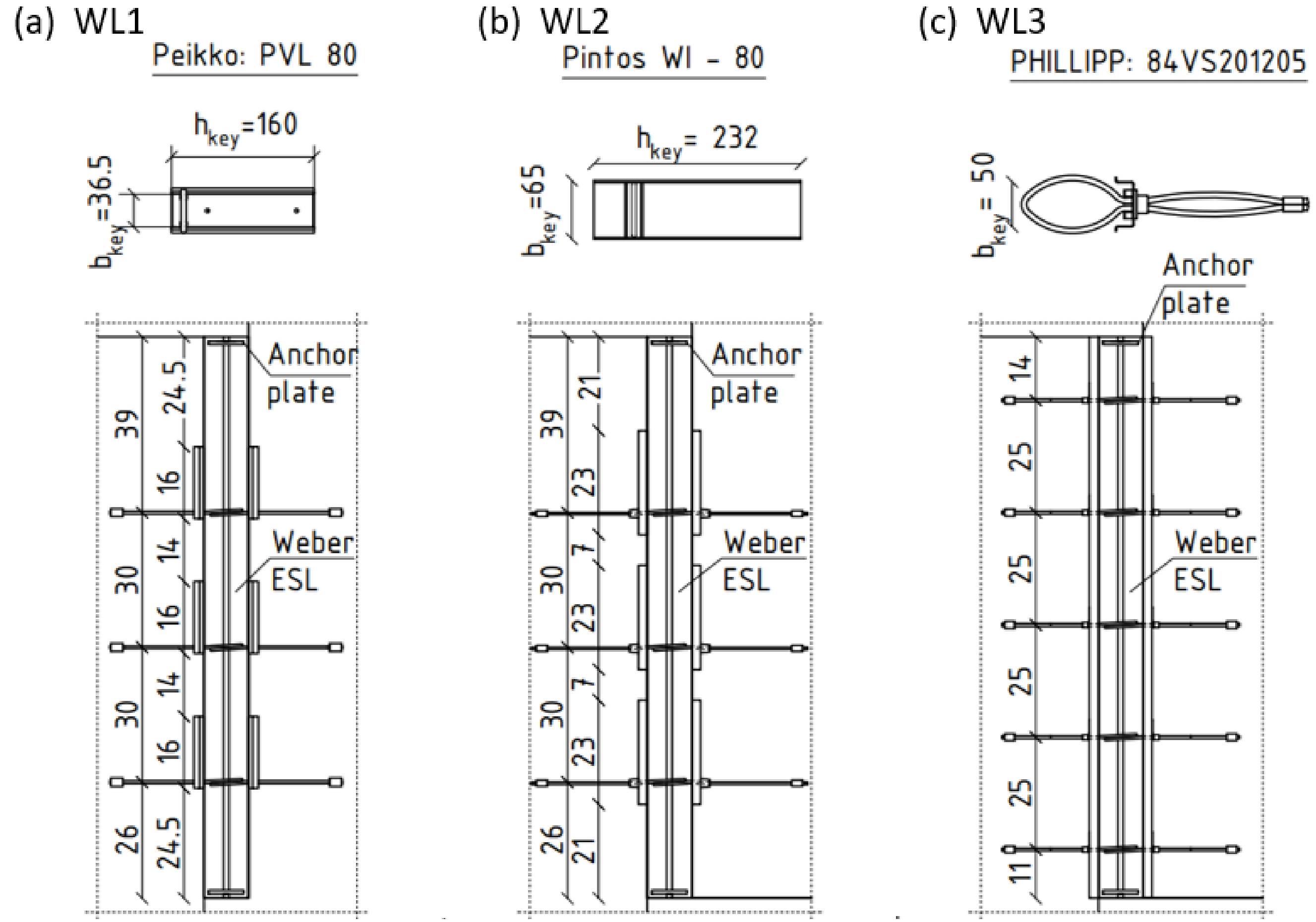
- Precast concrete connections – Consolis experimental study
  - Wire loop and wire rail connections
    - Philipp non-structural rail
    - Peikko PVL80
    - Pintos WI 80





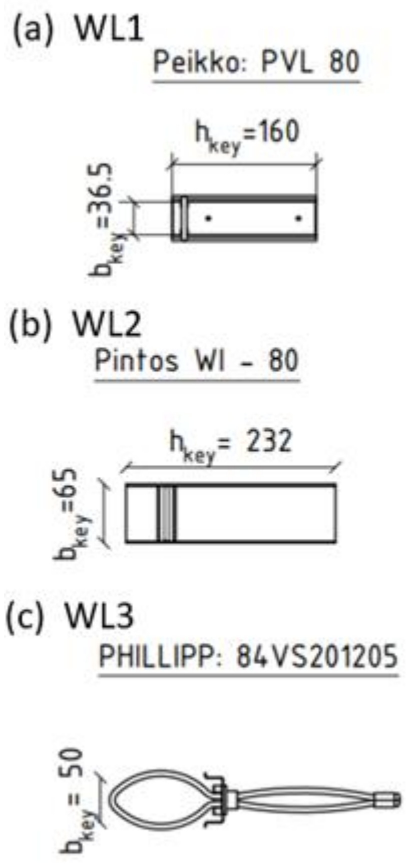

Design and modeling of precast concrete shear walls

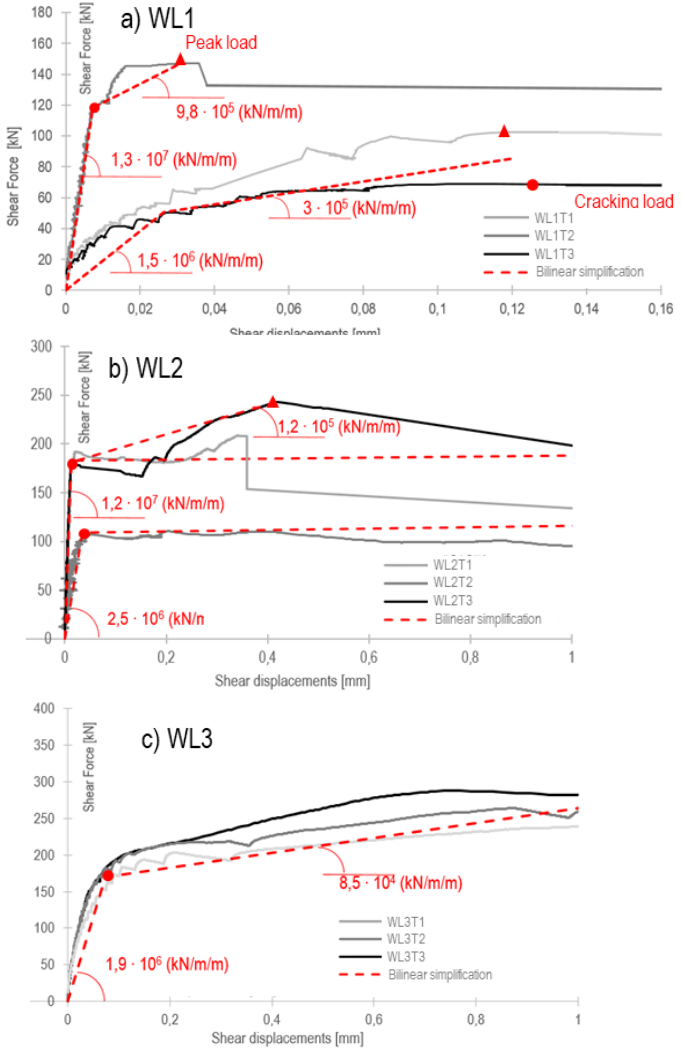
- Precast concrete connections – Consolis experimental study
  - Wire loop and wire rail connections
    - 3 specimens for each series
    - wire boxes installed in the wall panel casting process
    - reinforcement bar crosses the loops to ensure anchorage
    - shear keys formed by the boxes
    - force transfer achieved only after grout casting and curing
    - joint casting done with a dry mix non-sagging mortar widely used in Finland and the Baltic Countries

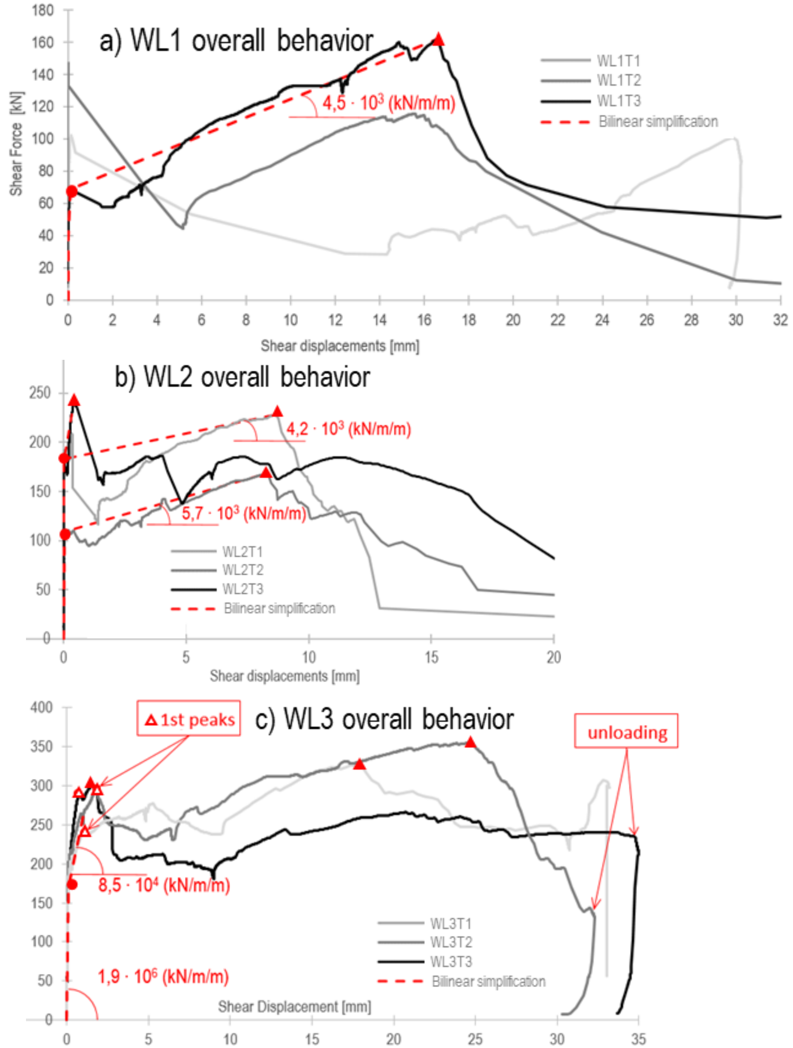


Design and modeling of precast concrete shear walls

- Precast concrete connections – Consolis experimental study
  - Wire loop and wire rail connections

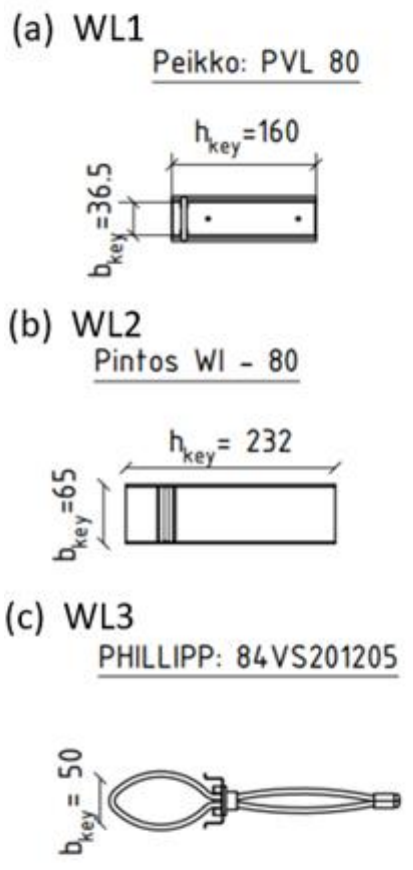


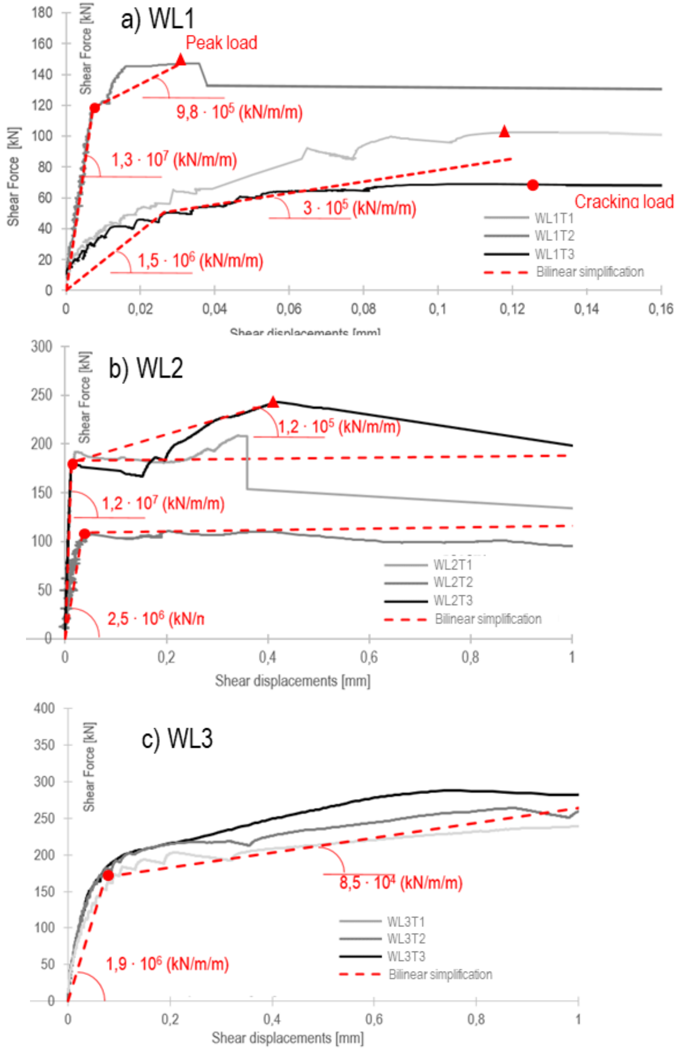




Design and modeling of precast concrete shear walls

- Precast concrete connections – Consolis experimental study
  - Wire loop and wire rail connections





- Pre-cracking:
  - WL1 & WL2:
    - spread of stiffness values
    - inconsistent behavior
  - WL3:
    - slippage leads to progressive cracking and loading of wires
    - consistent behavior
- Post-cracking:
  - The stress redistribution due cracking → force decrease and slippage increase
  - WL1:
    - 2 specimens: cracking load = peak load;
    - 1 specimen: peak load at very high shear slip
  - WL2: peak load at very high shear slip
  - WL3: very ductile behavior

Design and modeling of precast concrete shear walls

- Precast concrete connections – Consolis experimental study
  - Concluding remarks
    - Steel Assemblies with grouted shear keys:
      - two-stage behavior:
        - pre-cracking monolithic equivalent behavior;
        - post-cracking behavior is depending on the layout of the reinforcement
      - bolted connection requires further investigations: joint compaction voids observed; embedded headed anchor failure mode is unsatisfactory
      - EC2 rel. 6.25 → acceptable accuracy for welded connections
    - Wire-loops:
      - **Inconsistent behavior (cracking loads, stiffness values)**
      - Insufficient post-cracking stiffness or non-existing (post-cracking behavior is not compatible with the desired structural behavior)
      - **EC2 rel. 6.25 → safe results for the cracking load, neglecting wires contribution**
      - according EC2, wires should have the same degree of ductility as the reinforcement; Young modulus should be around 190GPa

Design and modeling of precast concrete shear walls

- Precast concrete connections – Consolis experimental study
  - More information:
    - PhD thesis (English, available soon):
      - https://rei.gov.ro/teze-doctorat
    - Experimental program report available online (English):
      - D. Miclausoiu, “Experimental analysis of vertical connections for precast shear walls. ISBN 978-606-737-634-0,” UTPRESS: https://biblioteca.utcluj.ro/files/carti-online-cu-coperta/634-0.pdf, Cluj-Napoca, 2023
    - Scientific articles:
      - D. Miclăușoiu, G.-Á. Sándor, H. Constantinescu, B. Hegheș and M. Nedelcu, “EXPERIMENTAL STUDY OF PRECAST WALL CONNECTION WITH GROUTED SHEAR KEYS AND WELDED PLATES,” in 6th fib International Congress, Oslo, 2022
      - D. Miclăușoiu, G.-Á. Sándor, H. Constantinescu, B. Hegheș and M. Nedelcu, “EXPERIMENTAL STUDY OF PRECAST WALL CONNECTION WITH HIGH STRENGTH WIRE LOOPS,” in 14th fib PhD Symposium in Civil Engineering, Rome, 2022
      - D. Miclăușoiu, M. Nedelcu, T. Blanksvärd, “Experimental and numerical analysis of different vertical connections of precast shear walls with special regard towards deformability”, Structural Concrete, DOI: 10.1002/SUCO.202300429

Evaluation of shear wall behavior

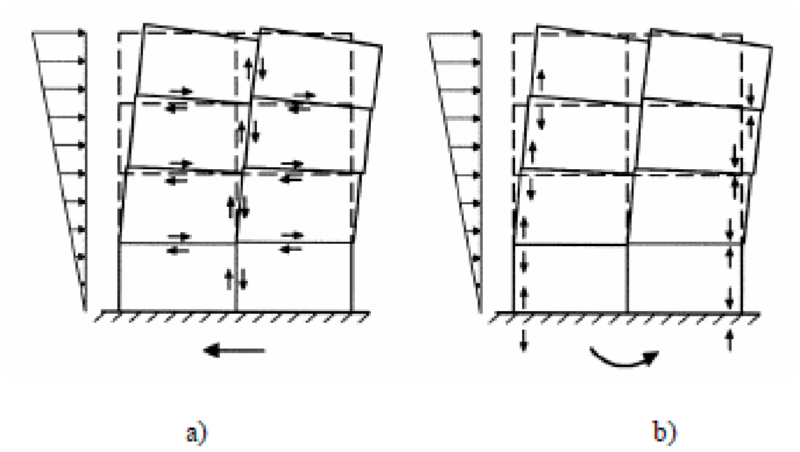
- Design and modeling of precast concrete shear walls:
  - Overview of wall panel connections
  - Element lay-outs with connections
  - Connection stiffnesses for FEM
- Evaluation of shear wall behavior:
  - Connection stiffnesses
  - Overall shear wall behavior
  - Simplified control calculations
  - Verification of bearing capacities
- Q&A session

Evaluation of shear wall behavior

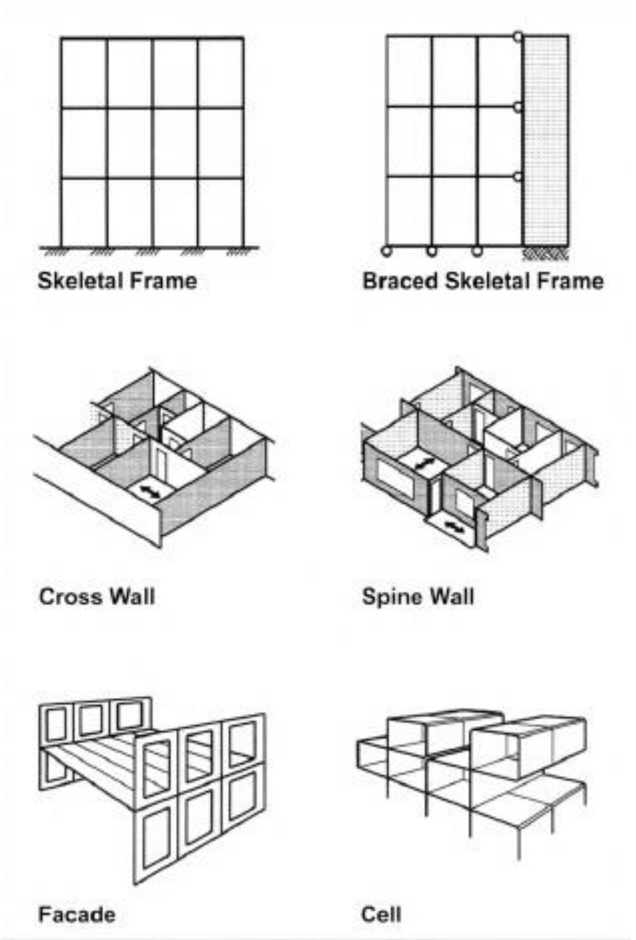
- Evaluation of shear wall behavior:
  - Wall design approaches
  - Wall connections design approaches
  - Design of walls for biaxial bending
  - Shear design of walls

Evaluation of shear wall behavior

- Shear wall behavior



In-plane action of prefabricated wall, a) shear forces, b) tensile and compressive forces (fib 43)

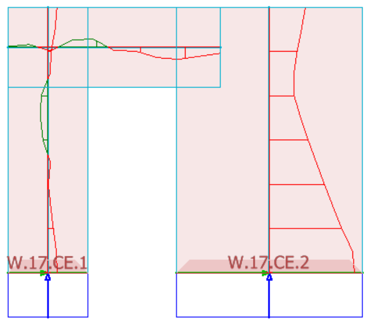


Precast concrete building systems (Structural Precast Concrete Handbook)

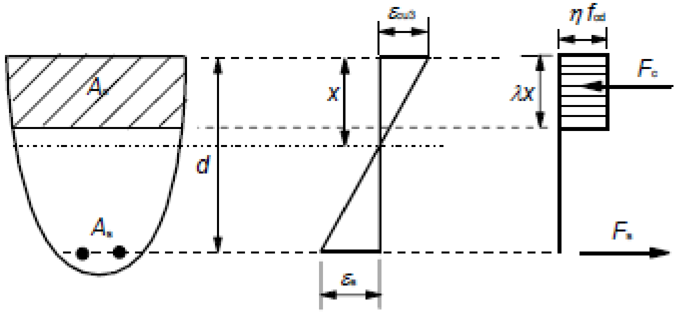
Evaluation of shear wall behavior

- Wall design approaches – biaxial bending
  - Surface reinforcement
  - Concealed rebar
  - Nonlinear design methods

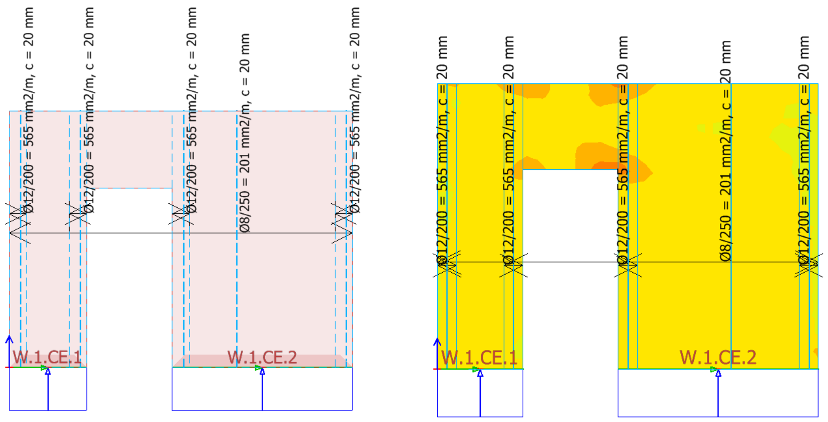
  - “Classical” ULS approach :
    - linear strain – plastic stress distribution
    - Linear elastic model for structural response
    - Plastic element/section design



Concealed rebar design approach



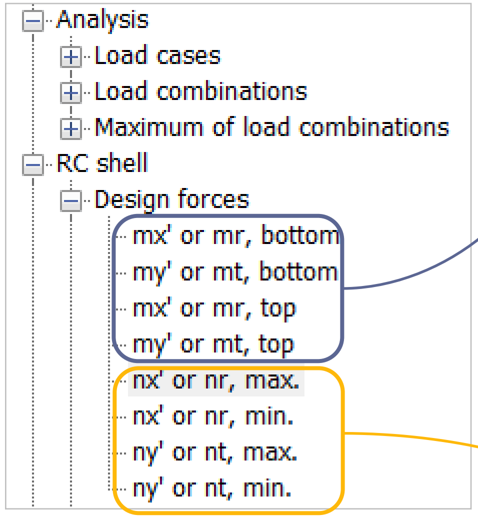
Rectangular stress distribution according to Figure 3.5 of EN 1992-1-1

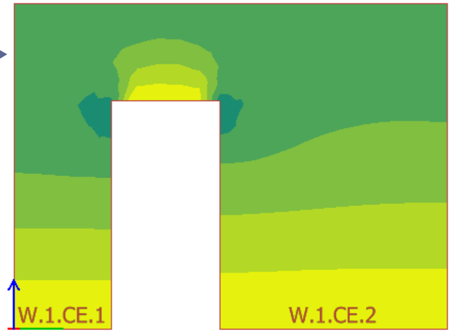


Surface reinforcement layout and check

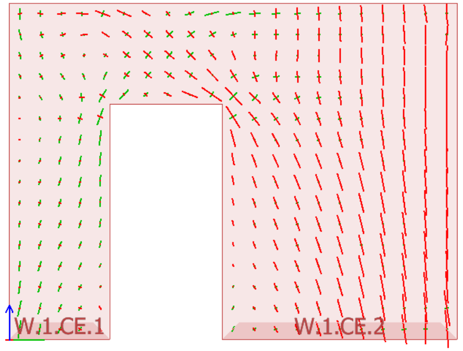
Evaluation of shear wall behavior

- Wall design approaches – biaxial bending
  - Surface reinforcement

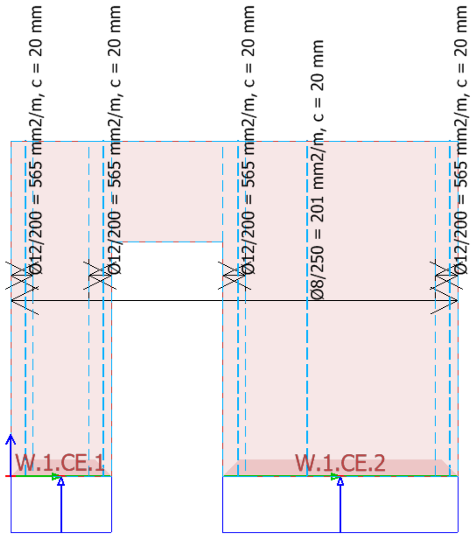


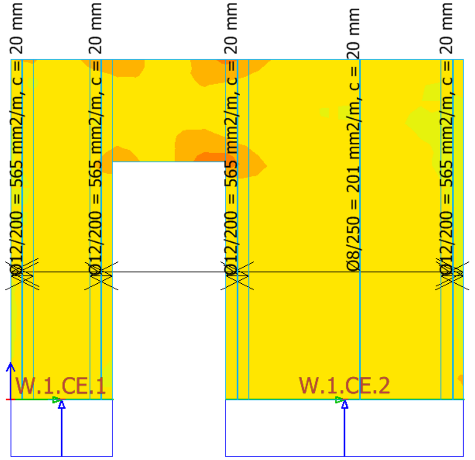


Internal forces due to out of plane loads



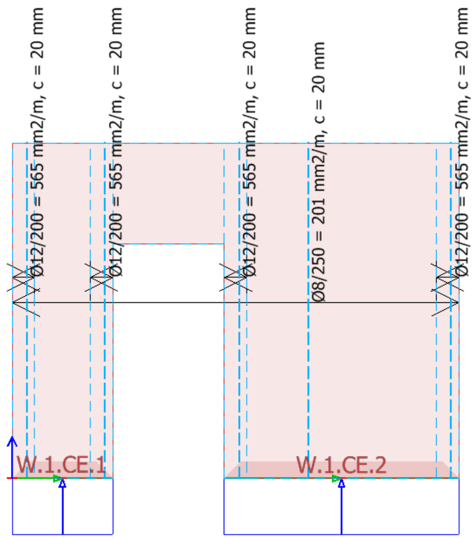
Internal forces due to in plane loads

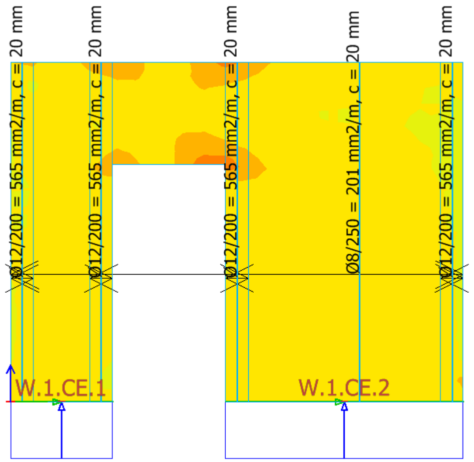




Evaluation of shear wall behavior

- Wall design approaches – biaxial bending
  - Surface reinforcement

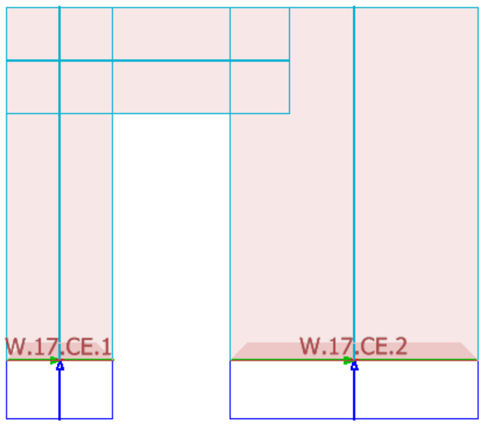


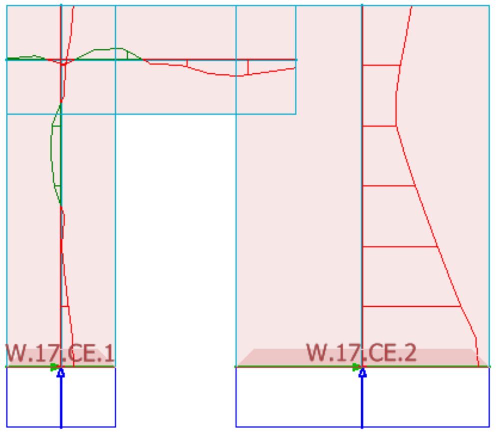


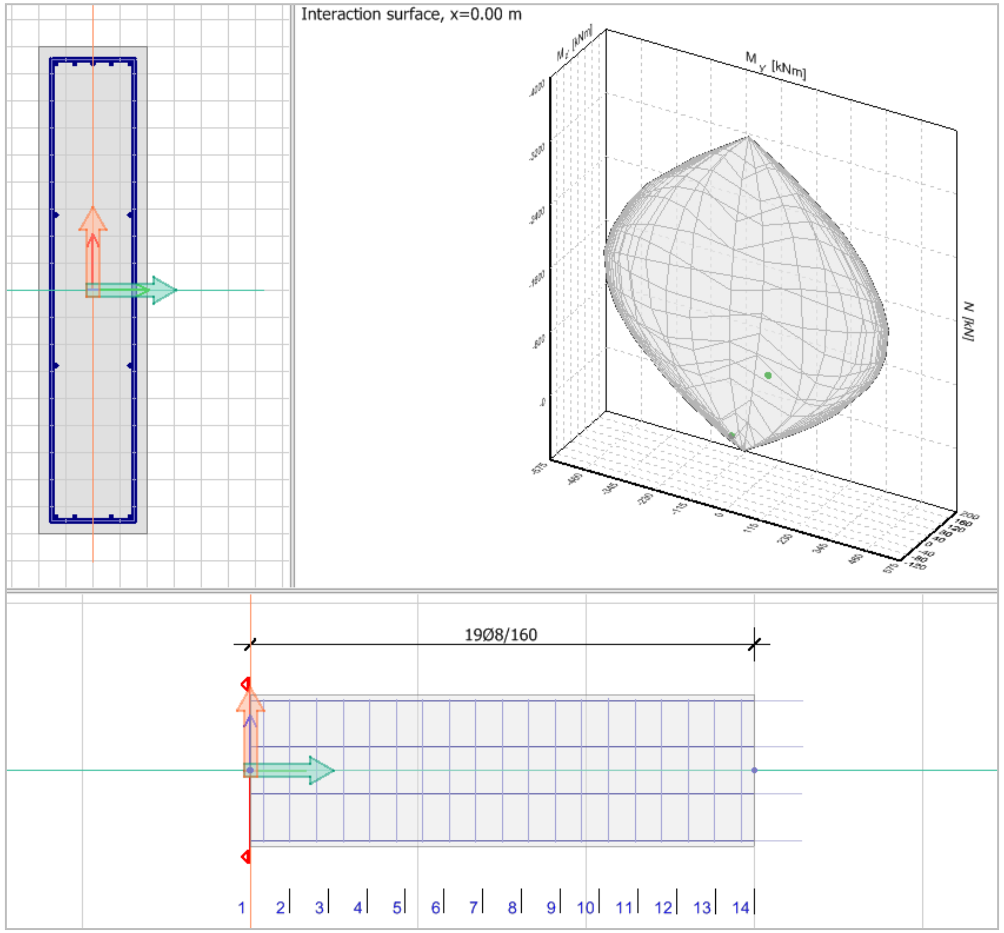
- Pros:
  - Any type of wall/slab can be designed regardless of configuration
- Cons:
  - Slow (needs additional preparations)
  - Based on elastic distribution of internal forces
  - Yields higher amount of reinforcement than the classical approach

Evaluation of shear wall behavior

- Wall design approaches – biaxial bending
  - Concealed rebar

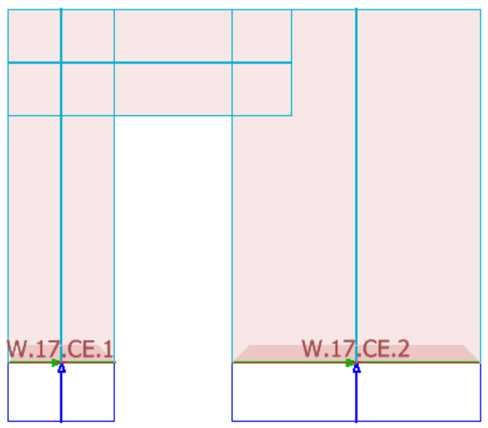


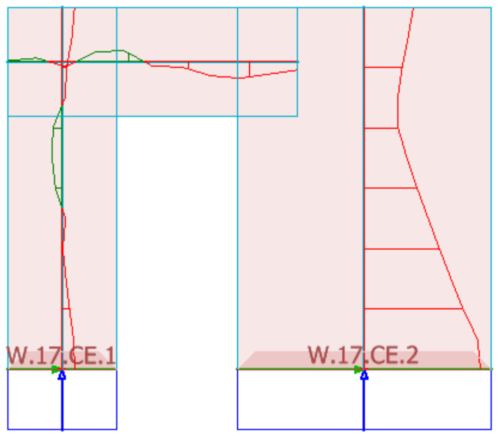




Evaluation of shear wall behavior

- Wall design approaches – biaxial bending
  - Concealed rebar



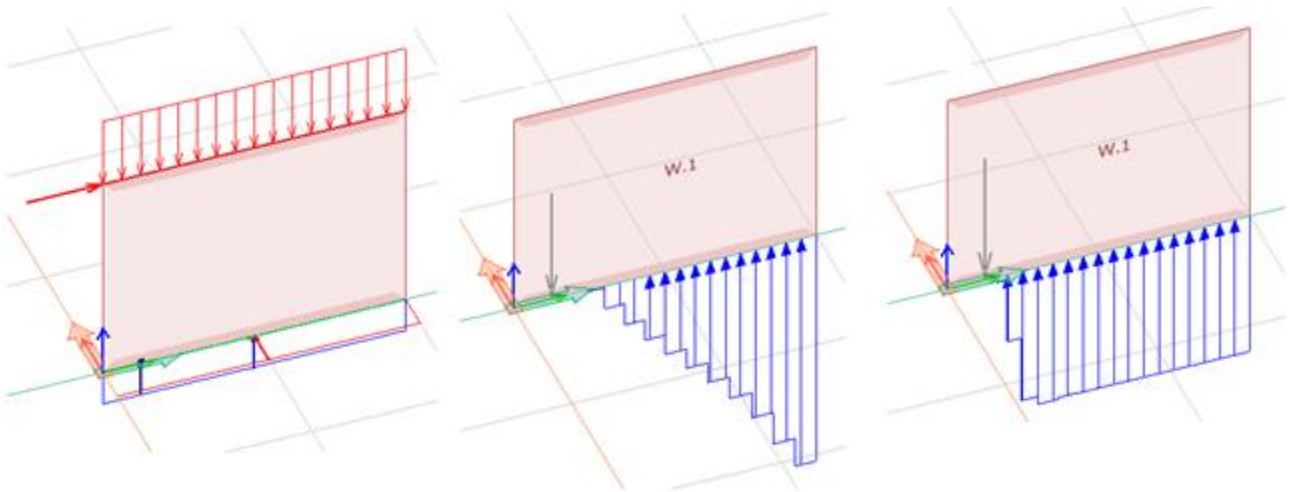


- Pros:
  - Quick
  - Design based on EC provisions for beam/column design

- Cons:
  - Applicable only for limited situations (e.g . facades with big openings)
  - Basic supposition: wall elements behave as columns

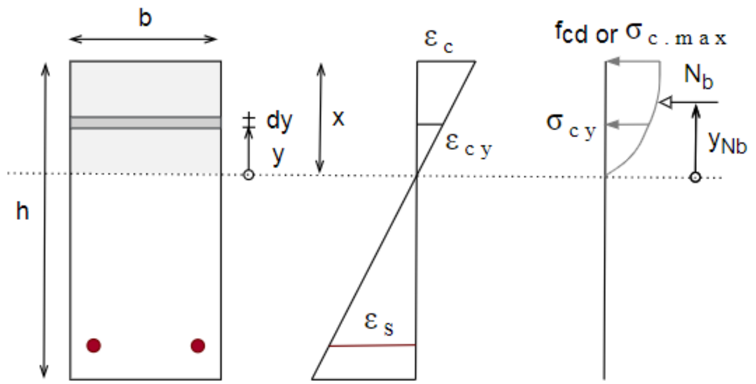
Evaluation of shear wall behavior

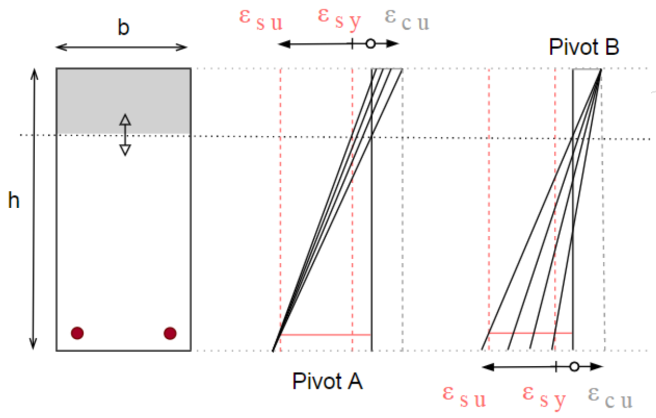
- Connection design approaches – vertical stress distribution
  - Linear elastic
  - Nonlinear with tension/compression only elements/interfaces
  - Nonlinear with plastic limit force
    - Previous approach + plastic limit

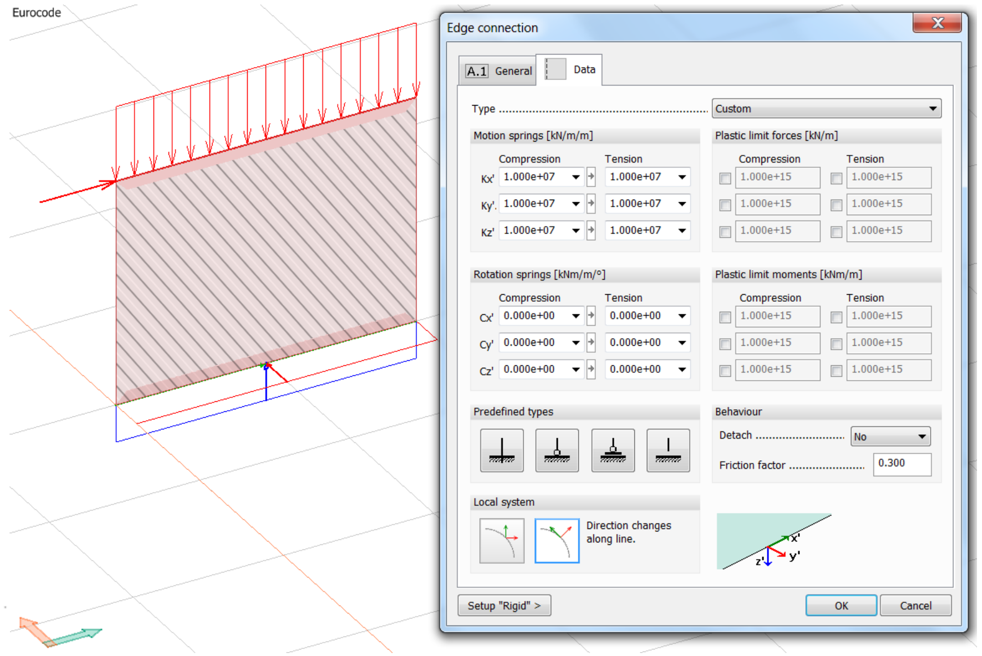


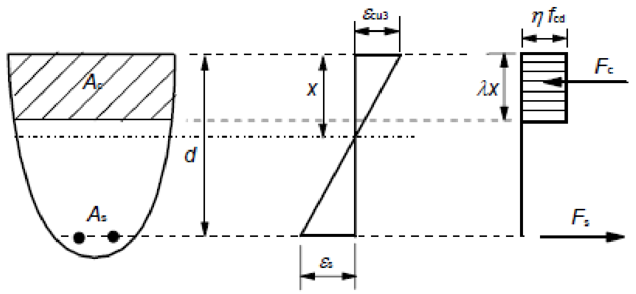
Evaluation of shear wall behavior

- Connection design approaches
  - Linear elastic
    - The common approach in design
    - ULS design of section
      - General design method
      - Simplified design procedures



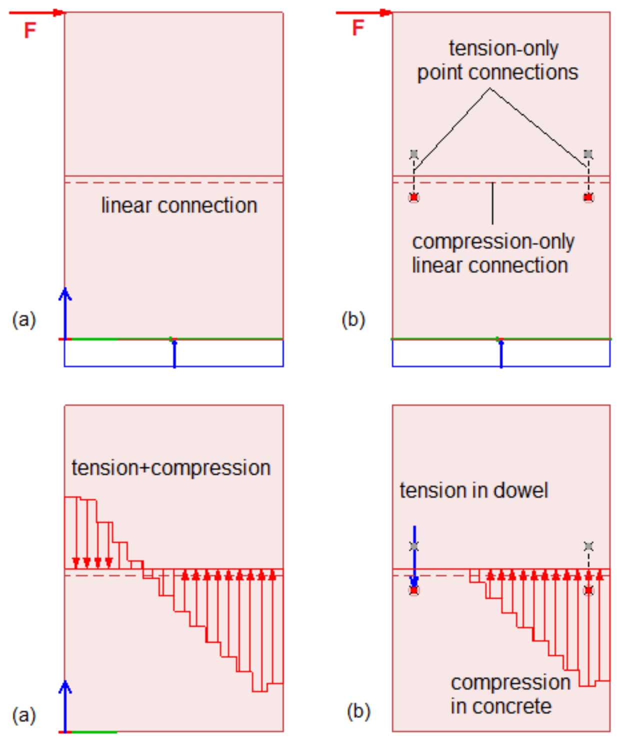






Evaluation of shear wall behavior

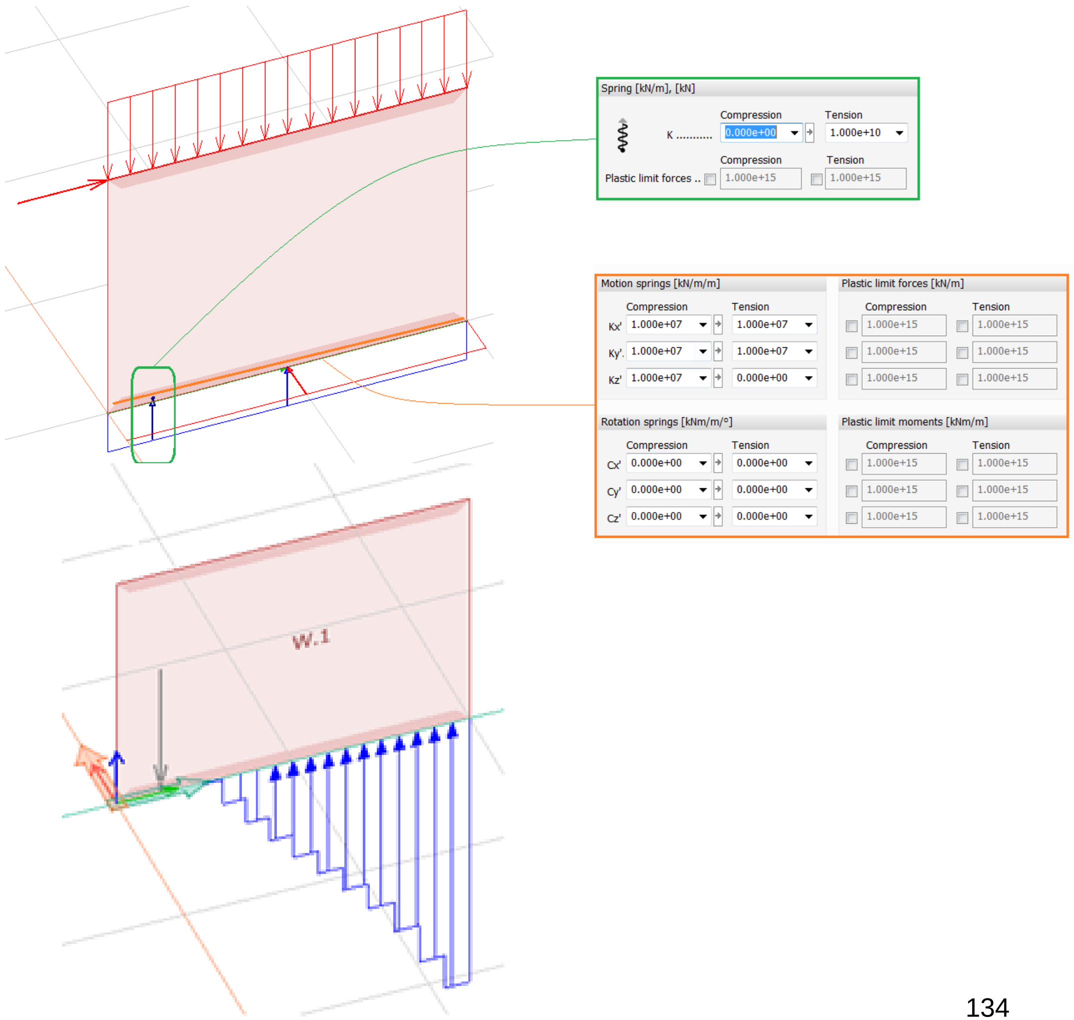
- Connection design approaches
  - Nonlinear with tension/compression only elements/interfaces
    - Concrete interface that accommodates only compression
    - Vertical tie that accounts for tension only



Evaluation of shear wall behavior

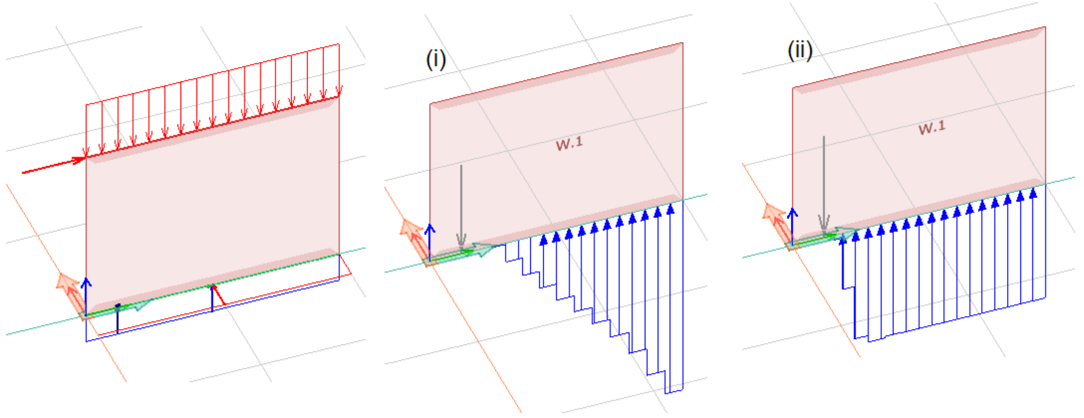
- Connection design approaches
  - Nonlinear with tension/compression only elements/interfaces
    - Concrete interface that accommodates only compression
    - Vertical tie that accounts for tension only

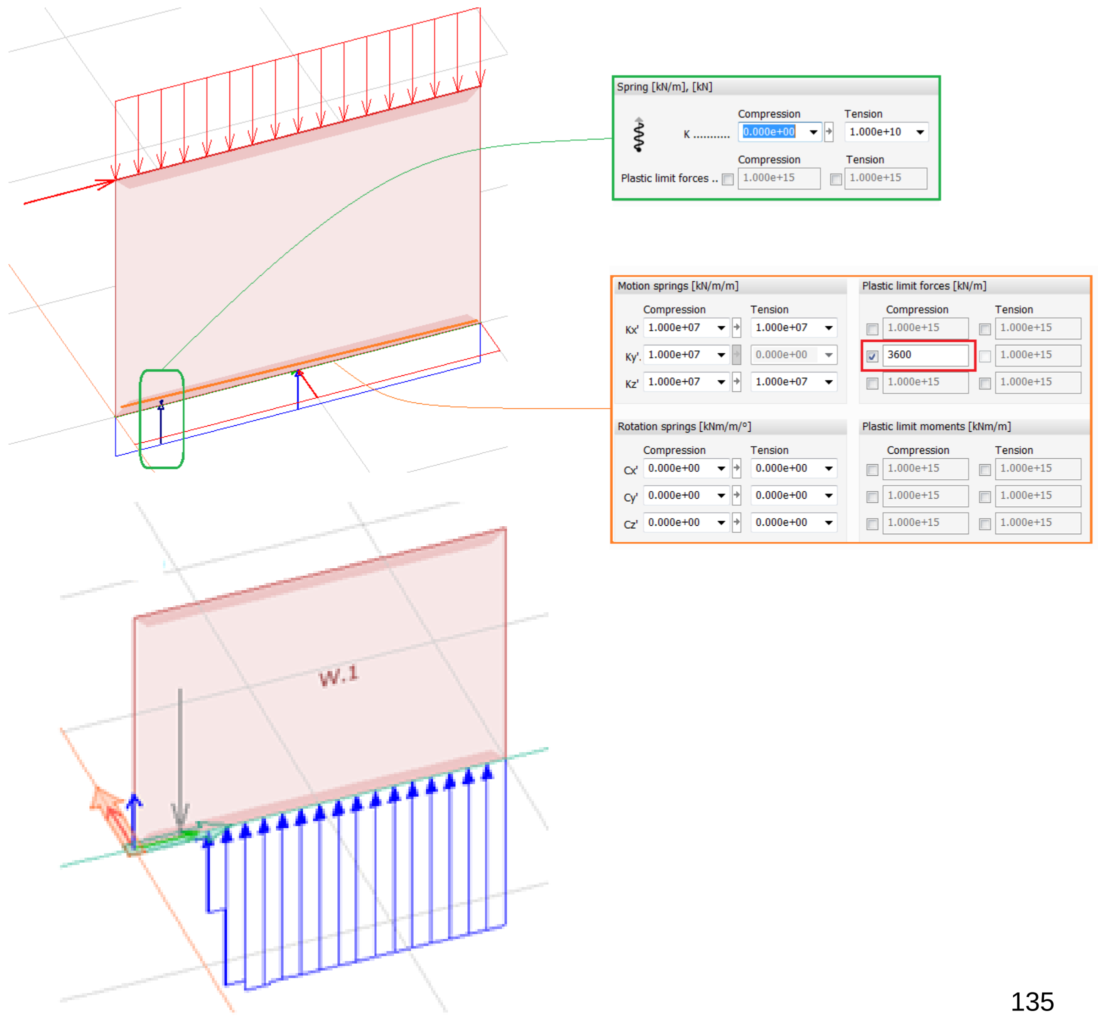
    - Limitations:
      - No limit for the capacity of the compressed concrete
      - No evaluation of deformation capacity is provided



Evaluation of shear wall behavior

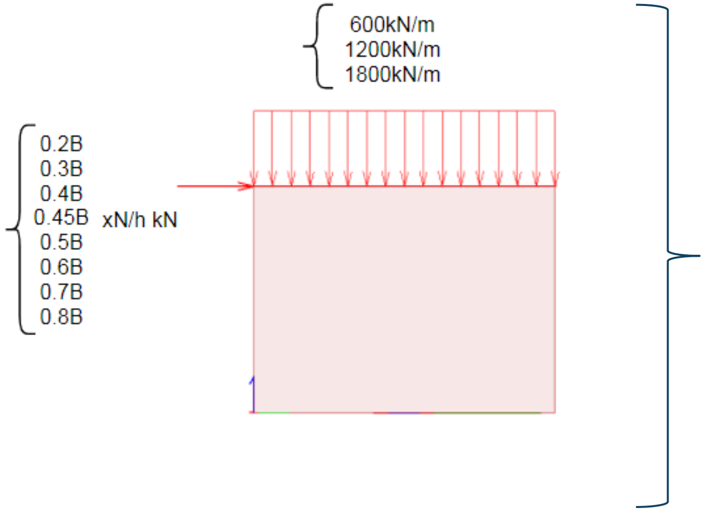
- Connection design approaches
  - Nonlinear with plastic limit force
    - In addition to previous nonlinear model:
      - A plastic limit for the compressive forces is imposed
    - Limitations:
      - Further increases the time needed for analysis
      - No evaluation of deformation capacity is provided



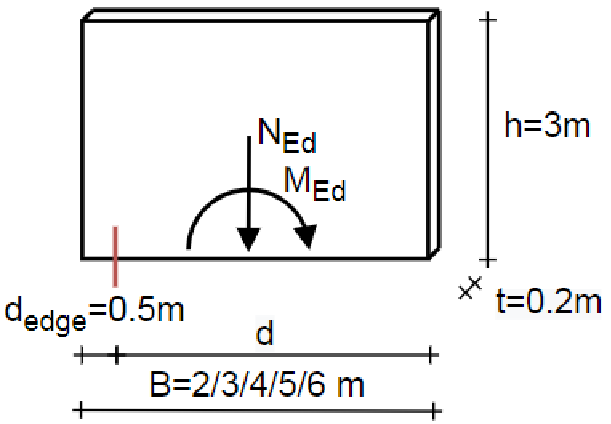


Evaluation of shear wall behavior

- Connection design approaches – vertical stress distribution
  - Comparative study
    - Five wall geometries:
      - length of the walls varies between 2 and 6m
      - vertical tie positioned at 0.5m from edge
    - Materials:
      - C30/37
      - S500



⇒ **120 cases**



- Loads:
  - Three gravitational loads cases:
    - 600kN/m (0.167xNcap)
    - 1200kN/m (0.334xNcap)
    - 1800kN/m (0.501xNcap)

✕

- 8 lateral load cases

$$F = (0.2\ to\ 0.8) * B * N/h$$

Evaluation of shear wall behavior

- Connection design approaches – vertical stress distribution
  - Comparative study
    - Four design approaches:
      - General design method (BENCHMARK)
      - ULS “classical” design approach
        a. *Design based on manual calculation – simplified design procedure (Procedure a)*
        b. *Automatic design – simplified design procedure (Procedure b)*
      - Nonlinear with tension/compression only elements/interfaces
        c. *Nonlinear analysis in FEM-Design (Procedure c)*
      - Nonlinear with plastic limit force
        d. *Nonlinear elastic-plastic analysis in FEM-Design (Procedure d)*

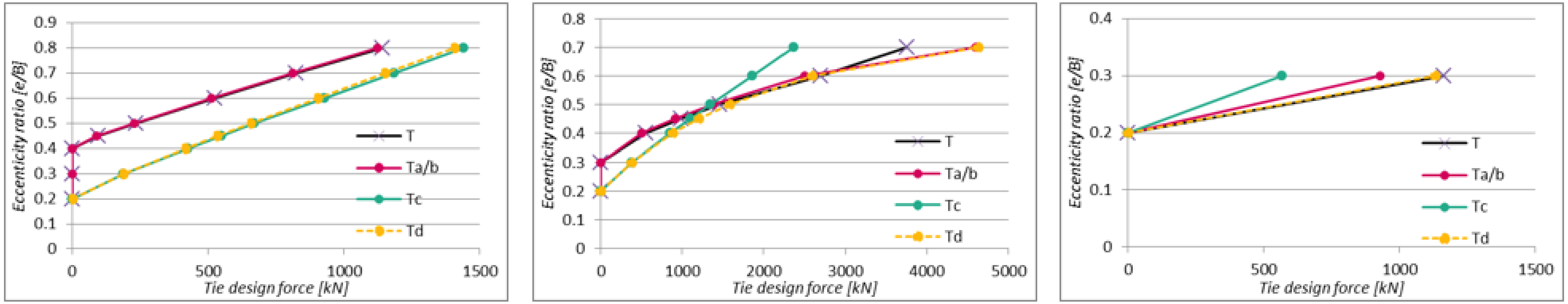
Applied to each 120 cases

Evaluation of shear wall behavior

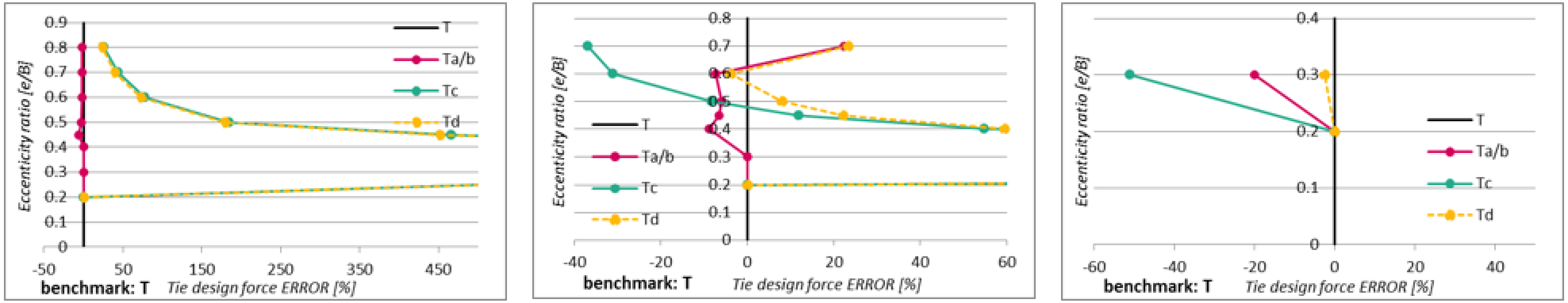
Wall	gravit. load	ecc. ratio	lat. load [kN]	overturning?	qu exceeded?	Mu exceeded?	Ta [kN]	Tb [kN]	err. Ta/b [%]	Tc [kN]	err. Tc [%]	Td [kN]	err. Td [%]	T [kN]
3mx0.2mx3m	600kN/m	0.2	360	NO	NO	NO	0	0	0	0	0	0	0	0
		0.3	540	NO	NO	NO	0	0	0	189	1000	188	1000	0
		0.4	720	NO	NO	NO	0	0	0	427	1000	417	1000	0
		0.45	810	NO	YES	NO	91	91	-6	549	466	535	452	97
		0.5	900	YES	NA	YES	229	229	-3	672	185	659	179	236
		0.6	1080	YES	NA	YES	513	513	-2	929	77	905	73	524
		0.7	1260	YES	NA	YES	811	811	-2	1185	43	1152	39	826
		0.8	1440	YES	NA	YES	1123	1123	-2	1441	26	1409	23	1142
	1200kN/m	0.2	720	NO	NO	NO	0	0	0	0	0	0	0	0
		0.3	1080	NO	NO	NO	0	0	0	379	1000	376	1000	0
		0.4	1440	NO	YES	YES	503	503	-9	854	55	880	59	552
		0.45	1620	NO	YES	YES	918	918	-7	1099	12	1199	22	982
		0.5	1800	YES	NA	YES	1375	1375	-6	1343	-8	1580	8	1463
		0.6	2160	YES	NA	YES	2498	2498	-7	1857	-31	2592	-4	2692
		0.7	2520	YES	NA	YES	4595	4595	22	2369	-37	4633	23	3758
		0.8	2880	---	---	---	---	---	---	---	---	---	---	---
	1800kN/m	0.2	1080	NO	NO	NO	0	0	0	0	0	0	0	0
		0.3	1620	NO	YES	YES	930	930	-20	568	-51	1134	-2	1163
		0.4	2160	---	---	---	---	---	---	---	---	---	---	---
		0.45	2430	---	---	---	---	---	---	---	---	---	---	---
		0.5	2700	---	---	---	---	---	---	---	---	---	---	---
		0.6	3240	---	---	---	---	---	---	---	---	---	---	---
		0.7	3780	---	---	---	---	---	---	---	---	---	---	---
		0.8	4320	---	---	---	---	---	---	---	---	---	---	---

3mx0.2mx3m wall: tie design forces

Evaluation of shear wall behavior



3mx0.2mx3m wall comparative charts – tie design forces: (i) 600kN/m line load; (ii) 1200kN/m line load; (iii) 1800kN/m line load



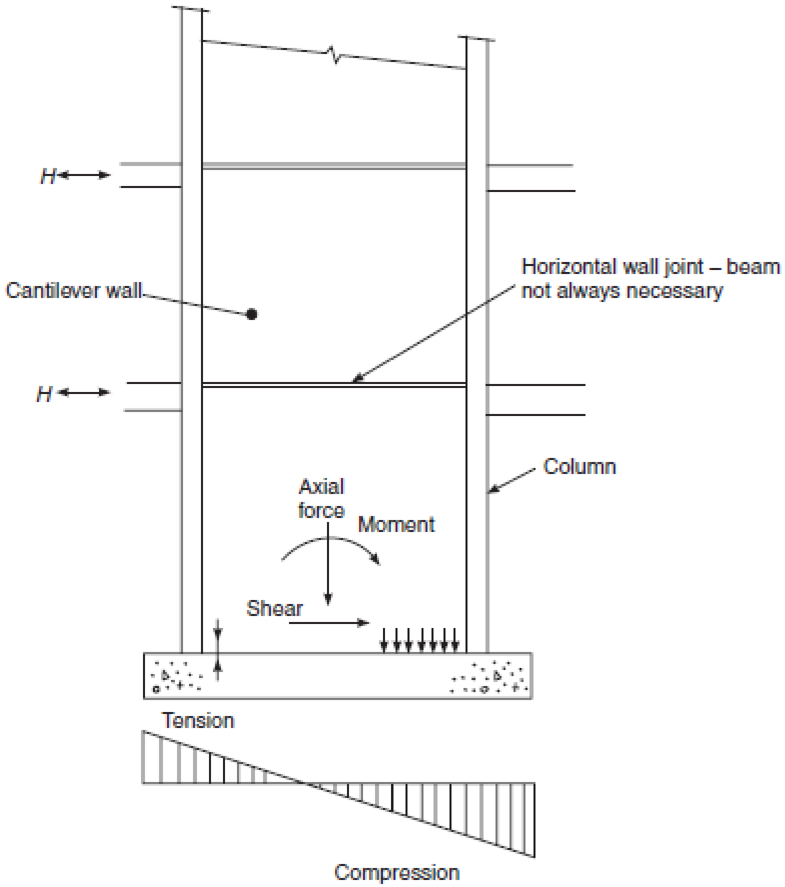
3mx0.2mx3m wall comparative charts – tie design force estimate errors: (i) 600kN/m line load; (ii) 1200kN/m line load; (iii) 1800kN/m line load

Evaluation of shear wall behavior

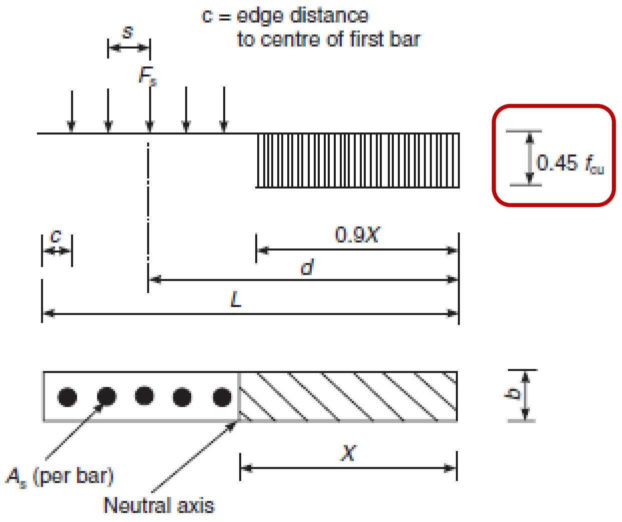
- Connection design approaches – vertical stress distribution
  - Comparative study conclusions:
    - ULS design approaches (a and b)provide the same results and the best estimate of structural behavior
    - Nonlinear design (c) overestimates tie design forces in the elastic range and, as plasticity starts to govern the behavior of the section, grossly underestimates the tie design forces
    - Nonlinear design with plastic limit force (d) overestimates tie design forces in the elastic range but, as plasticity spreads, provides very good estimates of tie design forces. This procedure is very demanding in terms of analysis time and effort to prepare the structural model
    - **ULS design approaches are the best option for day-to-day design and require the minimum amount of input and analysis time.**

Evaluation of shear wall behavior

- Bending with axial force – design principles for walls



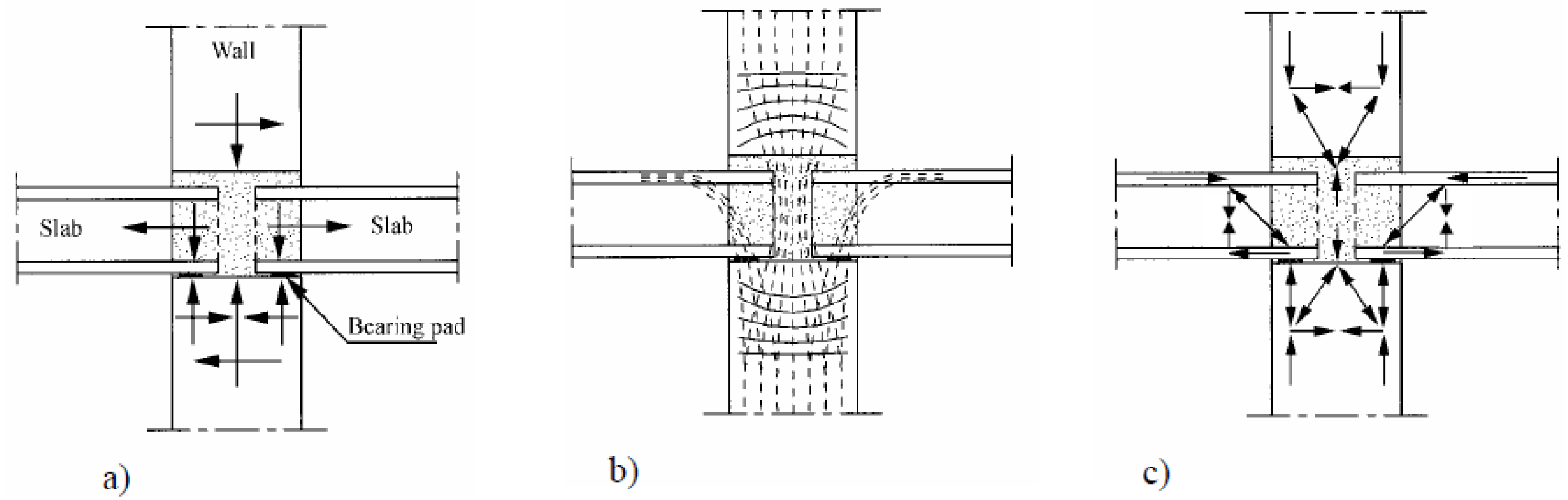
Analysis of precast cantilever walls
(Precast concrete structures – K.S. Elliott)



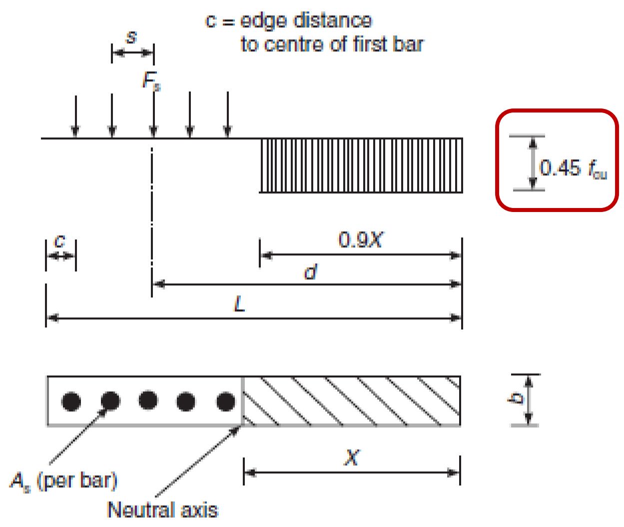
Design principles for cantilever walls
(Precast concrete structures – K.S. Elliott)

Evaluation of shear wall behavior

- Bending with axial force – design principles for walls



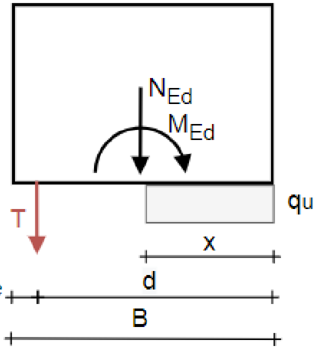
Slab-wall connection, a) forces, b) simplified stress analysis, c) strut-and-tie model (fib 43)



Design principles for cantilever walls (Precast concrete structures – K.S. Elliott)

- Bending with axial force– design principles for walls
  - Design approach – mathematical model

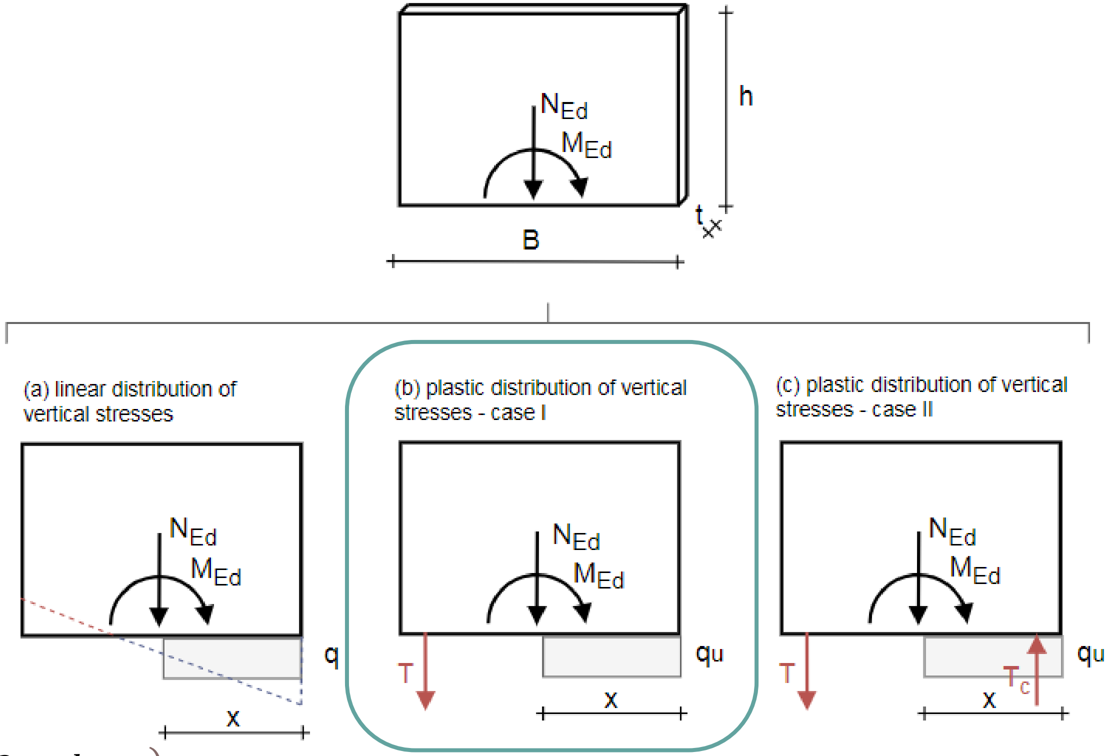
Starting point supposition: the section is subjected to plastic distribution of vertical stresses – case I



$$q_u = percent \cdot f_{cd} \cdot t^*$$

$$\begin{cases} T + N_{Ed} - q_u \cdot x = 0 \\ M_{Ed} + N_{Ed} \cdot e - q_u \cdot x \cdot \left(d - \frac{x}{2}\right) = 0 \end{cases}$$

$$T = q_u \cdot d \cdot \left(1 - \sqrt{1 - 2 \cdot \frac{M_{Ed} + N_{Ed} \cdot (B/2 - d_{edge})}{q_u \cdot d^2}}\right) - N_{Ed}$$



Supposition confirmed if:

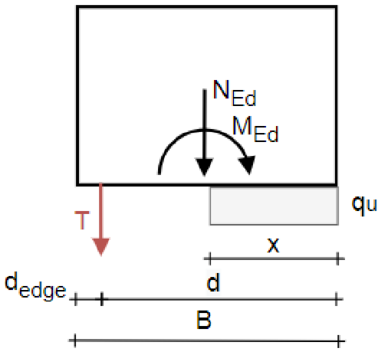
$$\begin{cases} 1 - 2 \cdot \frac{M_{Ed} + N_{Ed} \cdot (B/2 - d_{edge})}{q_u \cdot d^2} \geq 0 \\ x > 0 \\ T > 0 \end{cases}$$

(b) Plastic distribution of vertical stresses – case I

Evaluation of shear wall behavior

- Bending with axial force– design principles for walls
  - Design approach – mathematical model

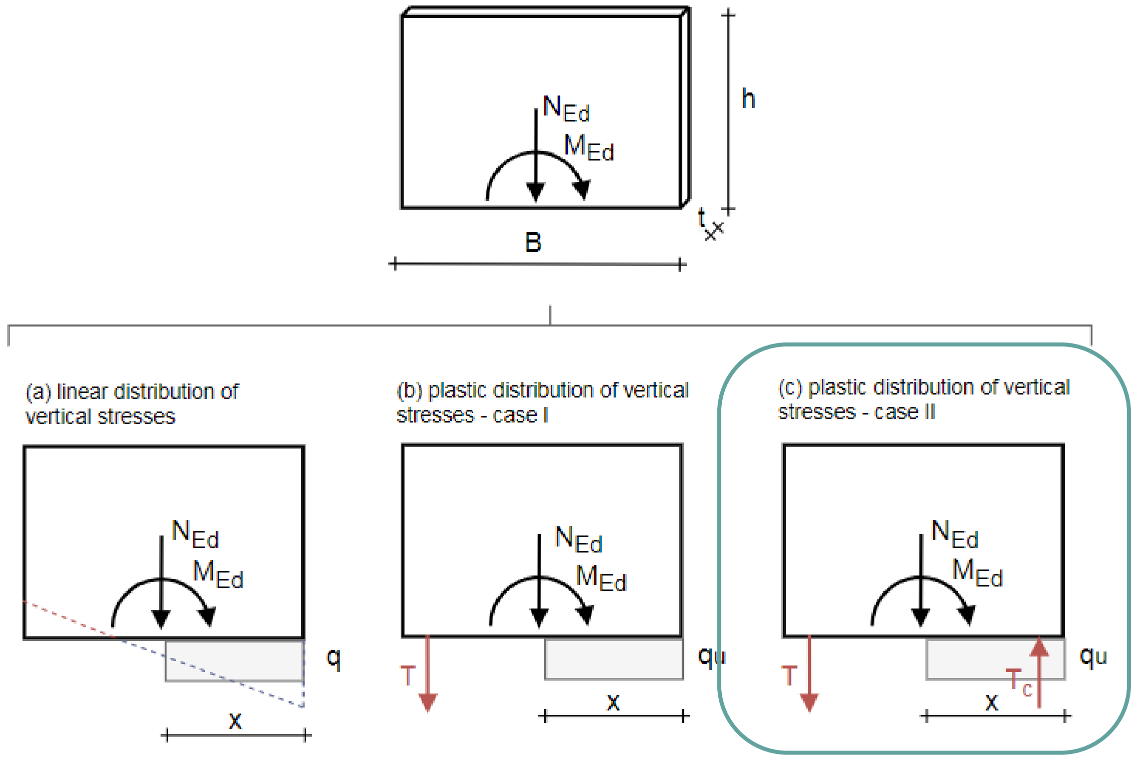
Starting point supposition: the section is subjected to plastic distribution of vertical stresses – case I



$$q_u = percent \cdot f_{cd} \cdot t^*$$

$$\begin{cases} T + N_{Ed} - q_u \cdot x = 0 \\ M_{Ed} + N_{Ed} \cdot e - q_u \cdot x \cdot \left(d - \frac{x}{2}\right) = 0 \end{cases}$$

$$T = q_u \cdot d \cdot \left(1 - \sqrt{1 - 2 \cdot \frac{M_{Ed} + N_{Ed} \cdot (B/2 - d_{edge})}{q_u \cdot d^2}}\right) - N_{Ed}$$

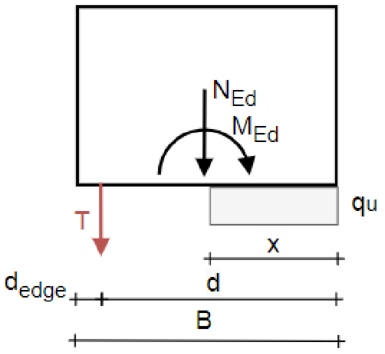


Supposition not confirmed if: $$1 - 2 \cdot \frac{M_{Ed} + N_{Ed} \cdot (B/2 - d_{edge})}{q_u \cdot d^2} < 0$$

(c) Plastic distribution of vertical stresses – case II

- Bending with axial force– design principles for walls
  - Design approach – mathematical model

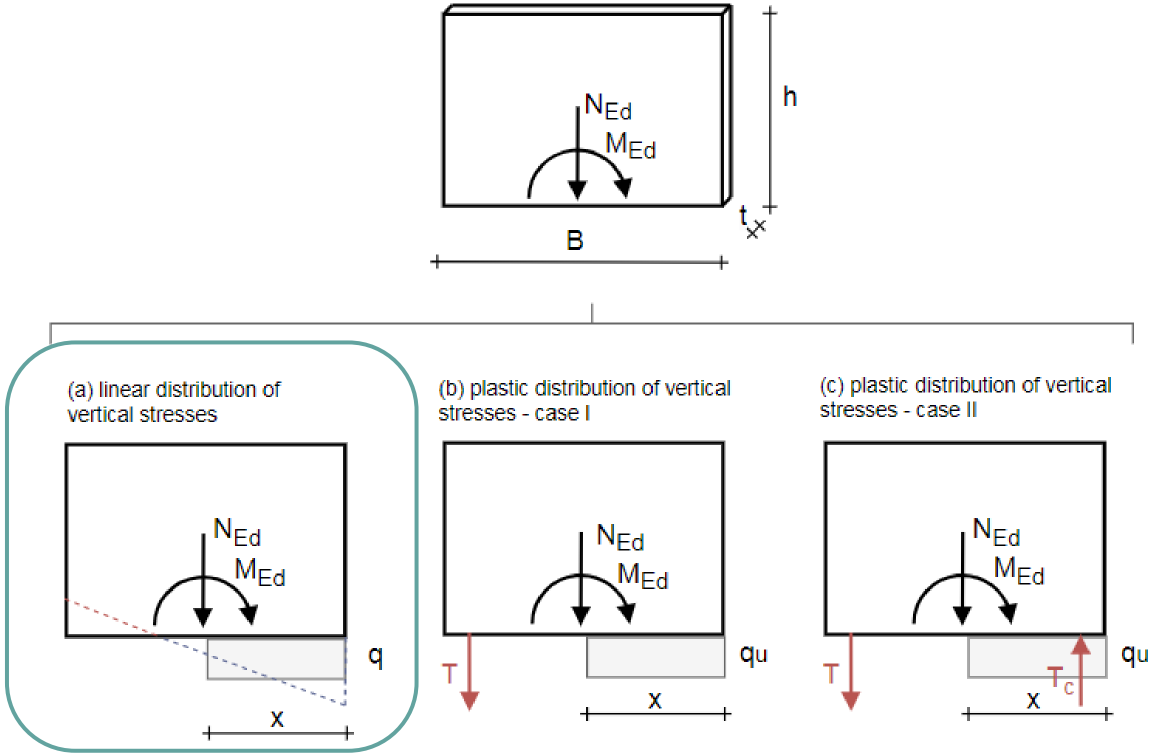
Starting point supposition: the section is subjected to plastic distribution of vertical stresses – case I



$$q_u = percent \cdot f_{cd} \cdot t^*$$

$$\begin{cases} T + N_{Ed} - q_u \cdot x = 0 \\ M_{Ed} + N_{Ed} \cdot e - q_u \cdot x \cdot \left(d - \frac{x}{2}\right) = 0 \end{cases}$$

$$T = q_u \cdot d \cdot \left(1 - \sqrt{1 - 2 \cdot \frac{M_{Ed} + N_{Ed} \cdot (B/2 - d_{edge})}{q_u \cdot d^2}}\right) - N_{Ed}$$



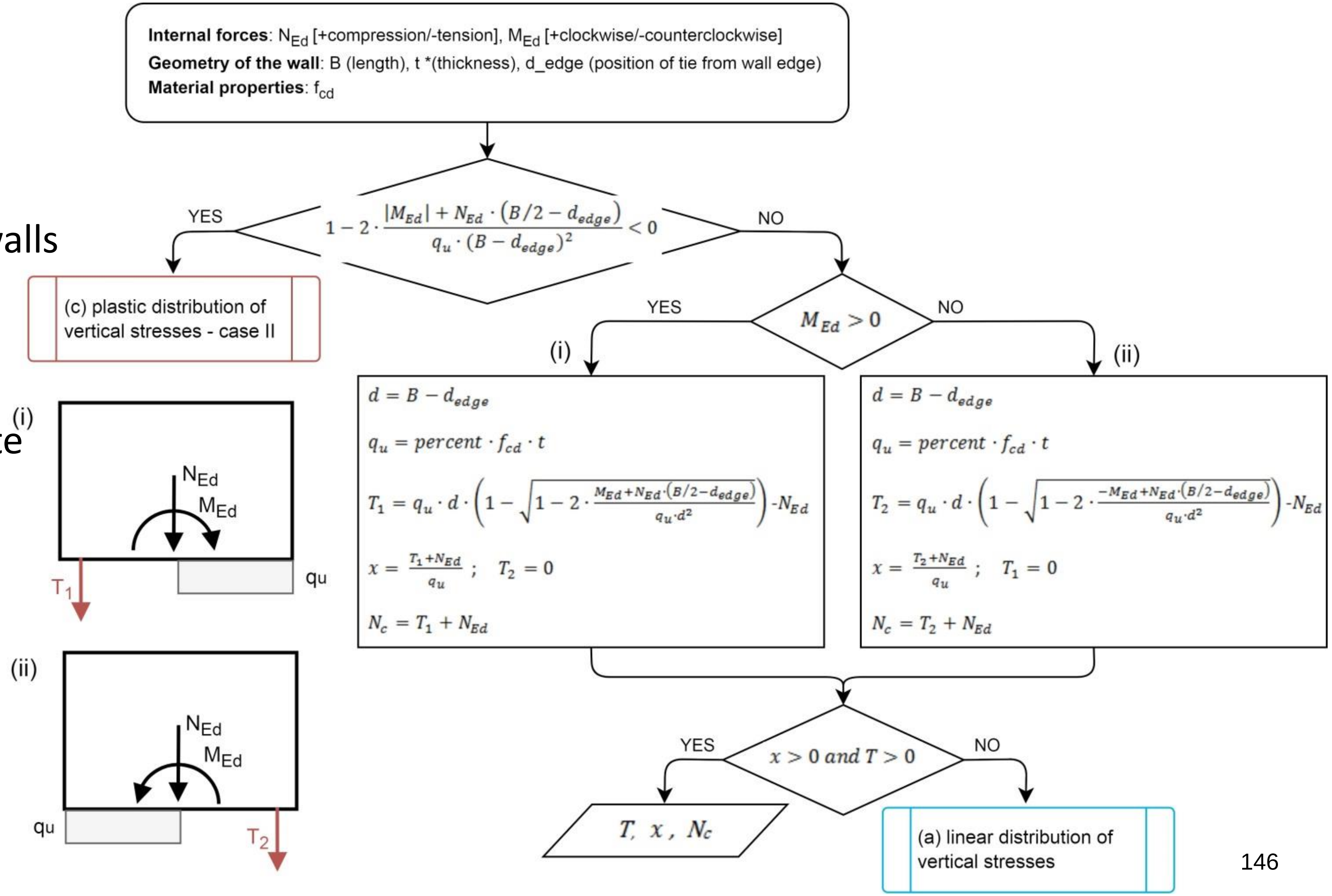
Supposition not confirmed if:

$$\begin{cases} 1 - 2 \cdot \frac{M_{Ed} + N_{Ed} \cdot (B/2 - d_{edge})}{q_u \cdot d^2} \geq 0 \\ x < 0 \text{ or /and } T < 0 \end{cases}$$

(a) Linear distribution of vertical stresses

Evaluation of shear wall behavior

- Bending with axial force– design principles for walls
  - Mathematical model
    - Case (a) needed to assess:
      - Ties for walls under tension
      - Buckling check for compressed walls
    - Case (b) “textbook” case:
      - Tie design
      - Compression resultant in concrete
    - Case (c): REDESIGN



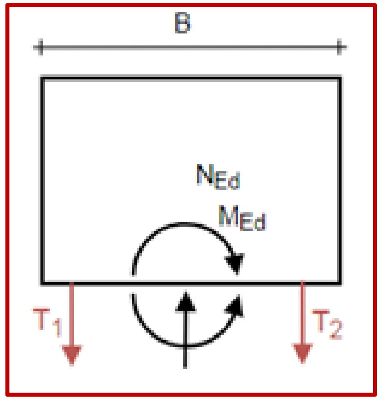
Evaluation of shear wall behavior

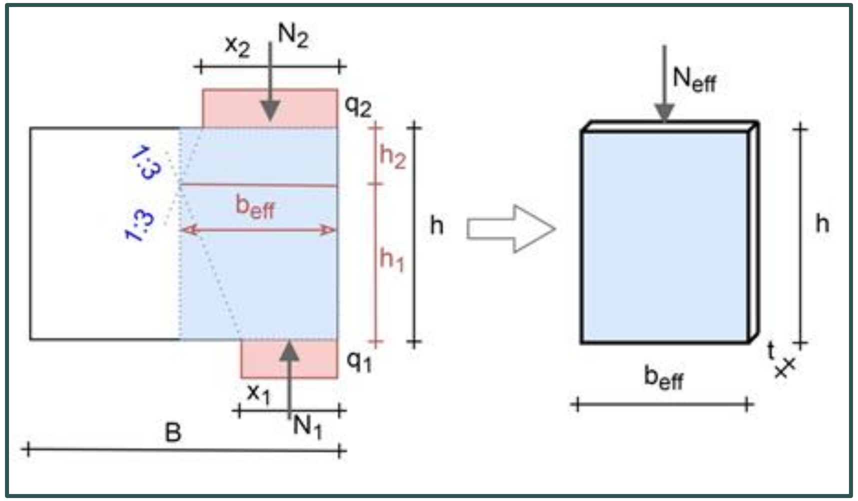
- Bending with axial force

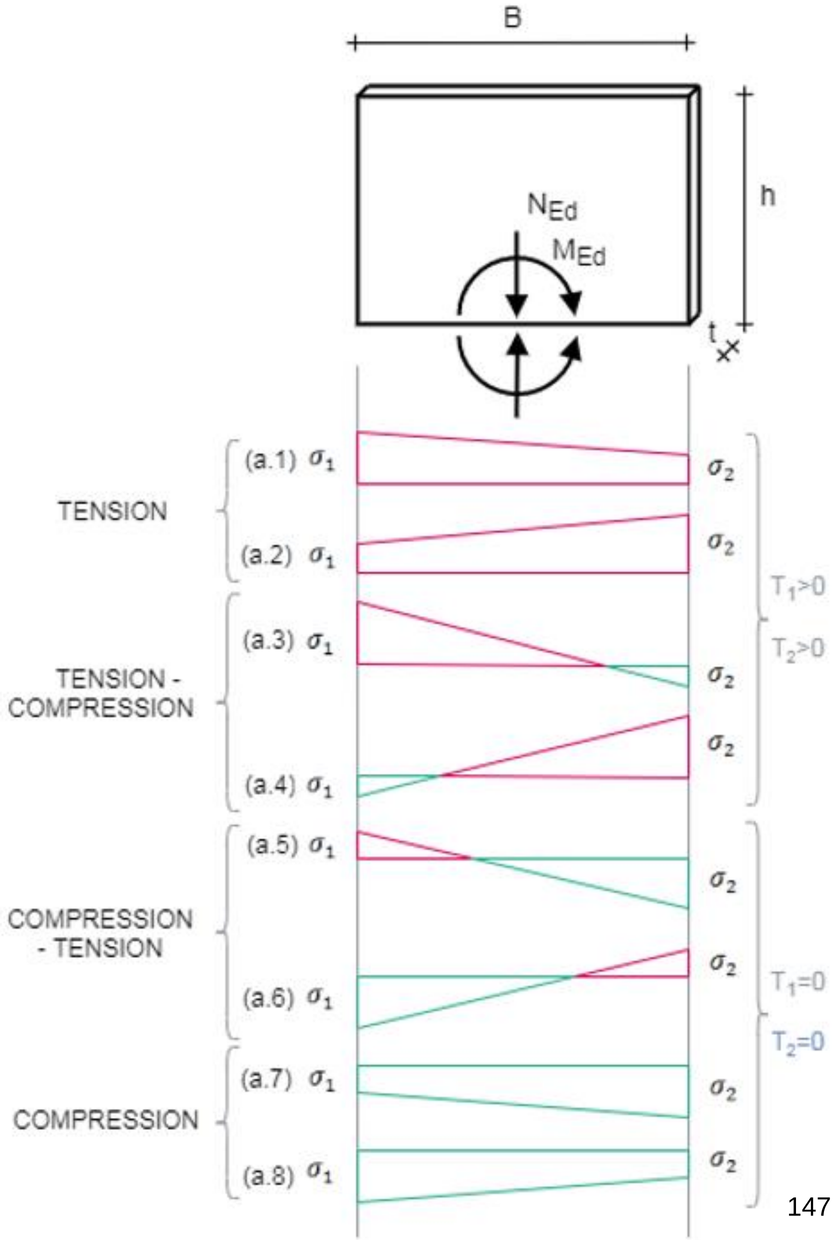
  (a) linear distribution of vertical stresses

- Whole section subjected to tension (a.1) & (a.2)
- Concrete section under tension and compression (a.3) & (a.4)

- Concrete section under compression and tension (a.5) & (a.6)
- Entire section compressed (a.7) & (a.8)





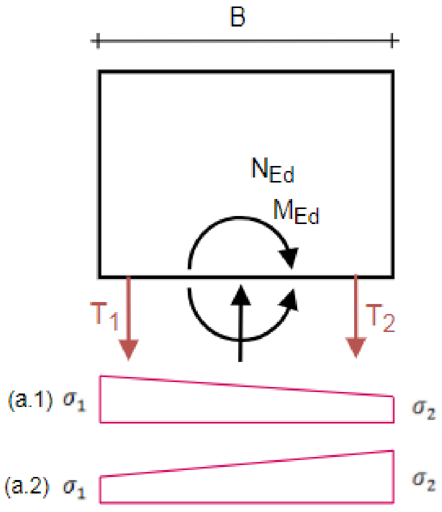


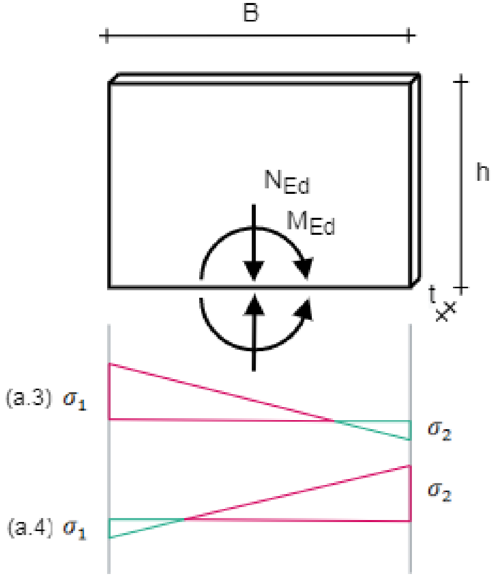
Evaluation of shear wall behavior

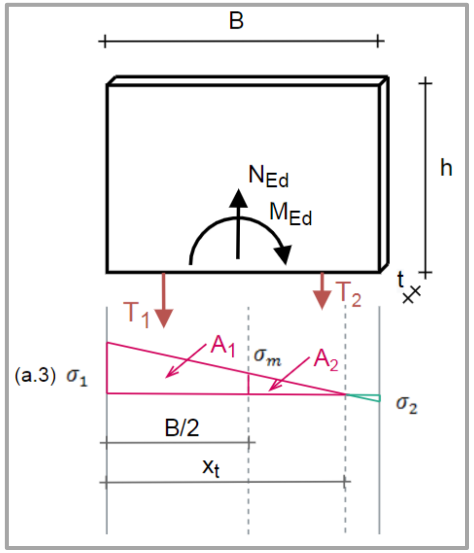
- Bending with axial force

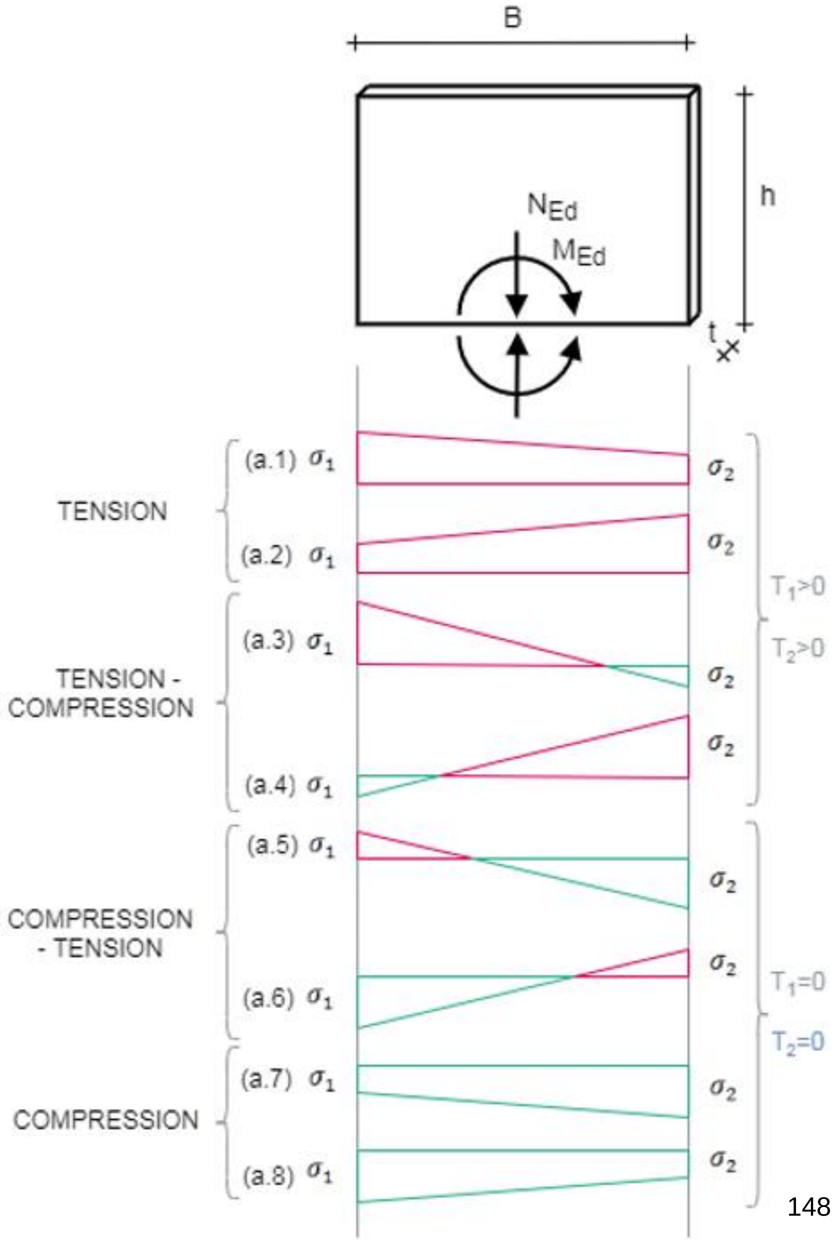
  (a) linear distribution of vertical stresses

- Whole section subjected to tension (a.1) & (a.2)
- Concrete section under tension and compression (a.3) & (a.4)







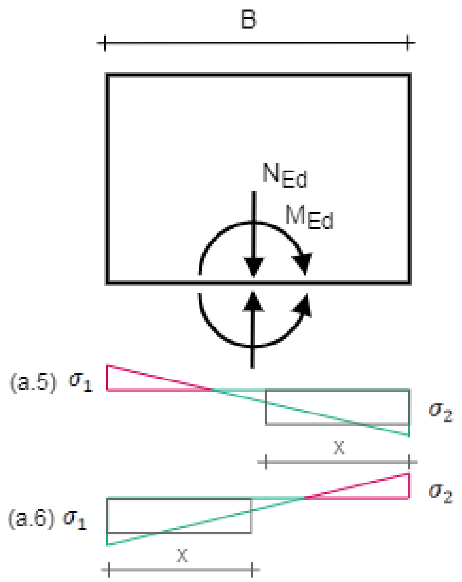


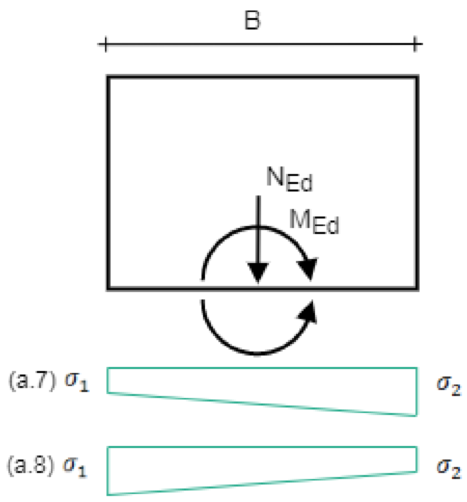
Evaluation of shear wall behavior

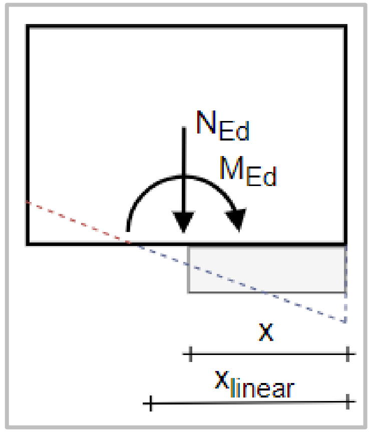
- Bending with axial force

  (a) linear distribution of vertical stresses

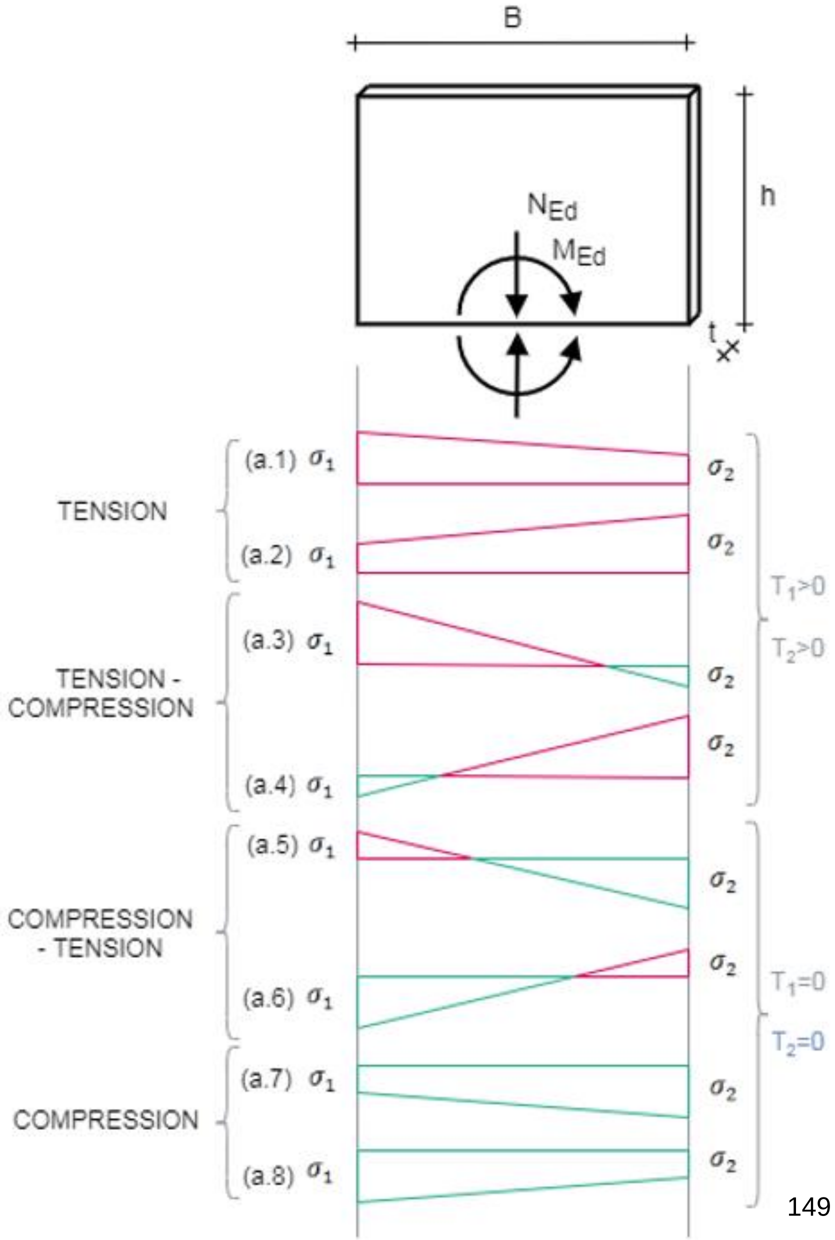
- Concrete section under compression and tension (a.5) & (a.6)
- Entire section compressed (a.7) & (a.8)



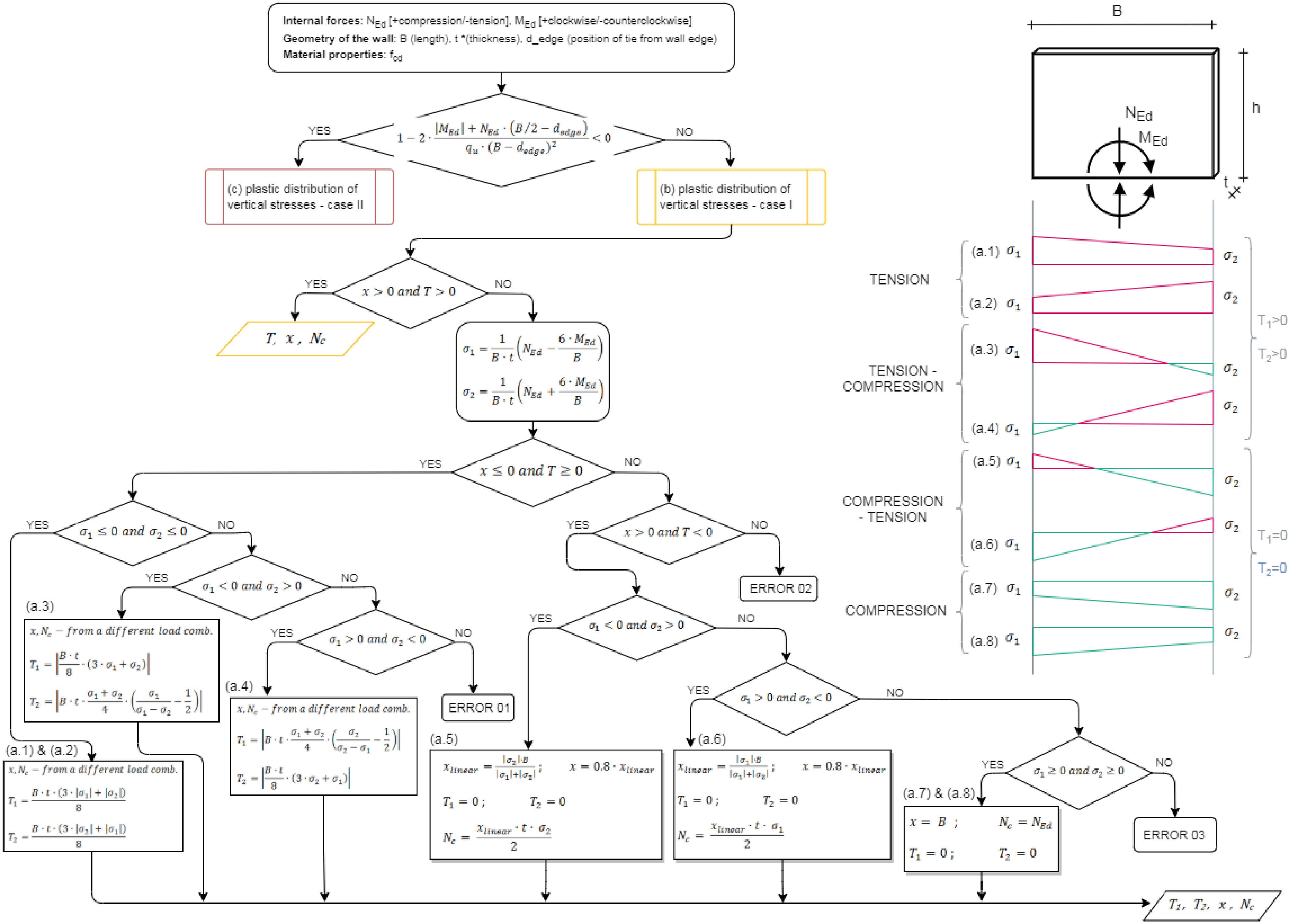




$$x = 0.8 \cdot x_{linear}$$



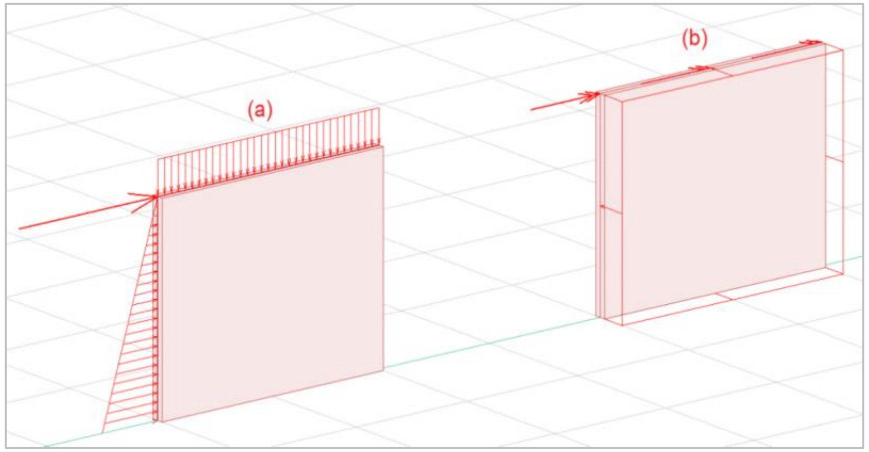
N-M linear

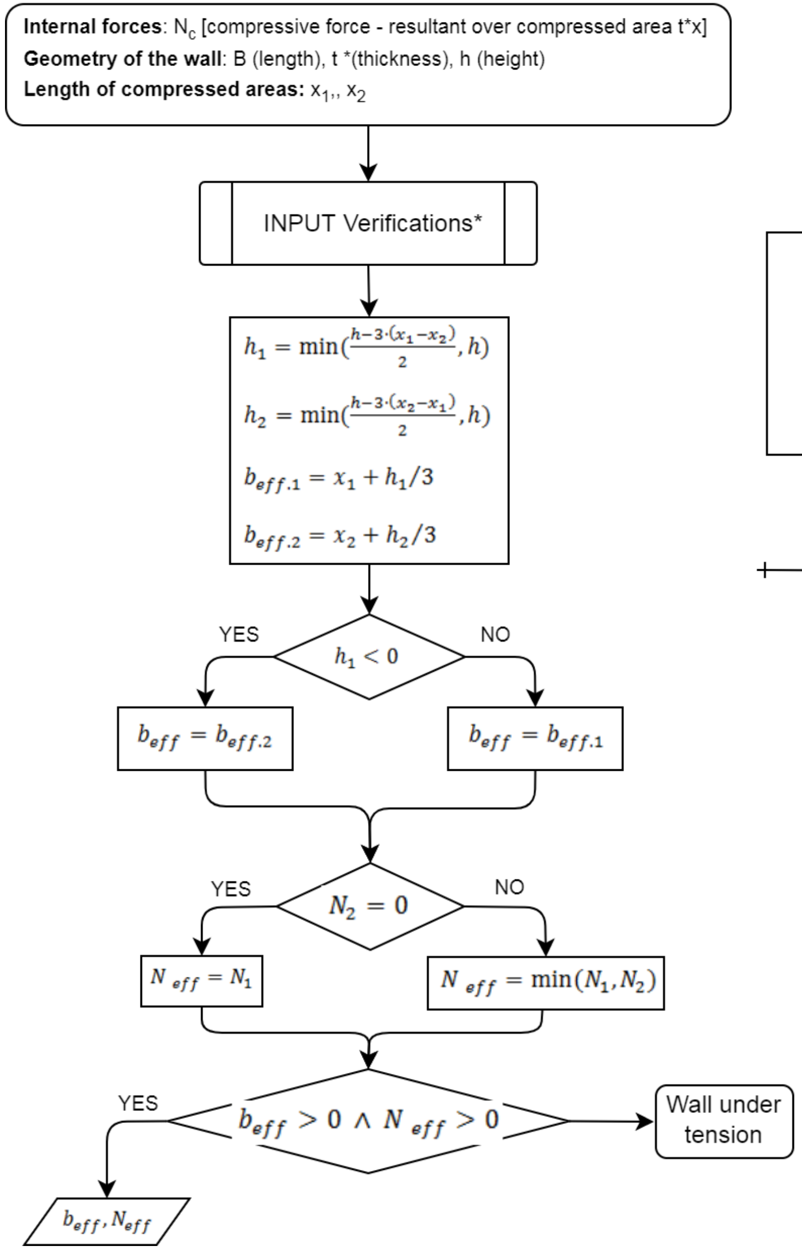


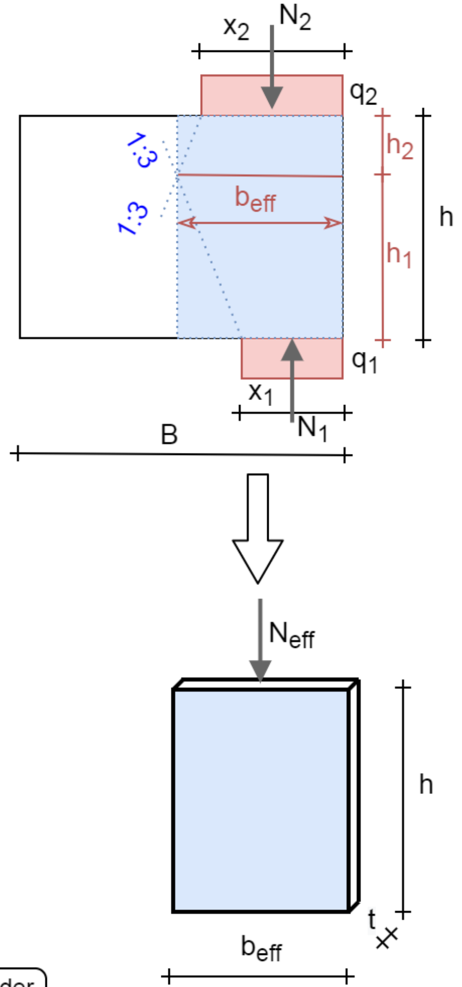
Internal forces: N_{Ed} [+compression/-tension], M_{Ed} [+clockwise/-counterclockwise]
Geometry of the wall: B (length), t *(thickness), d_edge (position of tie from wall edge)
Material properties: f_{cd}
$1 - 2 \cdot \frac{|M_{Ed}| + N_{Ed} \cdot (B/2 - d_{edge})}{q_u \cdot (B - d_{edge})^2} < 0$
YES
NO
(c) plastic distribution of vertical stresses - case II
(b) plastic distribution of vertical stresses - case I
$x > 0$ and $T > 0$
T, x, N_c
$\sigma_1 = \frac{1}{B \cdot t}\left(N_{Ed} - \frac{6 \cdot M_{Ed}}{B}\right)$
$\sigma_2 = \frac{1}{B \cdot t}\left(N_{Ed} + \frac{6 \cdot M_{Ed}}{B}\right)$
$x \leq 0$ and $T \geq 0$
$\sigma_1 \leq 0$ and $\sigma_2 \leq 0$
$\sigma_1 < 0$ and $\sigma_2 > 0$
$\sigma_1 > 0$ and $\sigma_2 < 0$
(a.3)
x, N_c – from a different load comb.
$T_1 = \left|\frac{B \cdot t}{8} \cdot (3 \cdot \sigma_1 + \sigma_2)\right|$
$T_2 = \left|B \cdot t \cdot \frac{\sigma_1 + \sigma_2}{4} \cdot \left(\frac{\sigma_1}{\sigma_1 - \sigma_2} - \frac{1}{2}\right)\right|$
(a.4)
x, N_c – from a different load comb.
$T_1 = \left|B \cdot t \cdot \frac{\sigma_1 + \sigma_2}{4} \cdot \left(\frac{\sigma_2}{\sigma_2 - \sigma_1} - \frac{1}{2}\right)\right|$
$T_2 = \left|\frac{B \cdot t}{8} \cdot (3 \cdot \sigma_2 + \sigma_1)\right|$
ERROR 01
(a.1) & (a.2)
x, N_c – from a different load comb.
$T_1 = \frac{B \cdot t \cdot (3 \cdot |\sigma_1| + |\sigma_2|)}{8}$
$T_2 = \frac{B \cdot t \cdot (3 \cdot |\sigma_2| + |\sigma_1|)}{8}$
$x > 0$ and $T < 0$
ERROR 02
$\sigma_1 < 0$ and $\sigma_2 > 0$
$\sigma_1 > 0$ and $\sigma_2 < 0$
(a.5)
$x_{linear} = \frac{|\sigma_2| \cdot B}{|\sigma_1| + |\sigma_2|}$; $x = 0.8 \cdot x_{linear}$
$T_1 = 0$; $T_2 = 0$
$N_c = \frac{x_{linear} \cdot t \cdot \sigma_2}{2}$
(a.6)
$x_{linear} = \frac{|\sigma_1| \cdot B}{|\sigma_1| + |\sigma_2|}$; $x = 0.8 \cdot x_{linear}$
$T_1 = 0$; $T_2 = 0$
$N_c = \frac{x_{linear} \cdot t \cdot \sigma_1}{2}$
$\sigma_1 \geq 0$ and $\sigma_2 \geq 0$
(a.7) & (a.8)
$x = B$; $N_c = N_{Ed}$
$T_1 = 0$; $T_2 = 0$
ERROR 03
T_1, T_2, x, N_c
B
h
t
N_{Ed}
M_{Ed}
TENSION
(a.1) σ_1 σ_2
(a.2) σ_1 σ_2
TENSION - COMPRESSION
(a.3) σ_1 σ_2
(a.4) σ_1 σ_2
$T_1>0$
$T_2>0$
COMPRESSION - TENSION
(a.5) σ_1 σ_2
(a.6) σ_1 σ_2
$T_1=0$
$T_2=0$
COMPRESSION
(a.7) σ_1 σ_2
(a.8) σ_1 σ_2

Evaluation of shear wall behavior

- Bending with axial force: effective wall width
  - Input for out-of-plane N-M design
  - + eccentricities
  - Buckling check







Evaluation of shear wall behavior

- Bending with axial force: effective wall width
  - Buckling length for walls

$\lambda_0 = l_0 / i$ – slenderness ratio

$$l_0 = \beta \cdot l_w$$

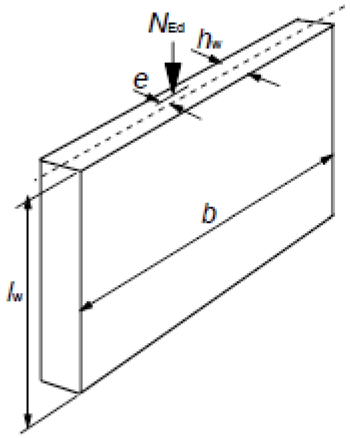
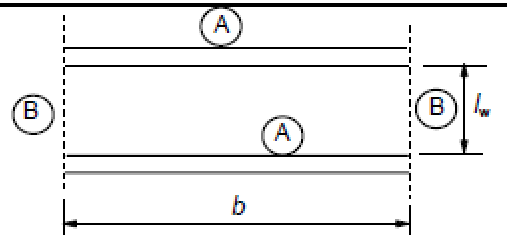
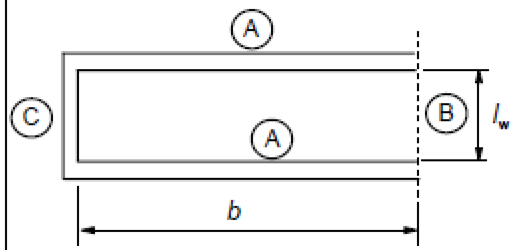
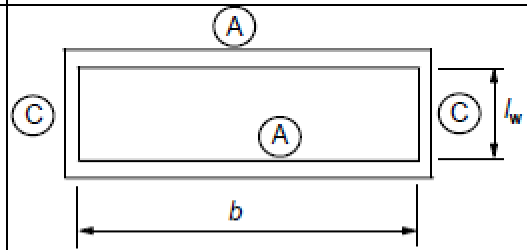


Table 12.1: Values of β for different edge conditions

Lateral restraint	Sketch	Expression	Factor β
along two edges			β = 1,0 for any ratio of l_w/b
Along three edges		$\beta = \dfrac{1}{1+\left(\dfrac{l_w}{3b}\right)^2}$	b/l_w : β 0,2 : 0,26 0,4 : 0,59 0,6 : 0,76 0,8 : 0,85 1,0 : 0,90 1,5 : 0,95 2,0 : 0,97 5,0 : 1,00
Along four edges		If $b \geq l_w$ $\beta = \dfrac{1}{1+\left(\dfrac{l_w}{b}\right)^2}$ If $b < l_w$ $\beta = \dfrac{b}{2l_w}$	b/l_w : β 0,2 : 0,10 0,4 : 0,20 0,6 : 0,30 0,8 : 0,40 1,0 : 0,50 1,5 : 0,69 2,0 : 0,80 5,0 : 0,96

(A) - Floor slab (B) - Free edge (C) - Transverse wall

Note: The information in Table 12.1 assumes that the wall has no openings with a height exceeding 1/3 of the wall height l_w or with an area exceeding 1/10 of the wall area. In walls laterally restrained along 3 or 4 sides with openings exceeding these limits, the parts between the openings should be considered as laterally restrained along 2 sides only and be designed accordingly.

Slenderness of column and walls according to 12.6.5.1 of En 1992-1-1

- Bending with axial force: local 2nd order effects
  - EN 1992-1-1
    - 5.8.2.(6) Second order effects can be ignored if less than 10%
    - 5.8.3 Simplified criteria for second order effects

$\lambda_0 = l_0 / i$ – slenderness ratio

$\lambda_{lim} = 20 \cdot A \cdot B \cdot C / \sqrt{n}$

$A = 1/(1 + 0.2\varphi_{ef})$ if φ_{ef} is not know, A=0.7 may be used

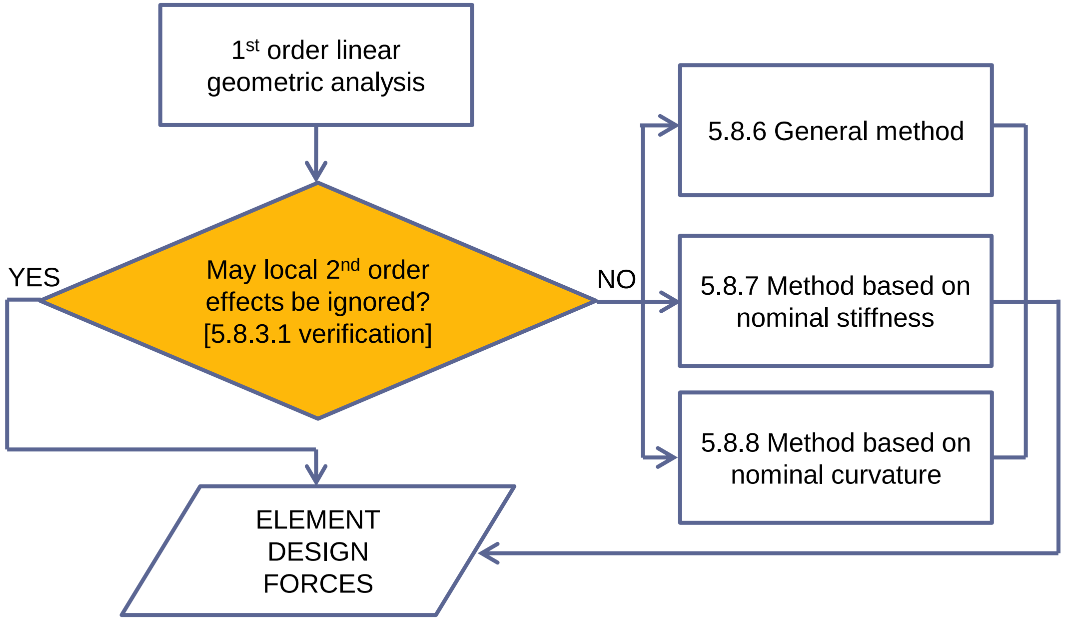
$B = \sqrt{1 + 2\omega}$ if ω is not know, B=1.1 may be used

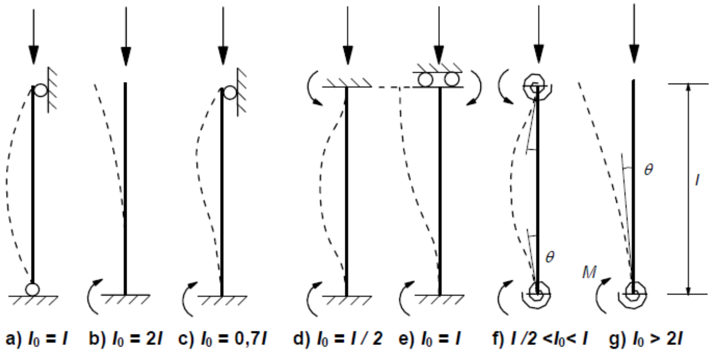
$C = 1.7 - r_m$ if r_m is not know, C=0.7 may be used

φ_{ef} effective creep ratio

$\omega = A_s f_{yd} / A_c f_{cd}$ mechanical reinforcement ratio

$n = N_{Ed} / A_c f_{cd}$ relative normal force





Examples of different buckling modes and corresponding effective lengths for isolated members acc. to Figure 3.5 of EN 1992-1-1

Evaluation of shear wall behavior

- Bending with axial force: local 2nd order effects
  - EN 1992-1-1
  - 5.8.7 Method based on nominal stiffness
    - Nominal stiffness is calculated

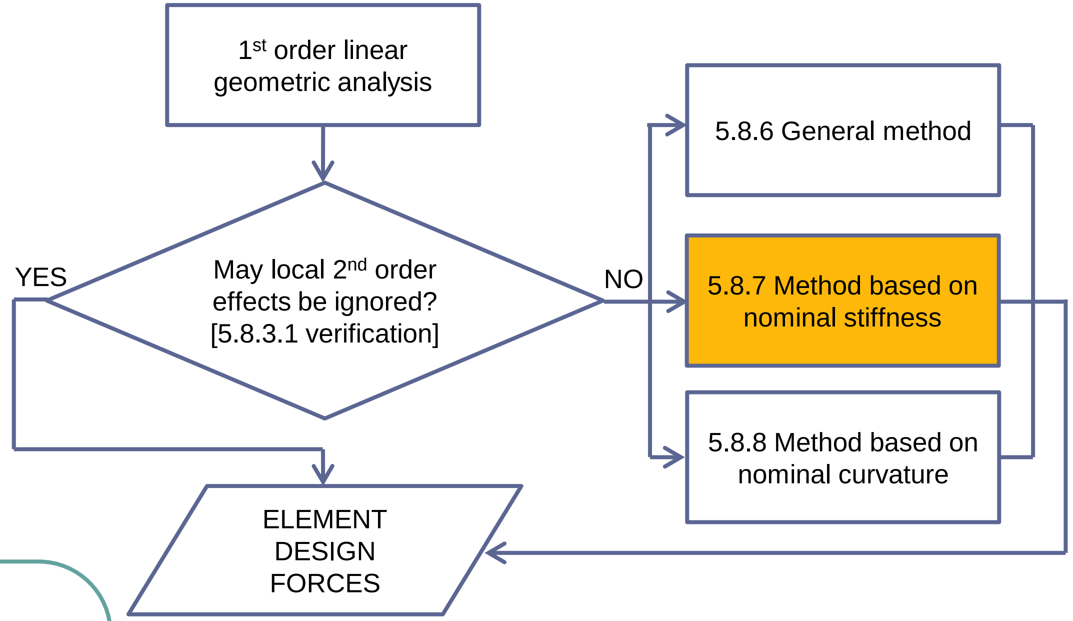
$$EI = K_c E_{cd} I_c + K_s E_s I_s$$

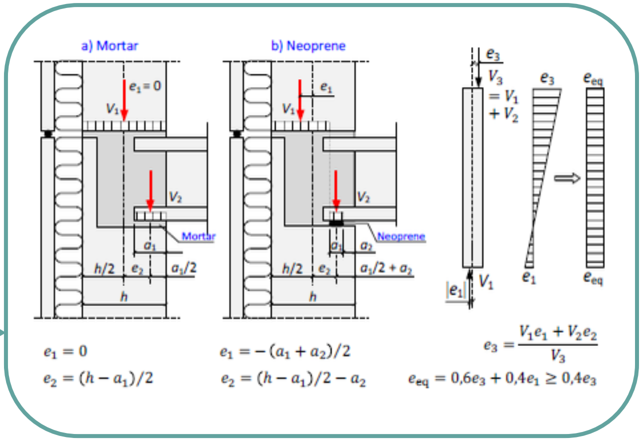
K_c factor for effects of cracking, creep etc.

K_s factor for contribution of reinforcement

    - Buckling load based on nominal stiffness N_B
    - Moment magnification factor

$$M_{Ed} = M_{0Ed}\left[1 + \frac{\beta}{N_B/N_{Ed} - 1}\right]$$





Eccentricity of loads must be accounted for in evaluation of M_{0Ed}

Evaluation of shear wall behavior

- Bending with axial force: local 2nd order effects
  - EN 1992-1-1
  - 5.8.7 Method based on nominal stiffness
    - Nominal stiffness is calculated

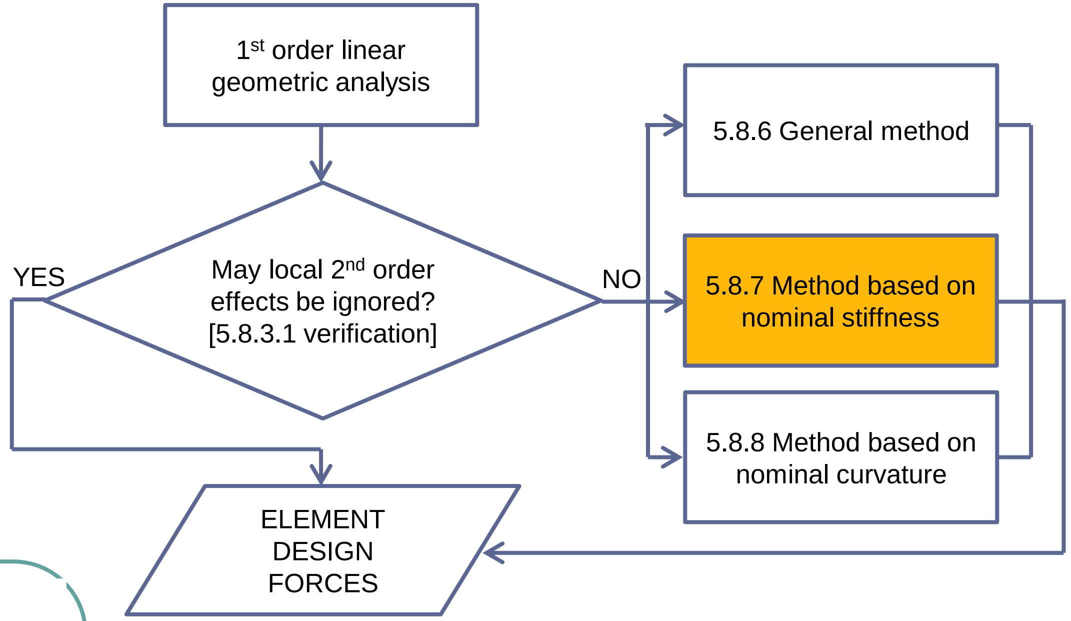
$$EI = K_c E_{cd} I_c + K_s E_s I_s$$

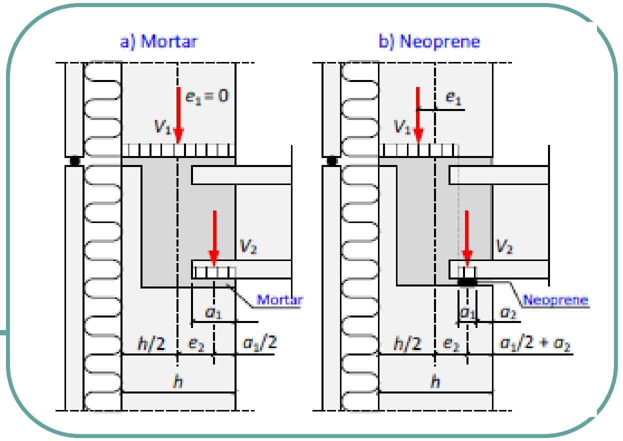
K_c factor for effects of cracking, creep etc.

K_s factor for contribution of reinforcement

    - Buckling load based on nominal stiffness N_B
    - Moment magnification factor

$$M_{Ed} = M_{0Ed}\left[1 + \frac{\beta}{N_B/N_{Ed} - 1}\right]$$

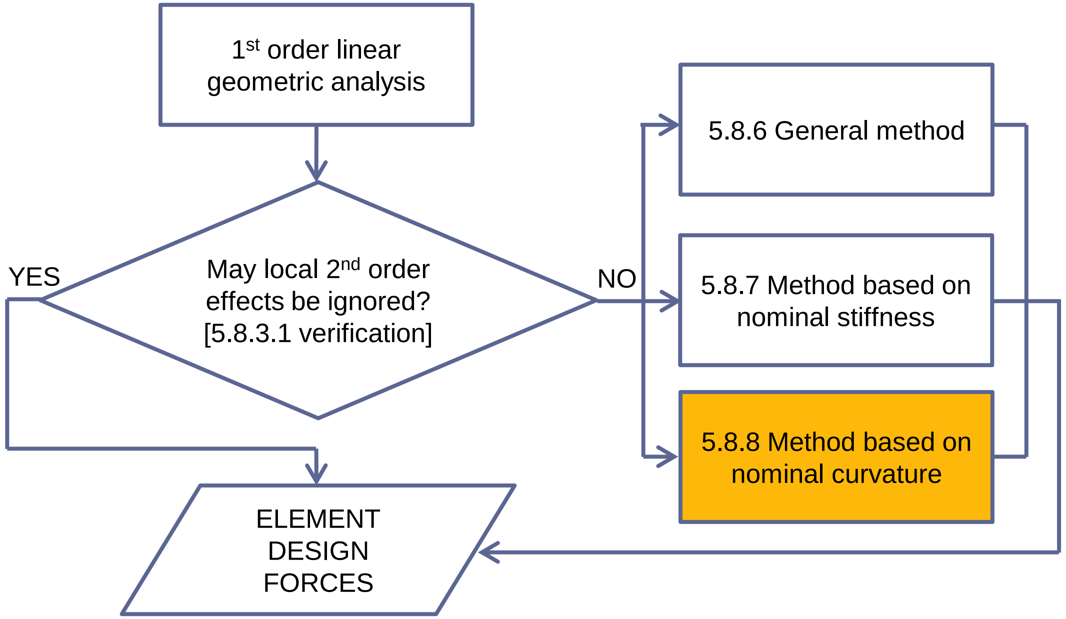




Eccentricity of loads must be accounted for in evaluation of first order bending moment

Evaluation of shear wall behavior

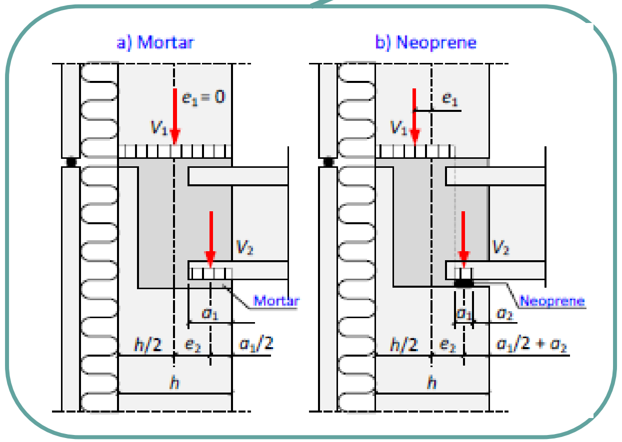
- Bending with axial force: local 2nd order effects
  - EN 1992-1-1
  - 5.8.8 Method based on nominal curvature
    - Provides a nominal second order moment
    - Based on the effective length and estimated maximum curvature
    - **Better for second order effects evaluation of shear walls than the stiffness method**



$$M_{Ed} = M_{0Ed} + M_2$$

$$M_2 = N_{Ed} \cdot e_2$$

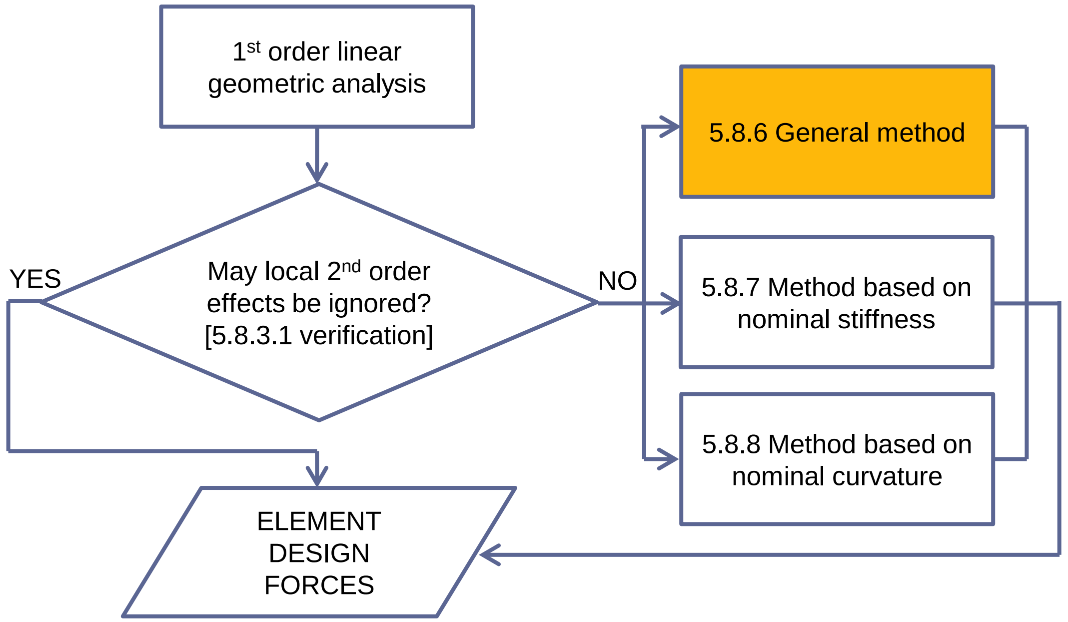
Nominal second order moment



Eccentricity of loads must be accounted for in evaluation of first order bending moment

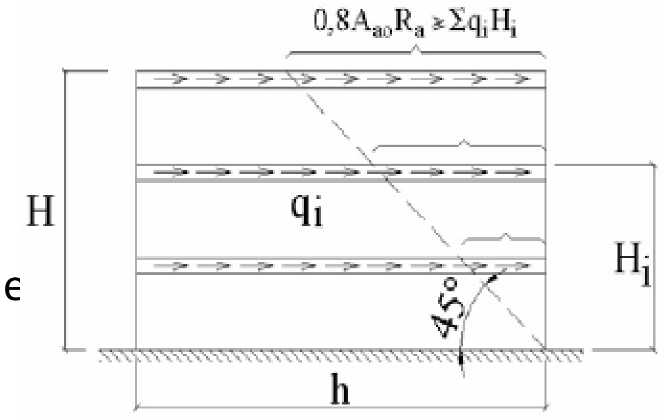
Evaluation of shear wall behavior

- Bending with axial force: local 2nd order effects
  - EN 1992-1-1
  - 5.8.6 General method
    - Based on non-linear analysis
    - **The most accurate one**
    - Has been used in the calibration of the two previous simplified methods (stiffness and curvature)
    - Applicable to both plain and reinforced walls
    - Simplified methods applicable only for reinforced walls



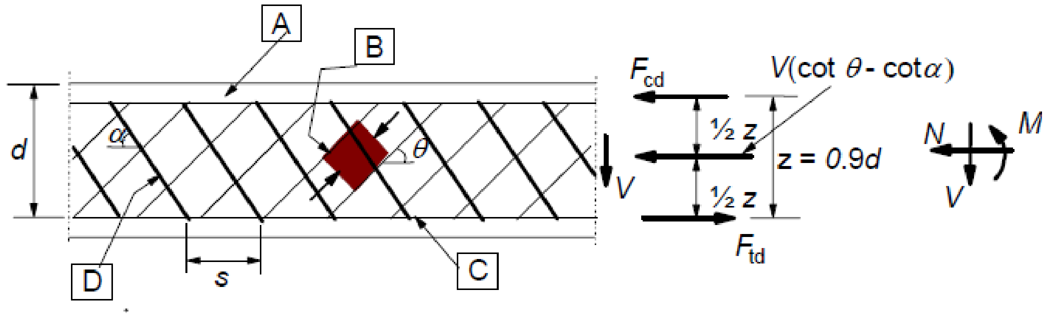
Evaluation of shear wall behavior

- Shear design of walls
  - No specific approach provided in EC
  - Romania code approach
    - Inclined section approach
    - Wall and connection reinforcement/joints intersected by the presumed 45° inclined crack need to accommodate the demand
    - If H/h>1 shear capacity of concrete and horizontal reinforcement are considered to resist shear
    - If H/h<1 part of the vertical reinforcement is also accounted for

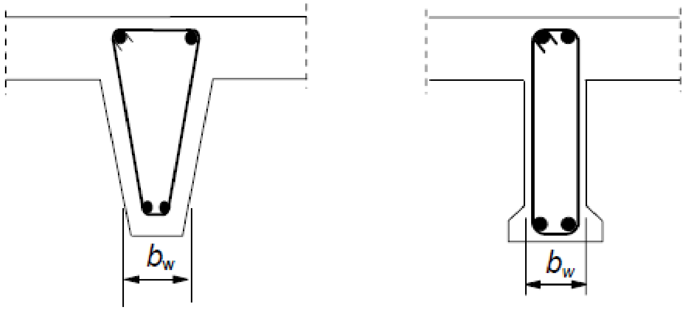


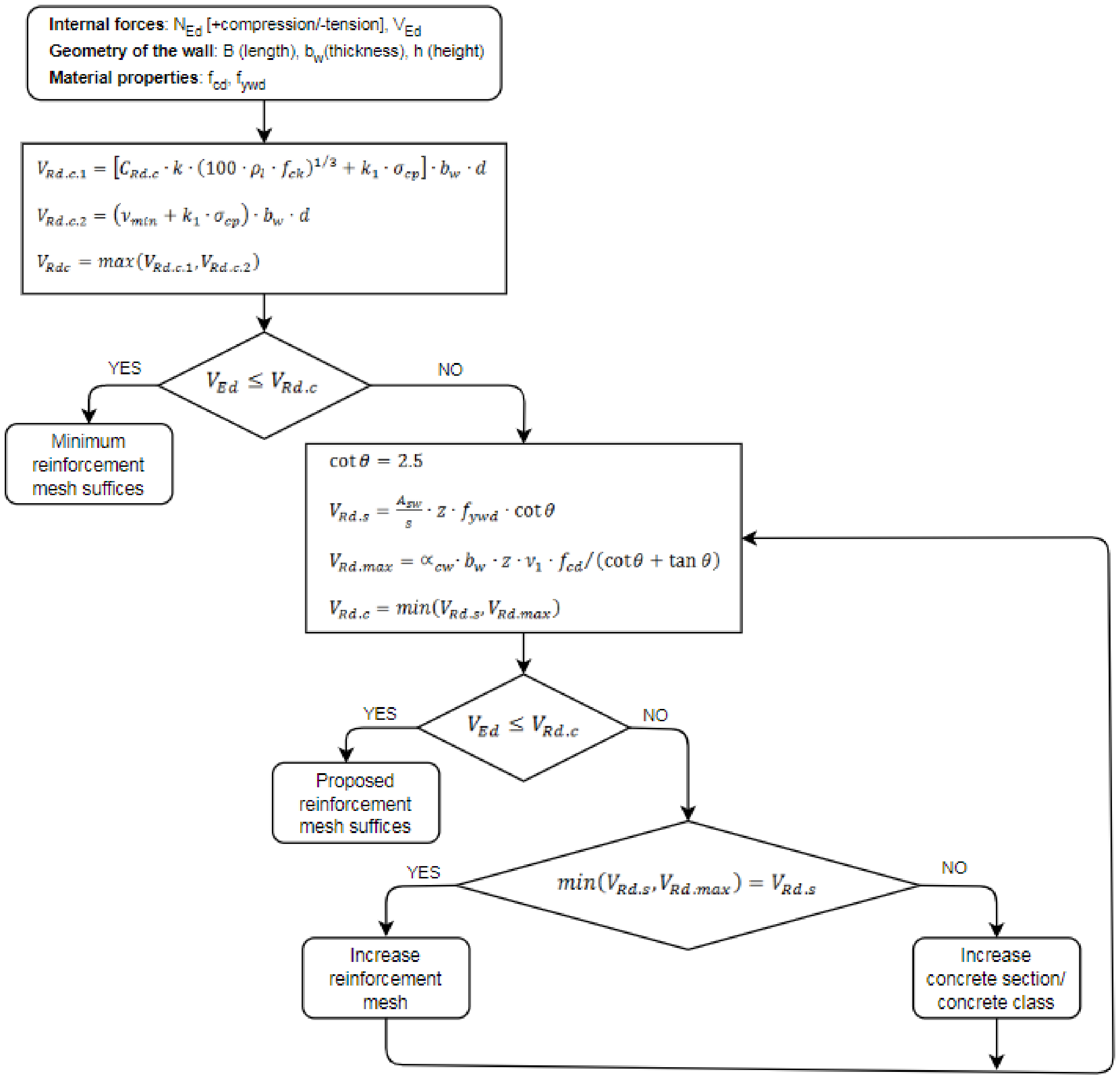
Evaluation of shear wall behavior

- Shear design of walls
  - Shear design according to EN 1992-1-1:2004 Chapter 6.2.2 & Chapter 6.2.3

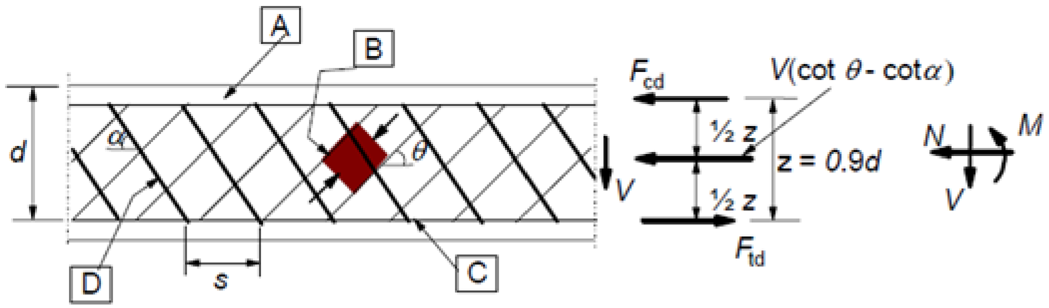


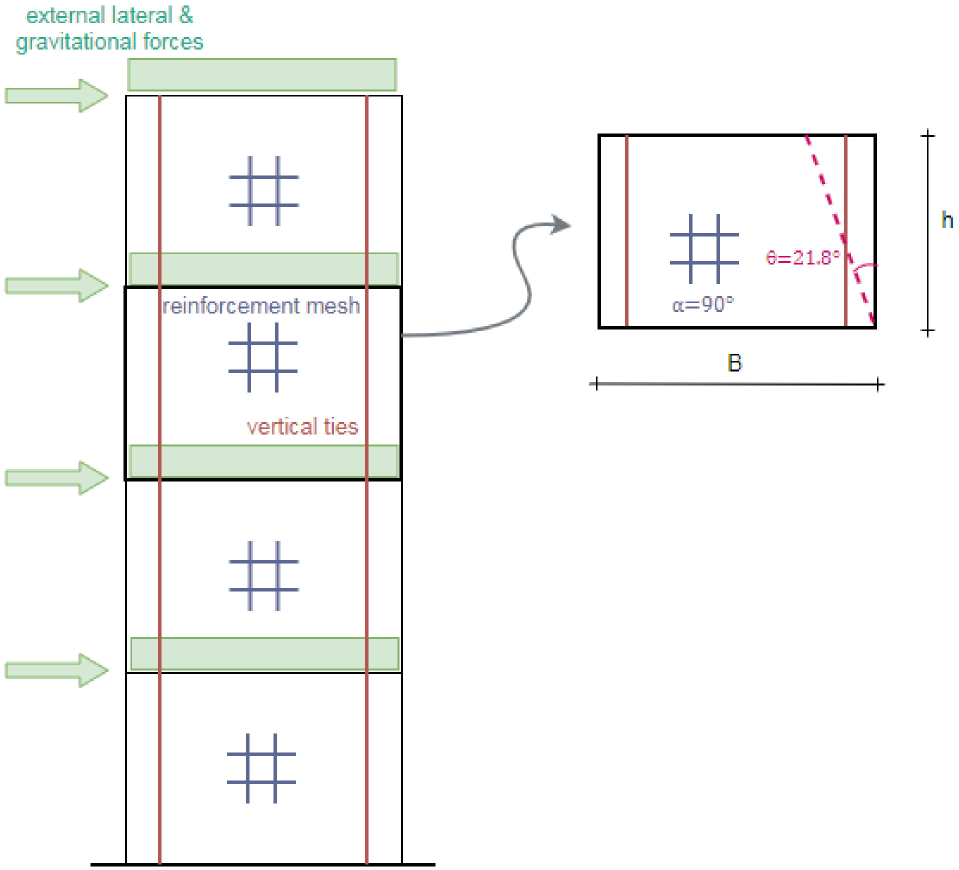
A - compression chord, B - struts, C - tensile chord, D - shear reinforcement





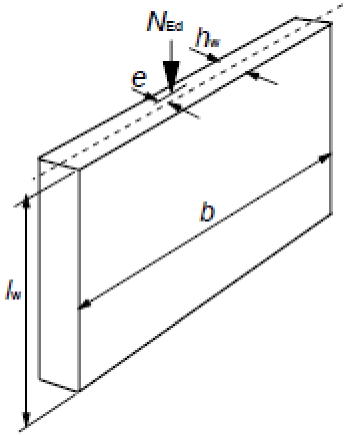
- Shear design of walls
  - Shear design according to EN 1992-1-1:2004 (6.2.2 & 6.2.3)
    - $1 \leq \cot\theta \leq 2.5$
    - $\cot\theta = 2.5\ (\theta = 21.8°)$– the upper value is considered as it yields the highest capacity for the inclined tie, namely the highest capacity for the shear reinforcement ($V_{Rd.s}$)
    - z=0.9·d (d=0.9*section length)

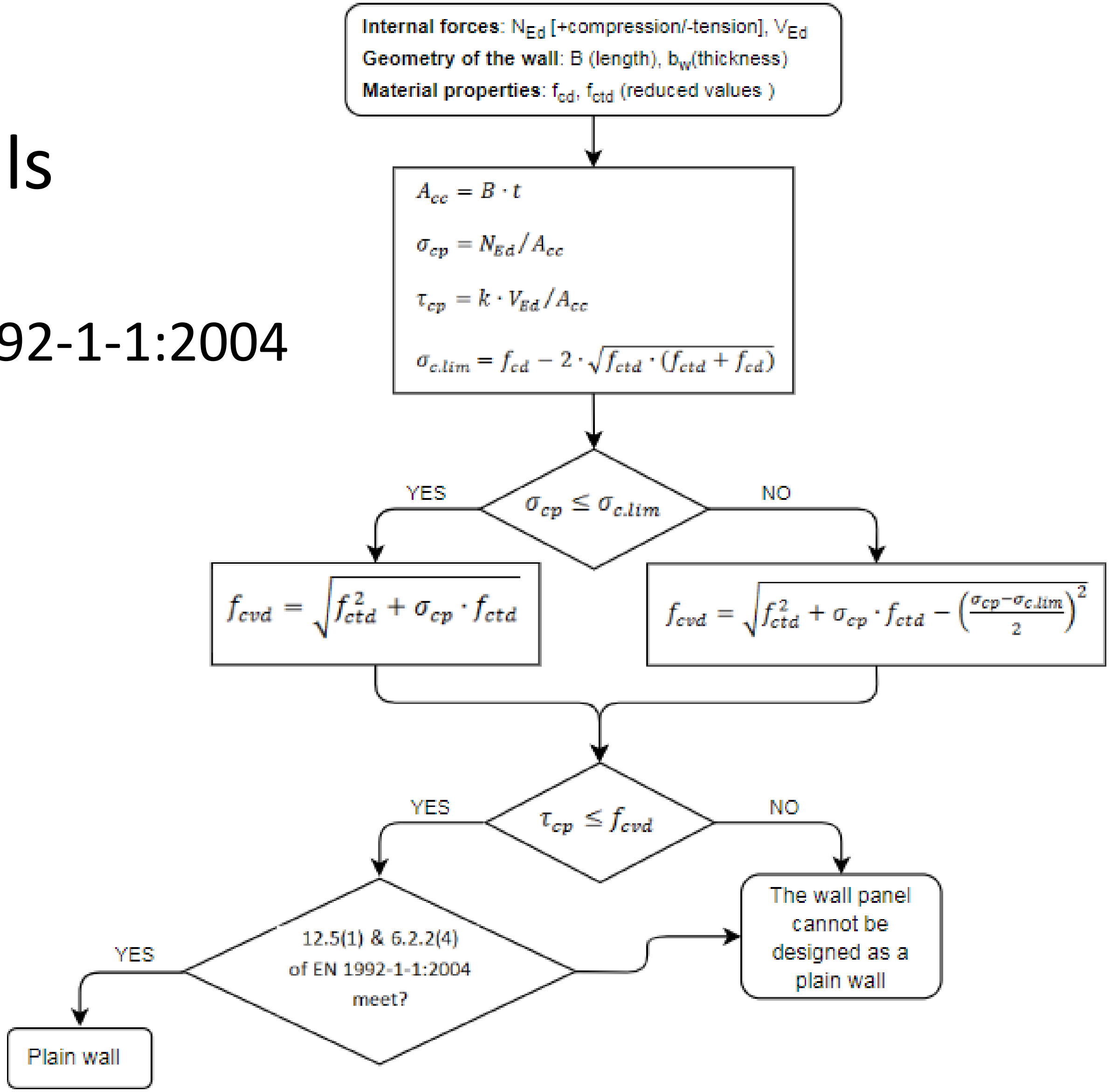




Evaluation of shear wall behavior

- Shear design of walls – plain walls (unreinforced walls)
  - Plain wall verification according to EN 1992-1-1:2004 Chapter 12.6.3
    - If $\tau_{cp} \leq f_{cvd}$ plain wall shear verification is validated





Evaluation of shear wall behavior

- Shear design of walls – plain walls (unreinforced walls)
  - Plain wall verification according to EN 1992-1-1:2004 Chapter 12.6.3
    - If $\tau_{cp} \leq f_{cvd}$ plain wall shear verification is validated
  - 12.5.(1) of EN 1992-1-1:2004 :
    - Since plain concrete members have limited ductility, linear analysis with redistribution or a plastic approach to analysis, e.g. methods without an explicit check of the deformation capacity, should not be used unless their application can be justified.
  - 6.2.2(4) of EN 1992-1-1:2004:
    - For the general case of members subjected to a bending moment and an axial force, which can be shown to be uncracked in flexure at the ULS, reference is made to 12.6.3.

Evaluation of shear wall behavior

- Shear design of connections
  - According to chapter 6.2.5 of EN 1992-1-1:2004
    - Horizontal connection

$$V_{cd} = V_{Ed} - \mu \cdot Compression \; - c * f_{ctd}$$

      - V_{cd} needs to be accommodated by additional shear joints (dowel effect)

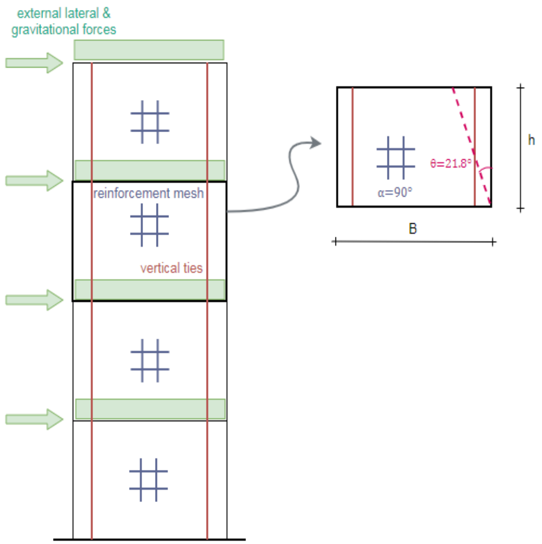
    - Vertical connection

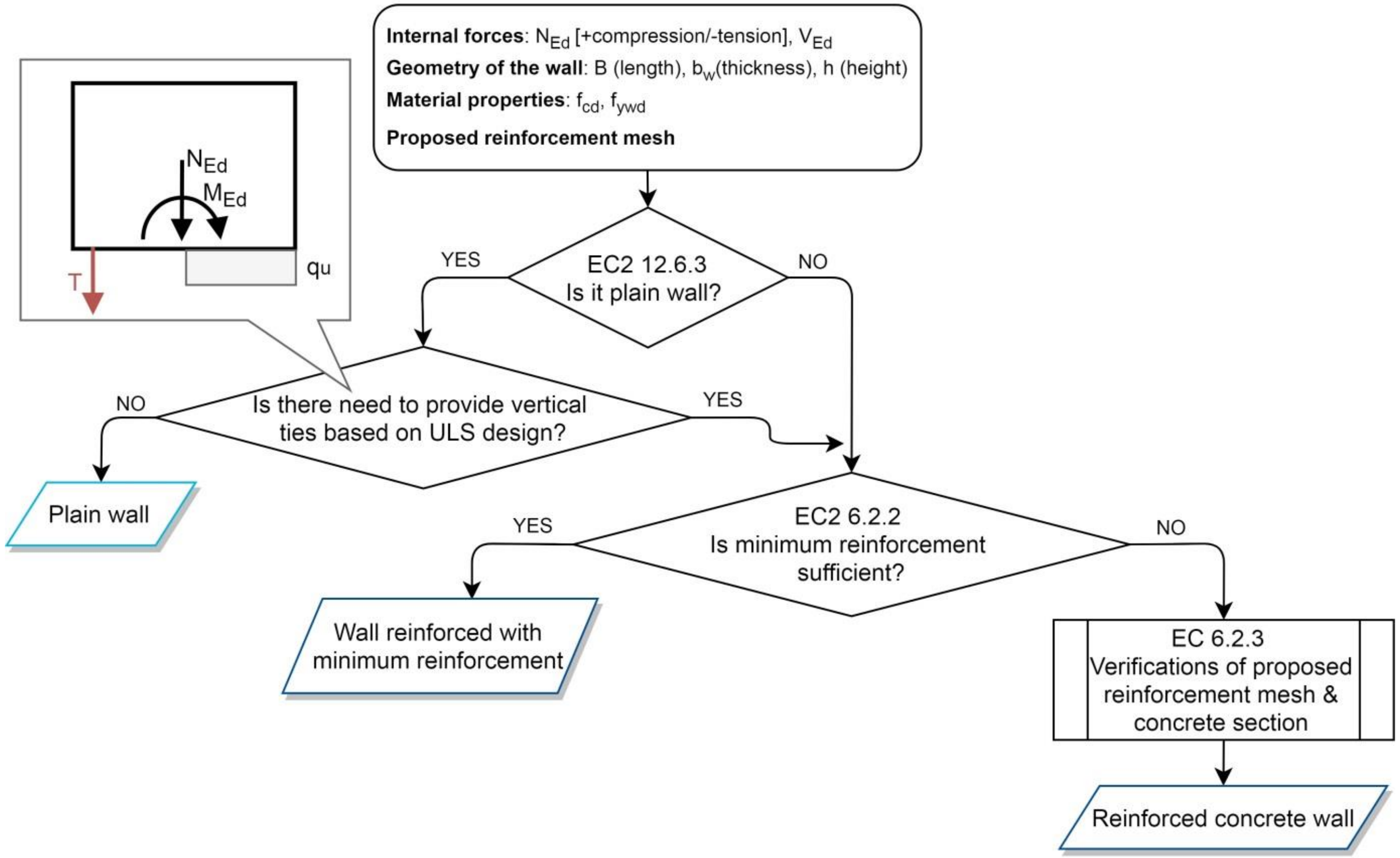
$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot (\mu \cdot \sin \propto + \cos \propto) \leq 0.5 \cdot v \cdot f_{cd}$$

      - A clamping force is assumed based on tension capacity of steel reinforcement/inserts
      - Shear capacity is evaluated based on the provided clamping

Evaluation of shear wall behavior

- Shear design of walls wrap-up
  - Plain wall verification according to EN 1992-1-1:2004 Chapter 12.6.3
  - Shear design according to EN 1992-1-1:2004 Chapter 6.2.2 & Chapter 6.2.3





Proposal for the shear design of precast walls with plain wall verification

Agenda/ 14:10 – 16:00

- Robustness requirements and solutions:
  - Interpretation of Eurocode provisions
  - Tyings and alternative load paths
  - Key-elements
- Issues related to the design and construction of precast concrete high-rise buildings
- Case studies of high-rise buildings:
  - Breaker Tower Bahrain (165 m)
  - Zalmhaven Tower Rotterdam (215 m)
- Q&A session

Robustness of buildings

What is an Alternative load path?

Activation of ‘catenary action’ as a result of:

- the collapse of a bridge
- a burned telephone pole

Robustness

Regulations in current Eurocodes

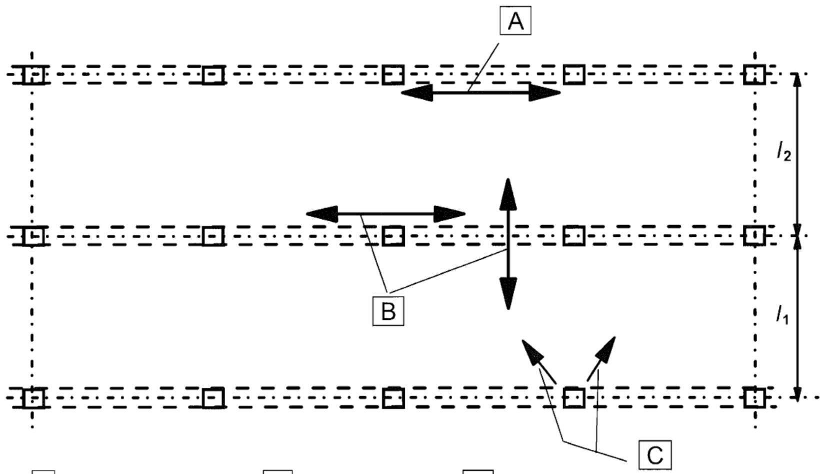
EN 1990 Basis of Design

- Section 2.1

  (4)P A structure shall be designed and executed in such a way that it will not be damaged by events such as :
  – explosion,
  – impact, and
  – the consequences of human errors,
  to an extent disproportionate to the original cause.

  NOTE 2 Further information is given in EN 1991-1-7.

- EN 1992-1-1 Design of Concrete structures, Section 9.10.2 Proportioning of ties



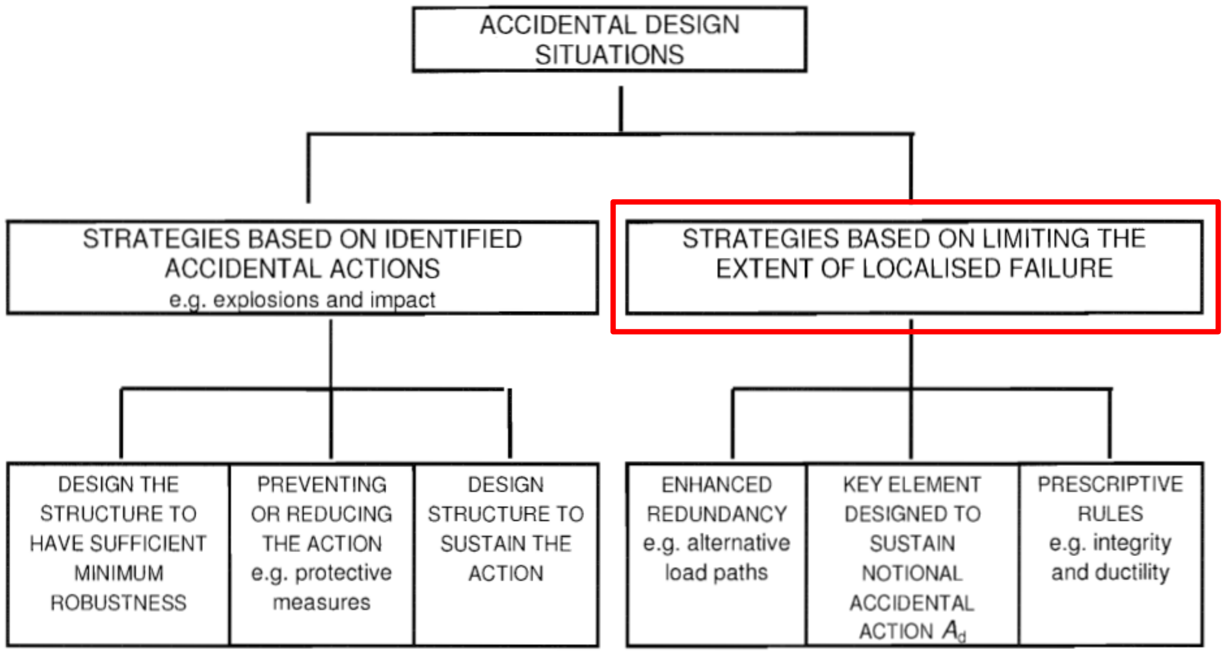
A - peripheral tie B - internal tie C - horizontal column or wall tie

Robustness

Regulations in current Eurocodes

EN 1991-1-7 Accidental Actions:

- known accidental actions
- limiting the extent of localised failure
  - Accidential action unknown
  - Failure from an unspecified cause



EN 1991-1-7, Annex A Design for localised failure from an unspecified cause

- Provides recommended strategies:
  - Horizontal and Vertical ties
  - Alternative Load paths
  - Key elements

Robustness

Regulations in current Eurocodes

EN-1991-1-7 Accidental Actions, Annex A:

- Consequence classes

Application of strategies:

- CC1: no specific consideration
- CC2a: effective horizontal ties
- CC2b: effective horizontal and vertical ties, or
- CC2b: notional removals and alternative load paths
- CC3: systematic risk assessment, alternative load paths, key-elements

Consequence class	Example of categorisation of building type and occupancy
1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of $1\,^1/_2$ times the building height.
2a Lower Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1 000 m^2 floor area in each storey. Single storey educational buildings All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey.
2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m^2 but not exceeding 5000 m^2 at each storey. Car parking not exceeding 6 storeys.
3	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5 000 spectators Buildings containing hazardous substances and /or processes

Robustness

2nd generation Eurocodes

EN 1990 Basis of Design:

- section 4.4 (1) A structure should be designed to have an adequate level of robustness so that during its design service life it will not be damaged by unforeseen adverse events to an extent disproportionate to the original cause.

  NOTE 1 Progressive collapse is an example of a damage that is disproportionate to the original cause.

  NOTE 2 For most structures, design in accordance with the Eurocodes is assumed to provide an adequate level of robustness without the need for any additional design measures to enhance structural robustness.

EN 1991-1-7 Accidental Actions

- Preface: Design methods for Robustness are moved to EN 1990
- Scope: Accidental Actions only (normative)
- Annex A approximately the same as 1st generation version (informative)

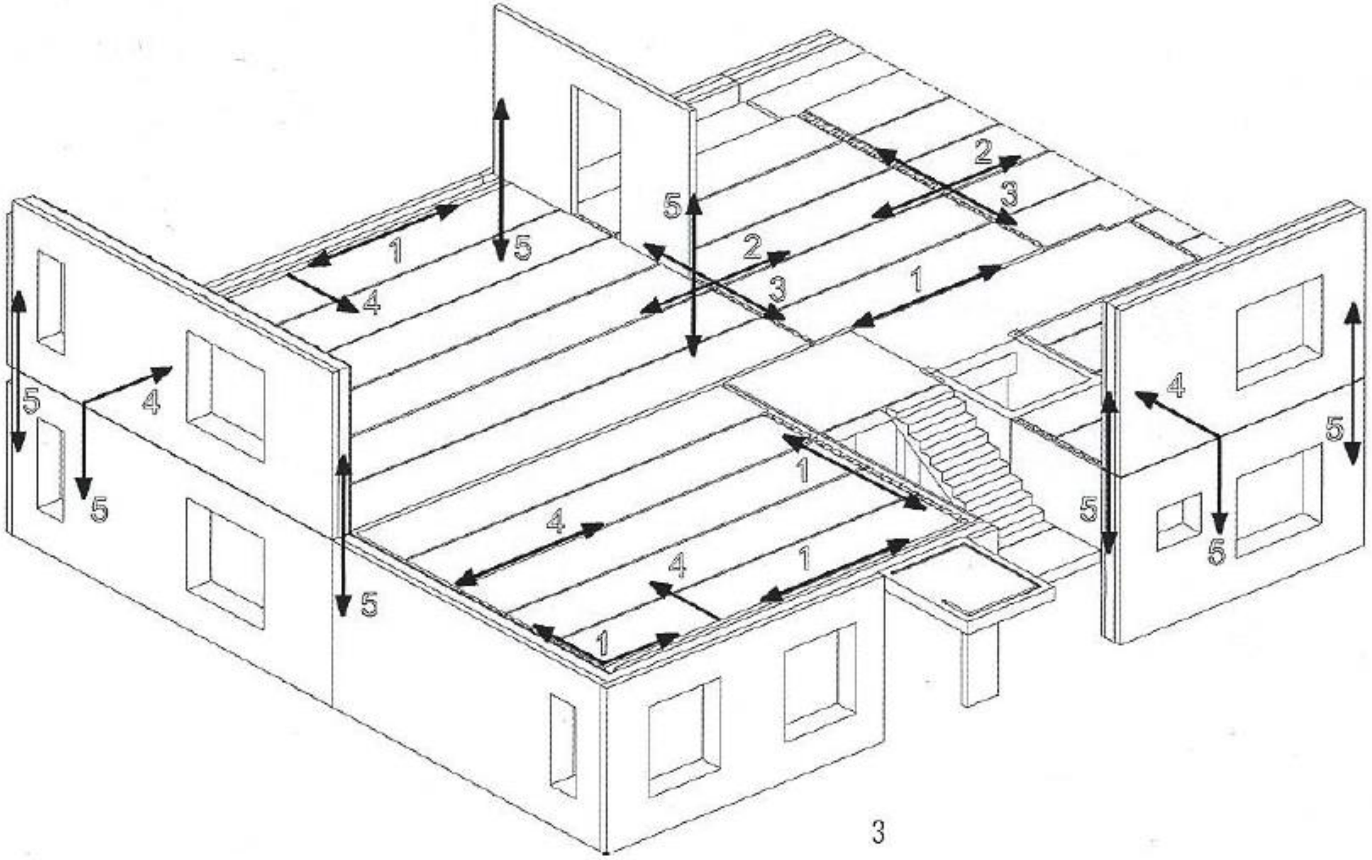
Agenda/ 14:10 – 16:00

- Robustness requirements and solutions:
  - Interpretation of Eurocode provisions
  - Tyings and alternative load paths
  - Key-elements
- Issues related to the design and construction of precast concrete high-rise buildings
- Case studies of high-rise buildings:
  - Breaker Tower Bahrain (165 m)
  - Zalmhaven Tower Rotterdam (215 m)
- Q&A session

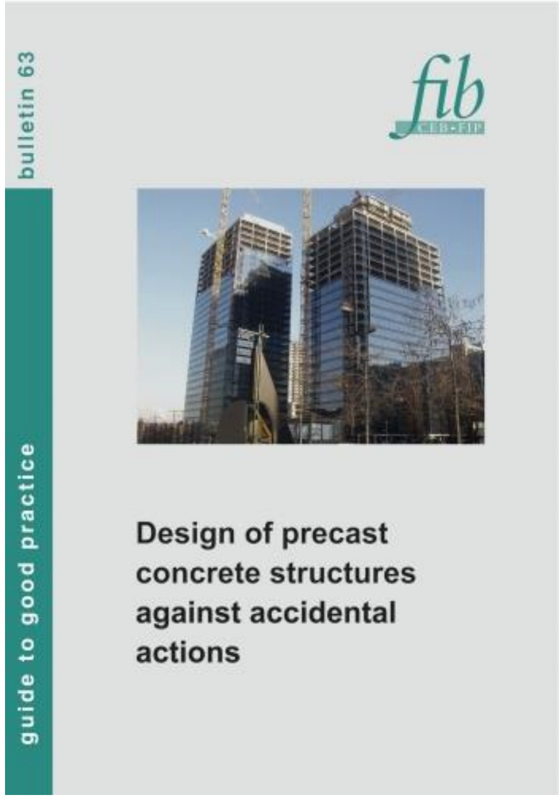
Robustness

Ties for enhancing the robustness of the building structure

- Literature
- Ties in precast concrete slabs / wall structures



- Peripheral ties
  1. Peripheral floor ties
- Internal ties
  2. Longitudinal floor ties
  3. Tansversal floor ties
- Horizontal ties to columns and walls
  4. Wall to floor ties
- Vertical ties
  5. Vertical wall ties

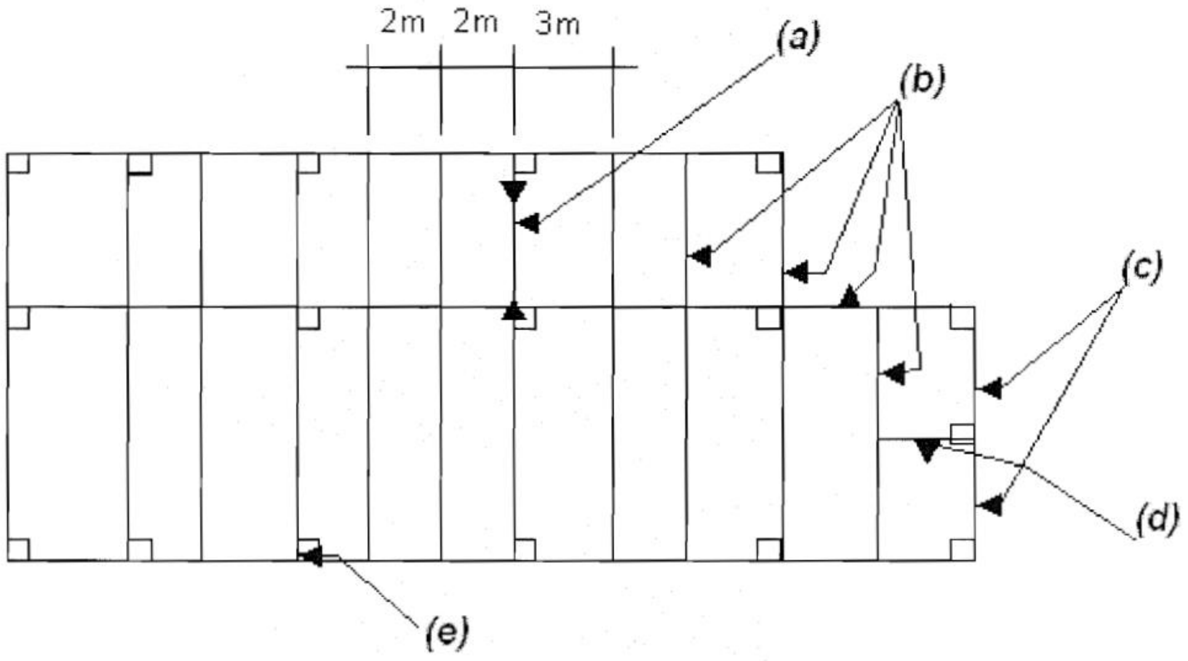


Robustness

Distinguishes the strategies for reinforcement Ties:

- Indirect approach: Enhance the Robustness with ties calculated from design equations
- EN 1991-1-7, Annex A presents such equations
- It's called the Tie-Force method
- Real Alternative load paths are not structurally considered with an indirect approach

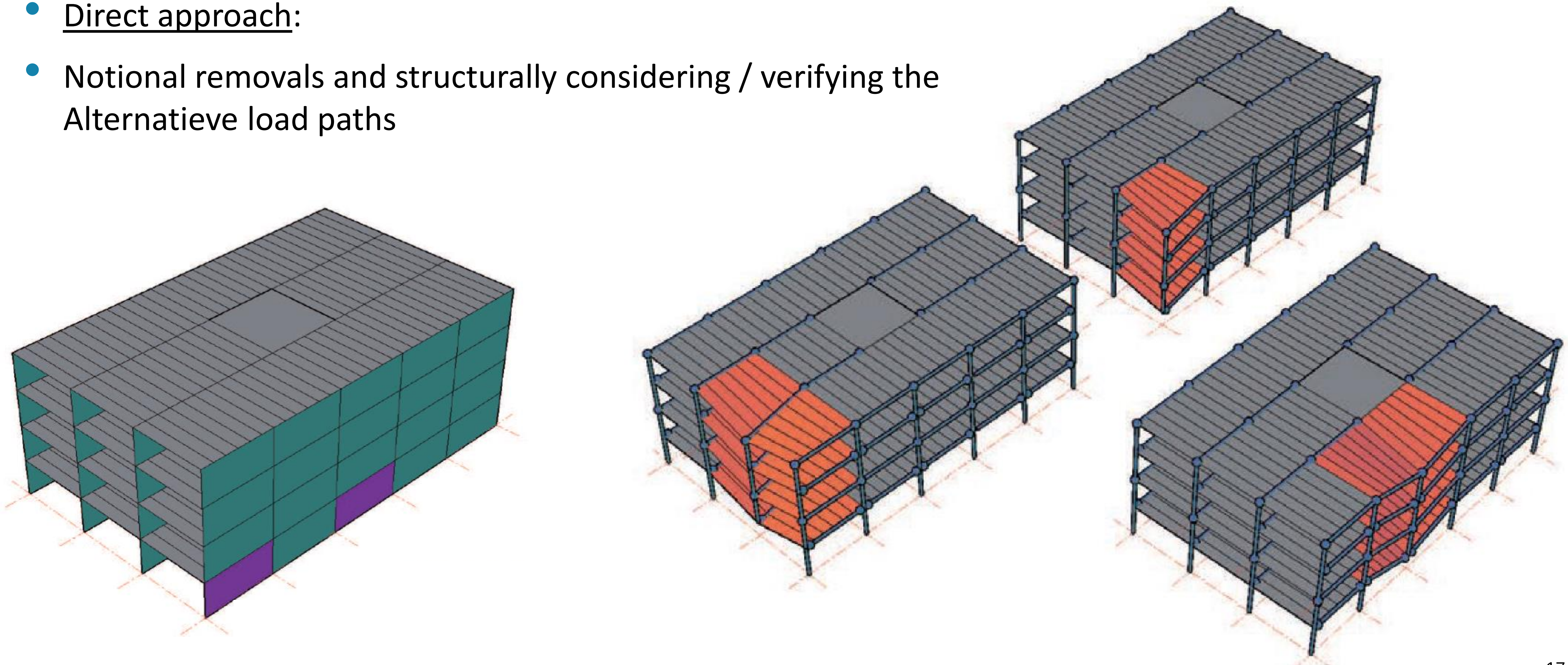
$$T_i = 0.8\,(g_k + \psi q_k)\,s\,L$$
$$T_p = 0.8\,(g_k + \psi q_k)\,s\,L$$



Robustness

Distinguishes the strategies for reinforcement Ties:

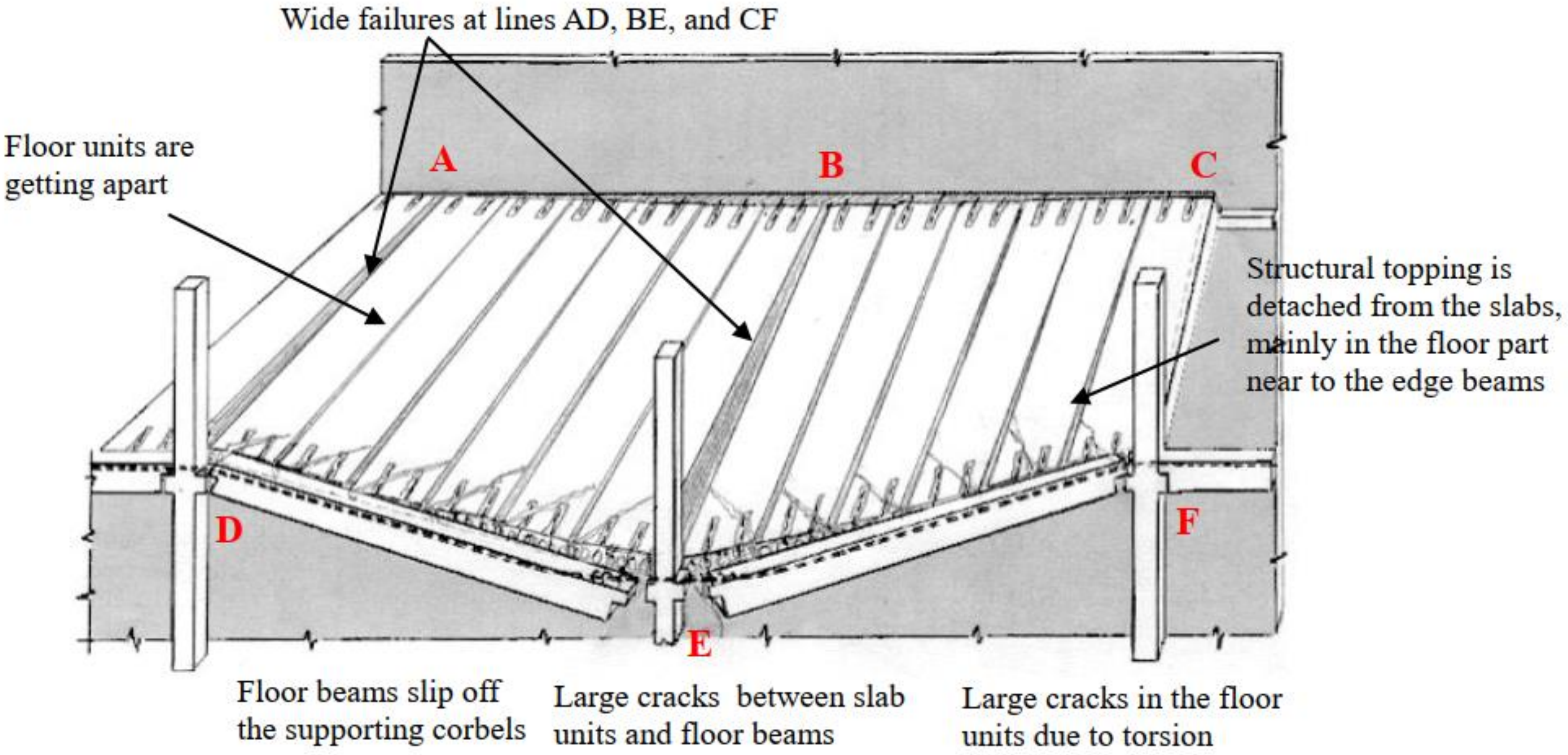
- Direct approach:
- Notional removals and structurally considering / verifying the Alternatieve load paths



Robustness

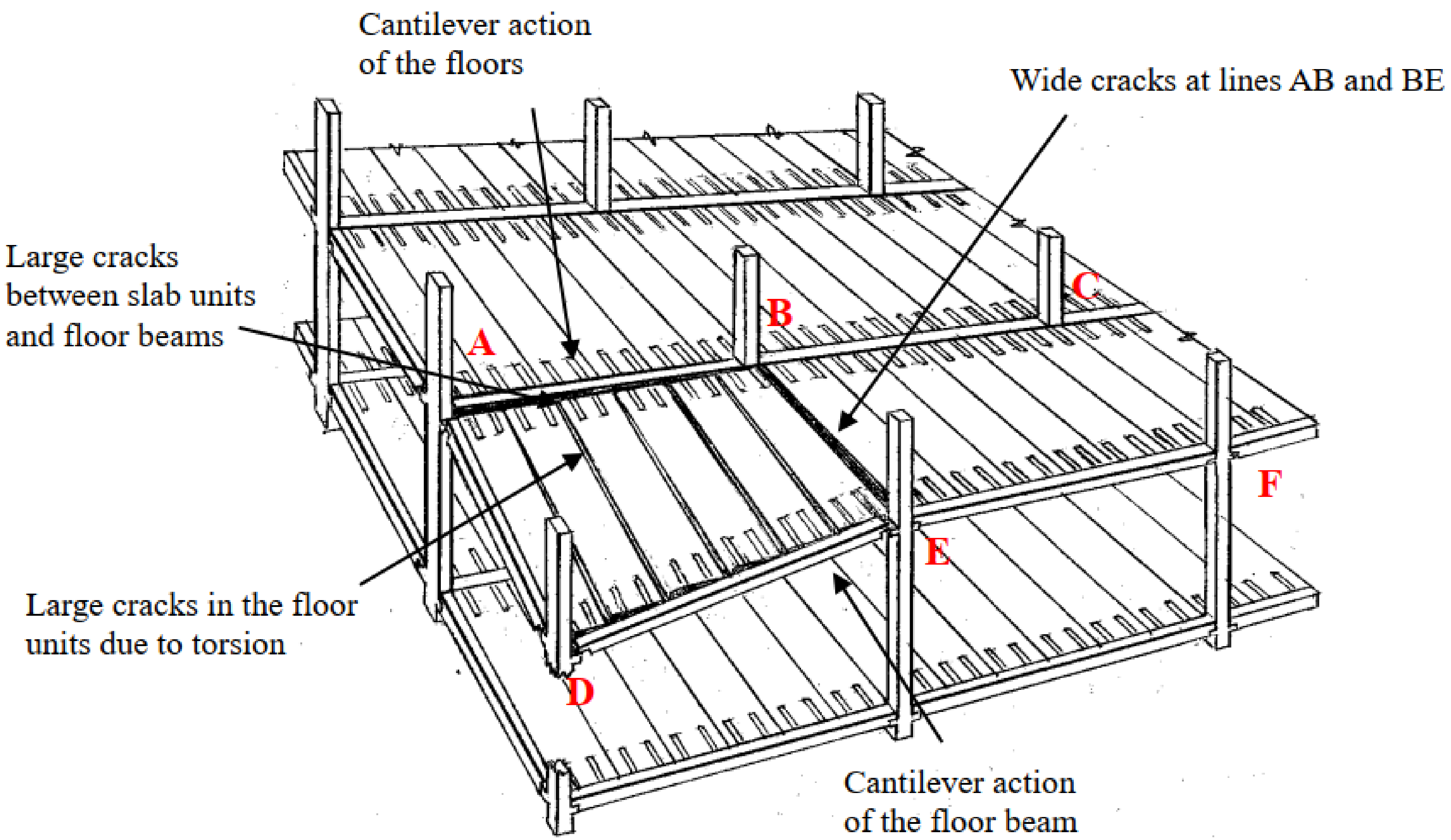
Alternative Load Paths:

- Effects of Notional removals in precast concrete HCS – Beam structures



Alternative Load Paths:

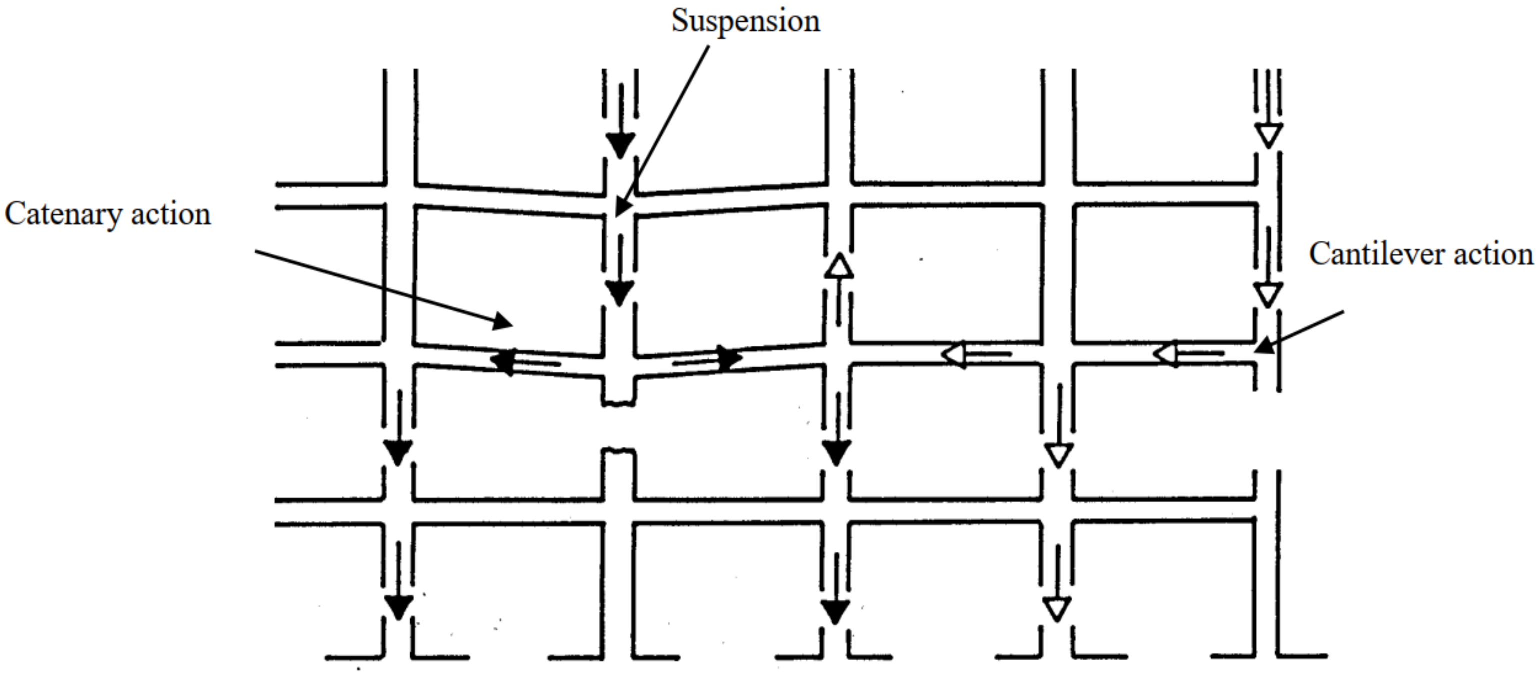
- Effects of Notional removals in precast concrete HCS – Beam structures

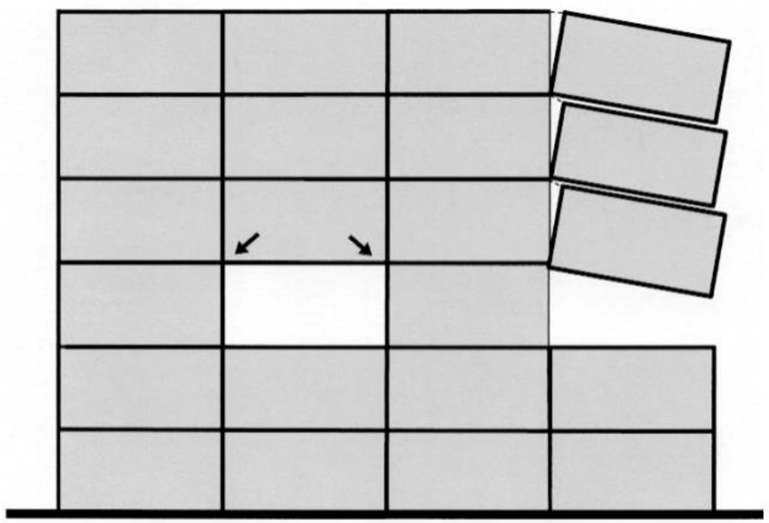


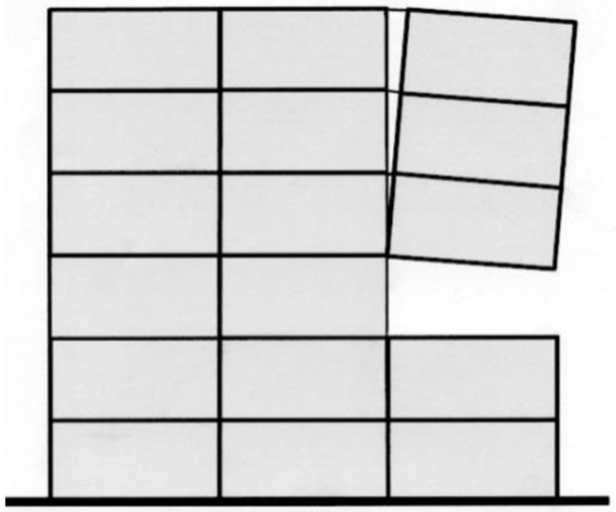
Robustness

Bearing mechanisms for Alternative Load Paths:

- Catenary action of the tie reinforcement
- Cantilever action
- Suspension of vertical elements
- Membrane actions of floors and the roof



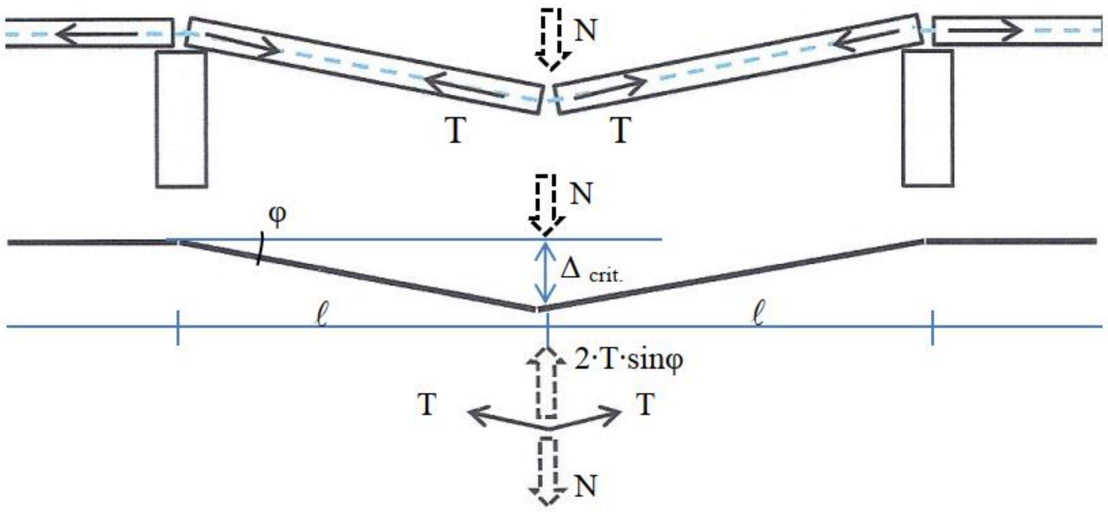




Robustness

Catenary action of tie reinforcement:

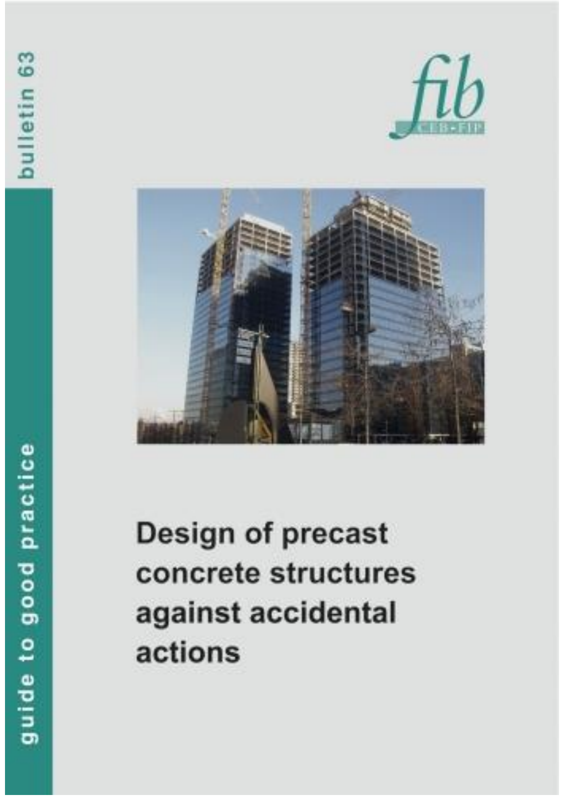
- Fib bulletin 63 provides practical linear analyses:



Equilibrium of forces gives:

$$2 \cdot T \cdot \sin\varphi = N \qquad \text{and,}$$

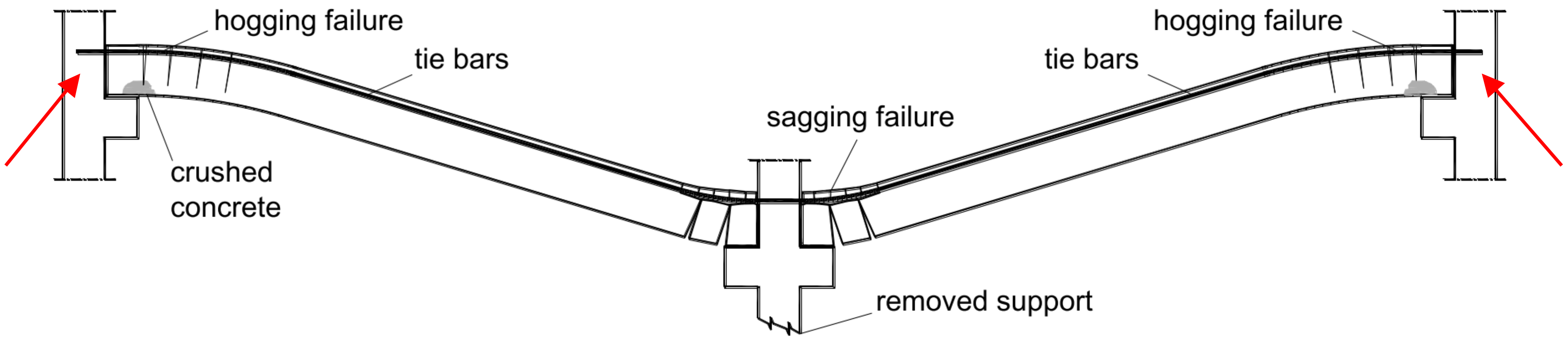
$$T = \frac{N}{2 \cdot \sin\varphi} = \frac{N}{2} \cdot \frac{\sqrt{\ell^2 + \Delta_{cr}^2}}{\Delta_{cr}} = \frac{N}{2}\sqrt{\left(\frac{\ell}{\Delta_{cr}}\right)^2 + 1}$$

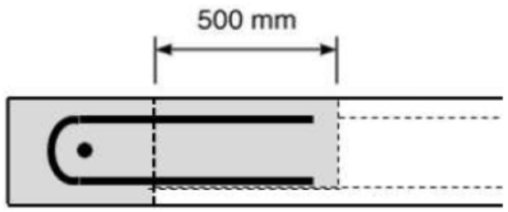


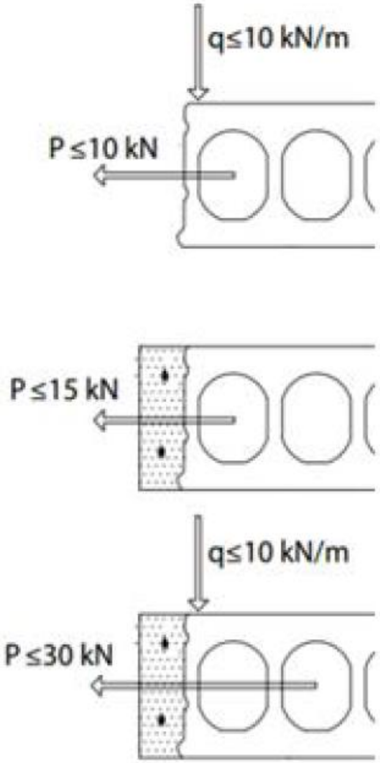
Robustness

Catinary action of tie reinforcement:

- Ties require to be anchored



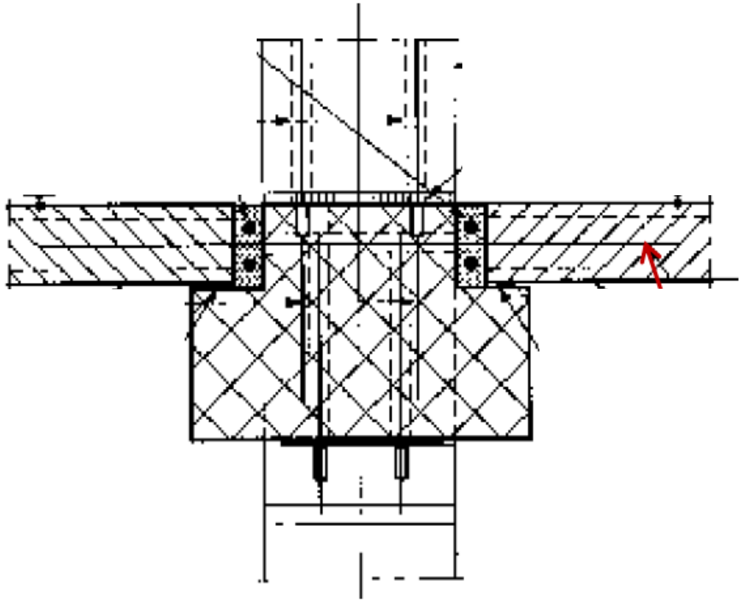




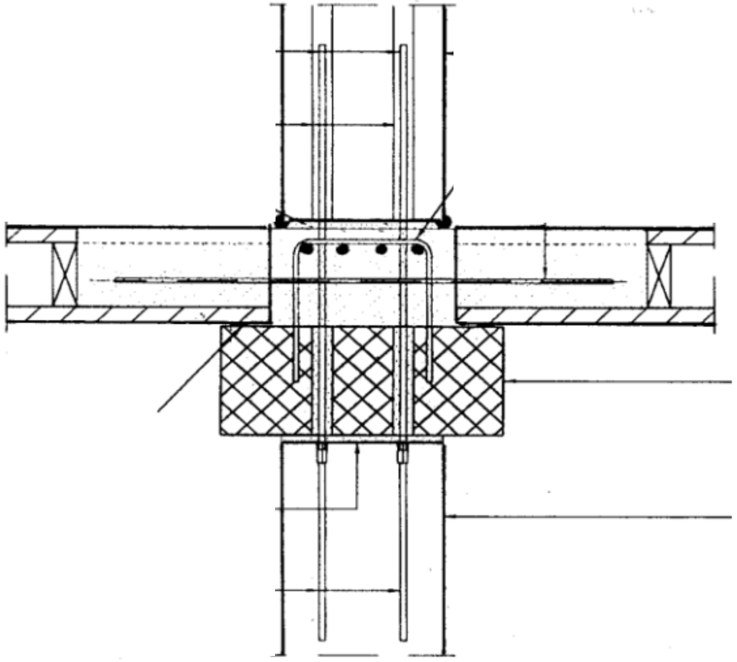
Robustness

Catinary action of tie reinforcement:

- Position of the Ties
- Apply the Ties under and not next to the column for the most effective functioning



less good solution

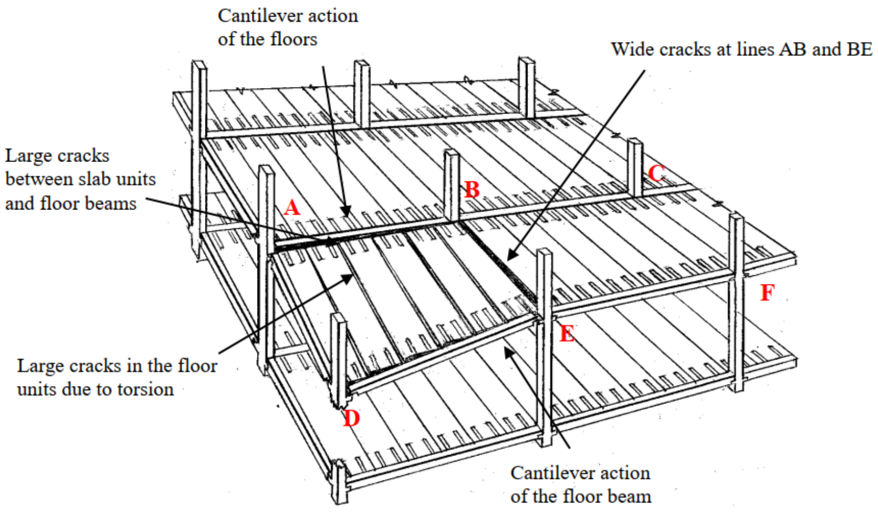


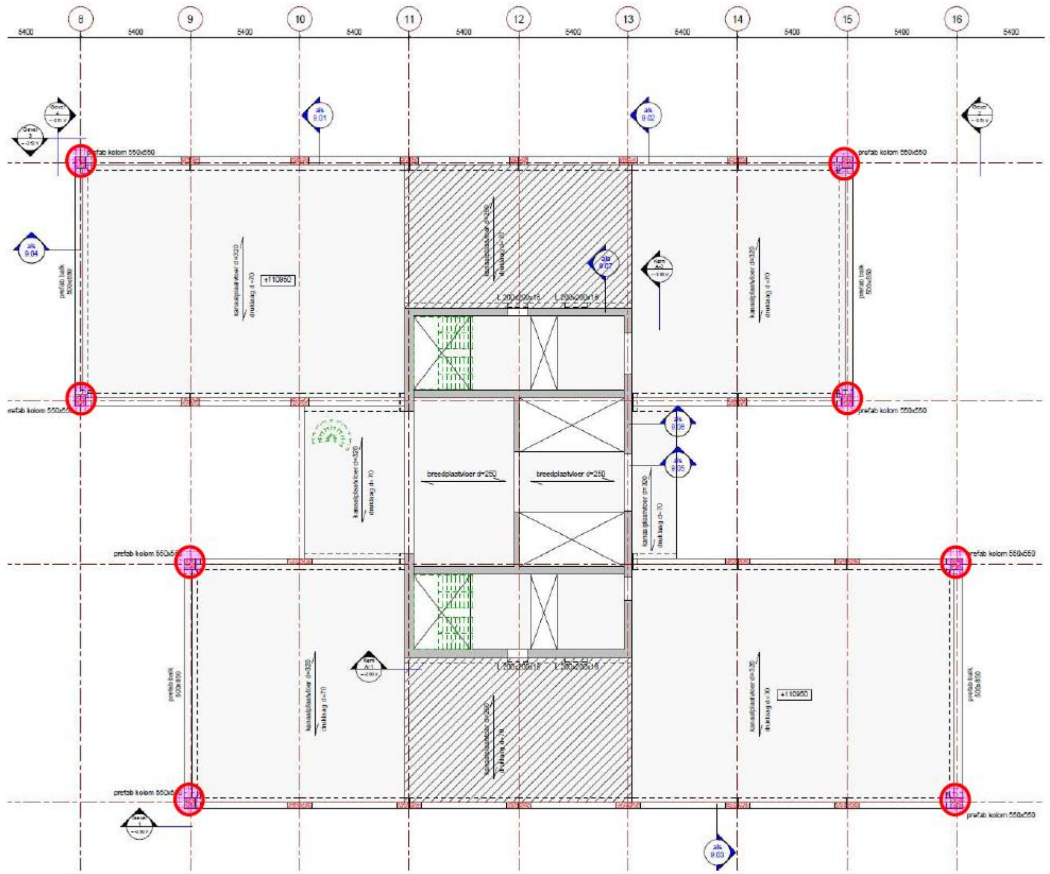
good solution

Robustness

Key-elements for columns:

- Corner column cannot be kept upright with ties
- Only with disproportionately large measures
- 34 kN/m^2 is inappropriate measure (belongs to the gas explosion Ronan Point Building UK which is a known structural load)
- Additional 20% on ULS normal forces and moment of columns





Agenda/ 14:10 – 16:00

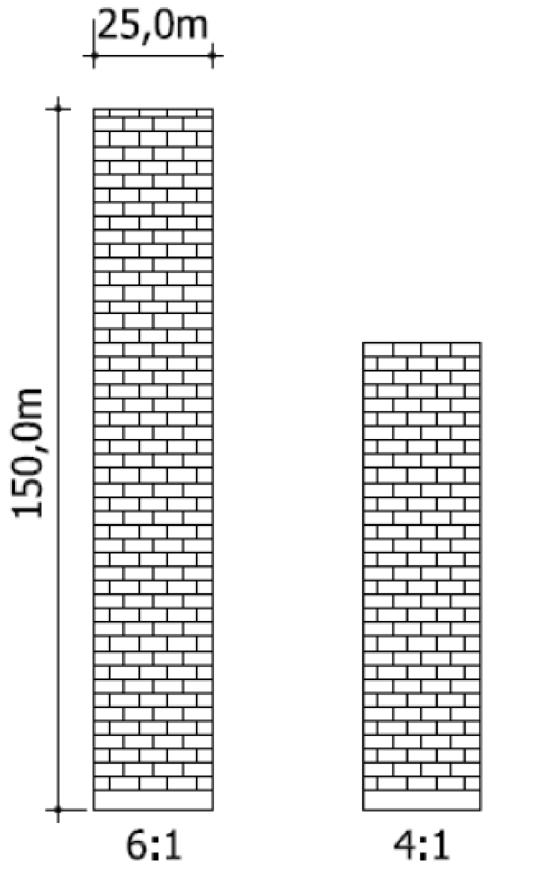
- Robustness requirements and solutions:
  - Interpretation of Eurocode provisions
  - Tyings and alternative load paths
  - Key-elements
- Issues related to the design and construction of precast concrete high-rise buildings
- Case studies of high-rise buildings:
  - Breaker Tower Bahrain (165 m)
  - Zalmhaven Tower Rotterdam (215 m)
- Q&A session

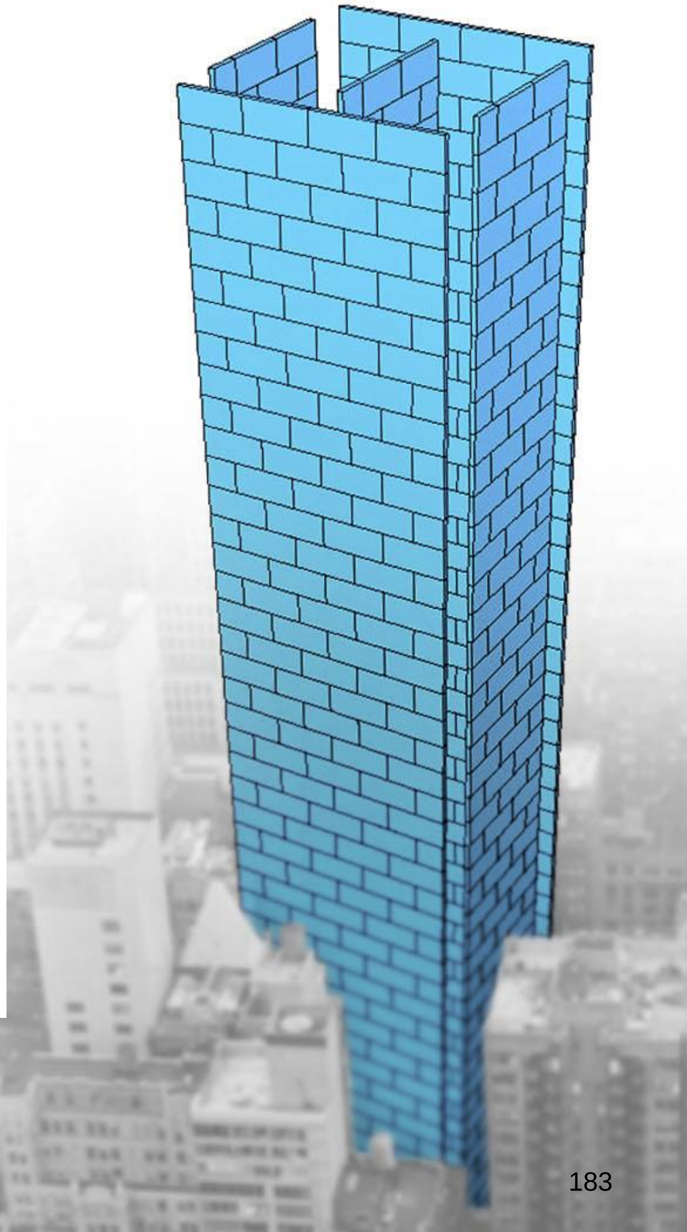
Design and Construction of Precast Concrete High-rise buildings

Definition of a high-rise building:

- When is a building considered to be a high-rise?
- Aspect ratio: “height divided by plan length”
- Often quoted: greater than 5 → Considere tall

- It’s not generally simple as that!





Design and Construction of Precast Concrete High-rise buildings

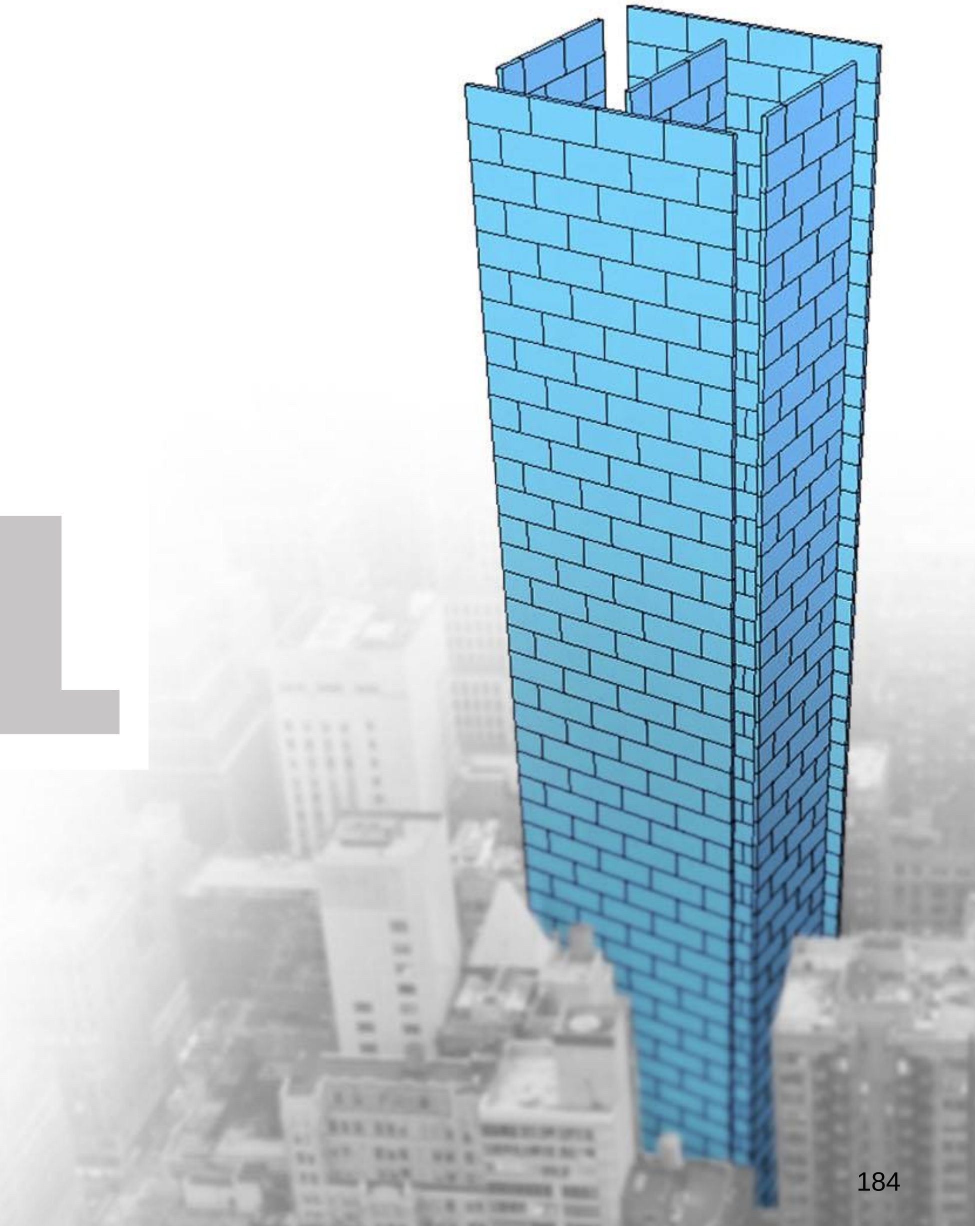
Counsel on Tall Buildings and Urban Habitat (CTBUH)

- Cat. A: Heigth relative to context

14 storey "tall building"
in provincial location

14 storey building amidst
"tall buildings"

- Cat. B: Proportion
- Cat. C: Different technoligies relevant to "tallness"



Design and Construction of Precast Concrete High-rise buildings

Counsel on Tall Buildings and Urban Habitat (CTBUH)

- Cat. A. Heigth relative to context
- Cat. B: Proportion

Slender building in urban landscape

Large footprint building

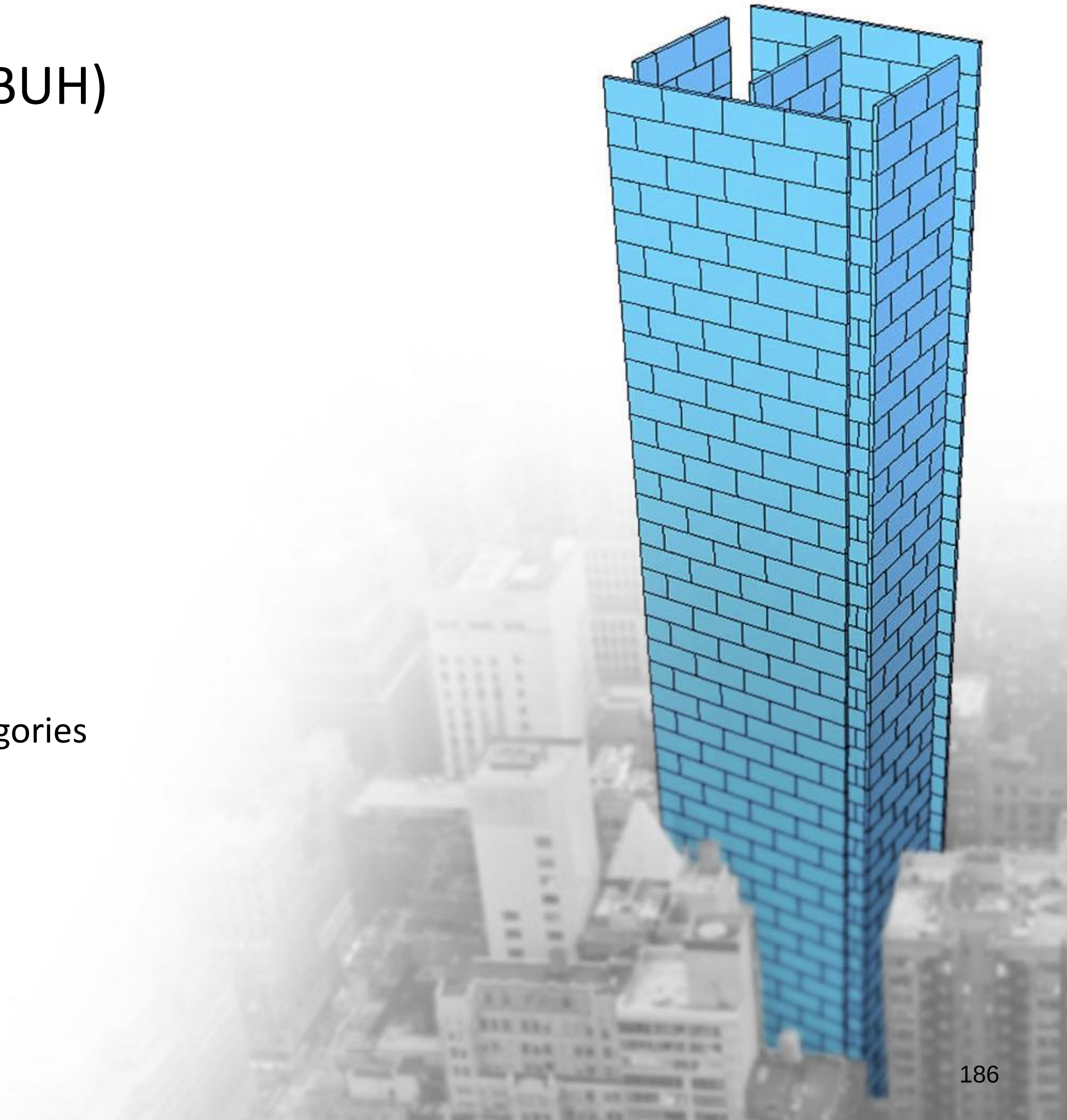
- Cat. C: Different technoligies relevant to “tallness”

Design and Construction of Precast Concrete High-rise buildings

Counsel on Tall Buildings and Urban Habitat (CTBUH)

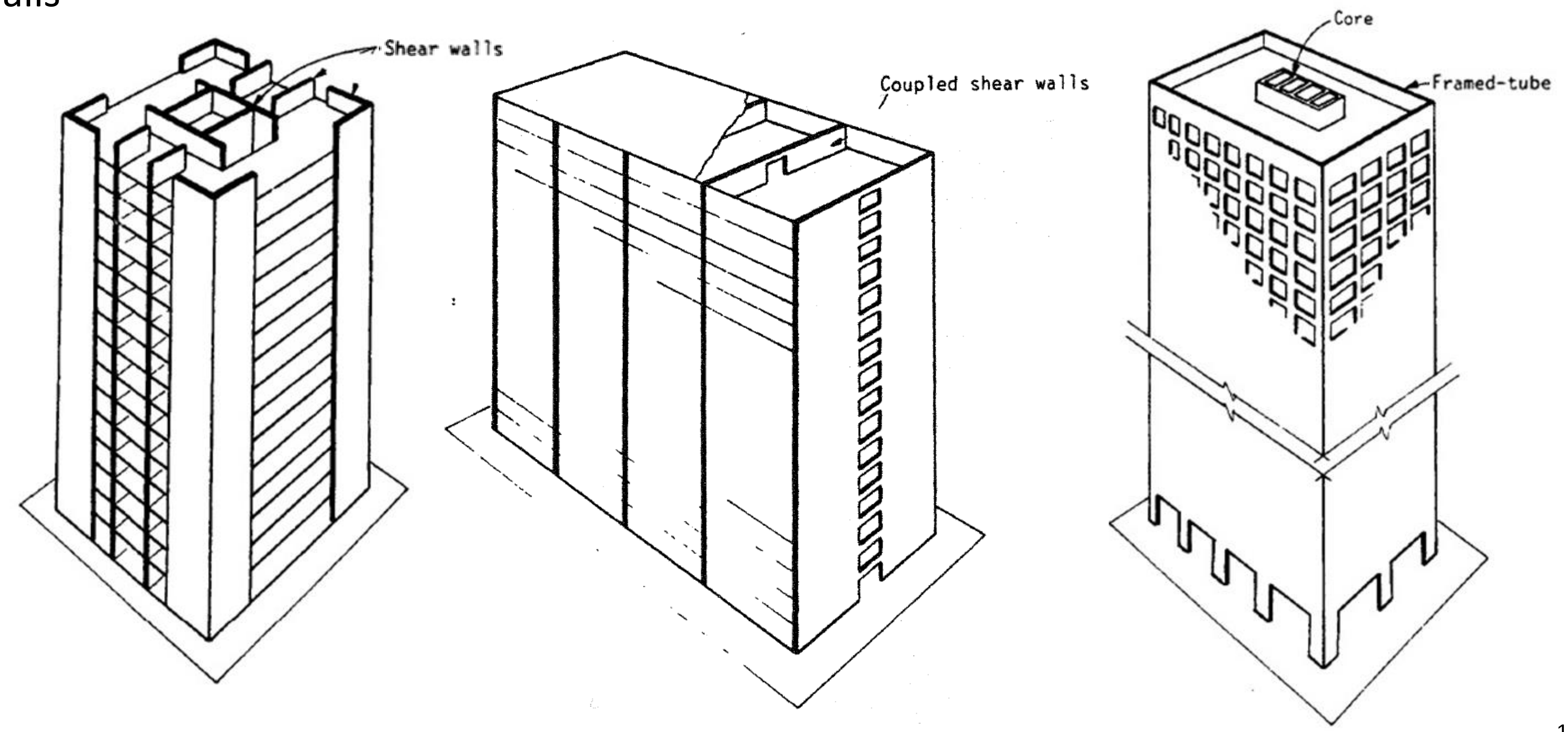
- Cat. A. Heigth relative to context
- Cat. B: Proportion
- Cat. C: Different technologies relevant to “tallness”
  - Vertical transportation technologies
  - Special windbracing systems

- CTBUH suggests:
  - 14 or more storeys, or over 50 meters high, subject to the categories described above

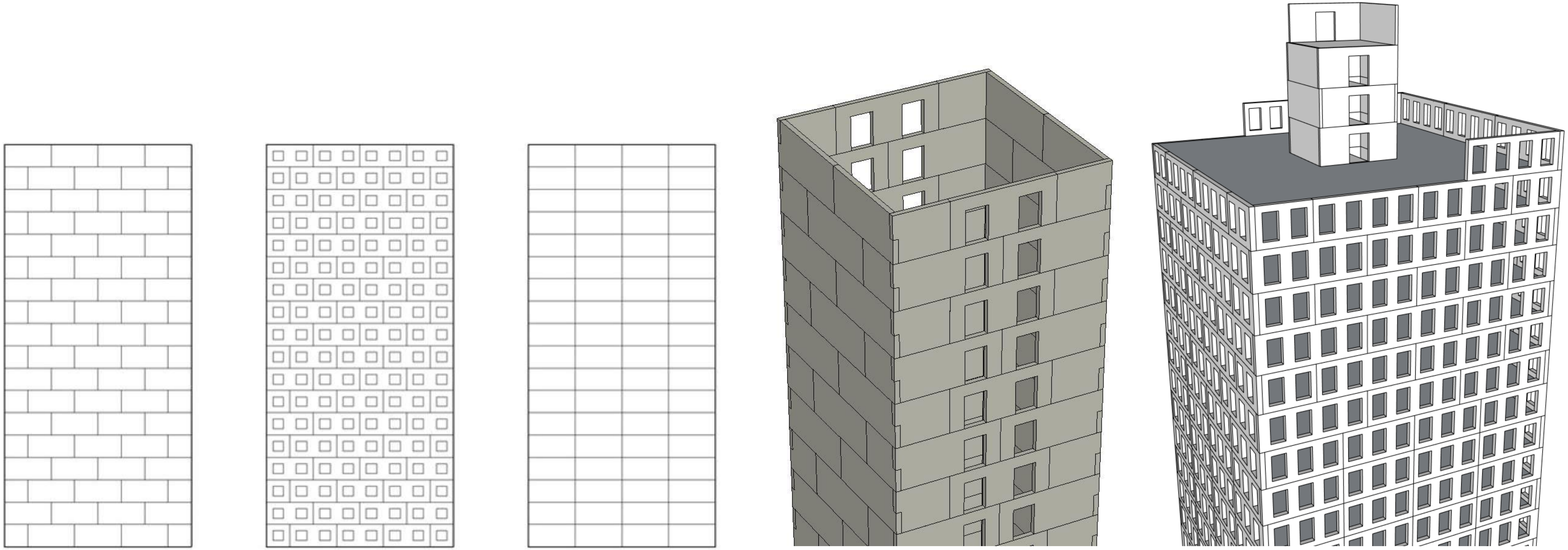


Tall Building Structural Forms related to precast concrete

- Shear walls
- Coupled shear walls
- Cores
- Framed tubes
- Tube-in-tubes

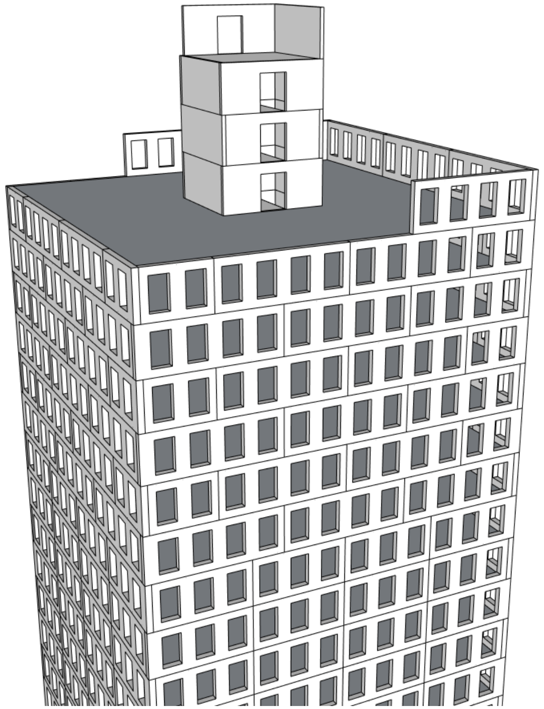


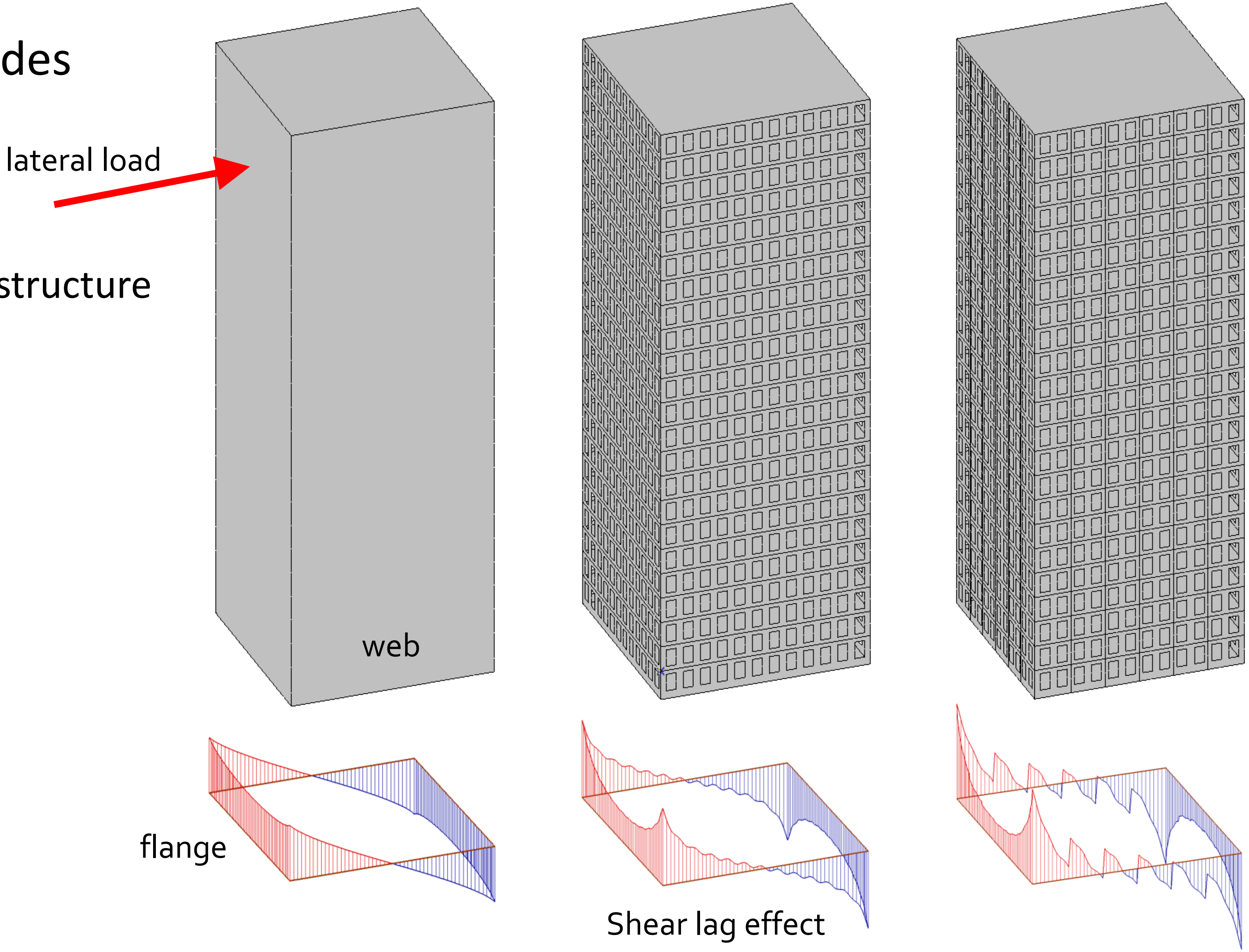
Precast Concrete element lay-outs



Structural behaviour of façades

- Tube structure
- Framed tube structure
- Precast concrete framed tube structure

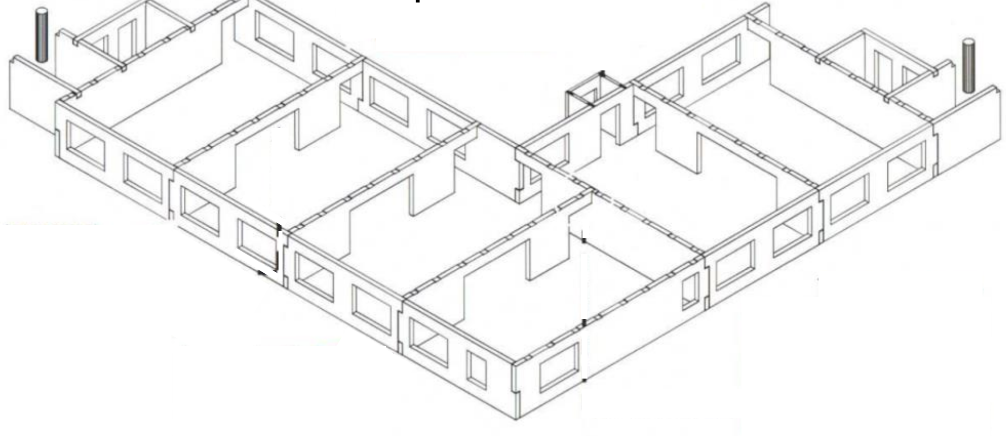


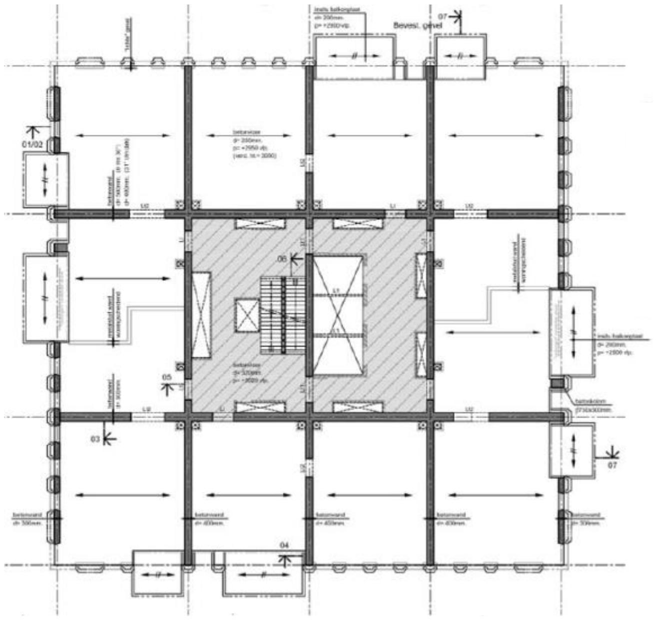


Design and Construction of high-rise buildings

Tall Building Structural Forms

Coupled shear walls





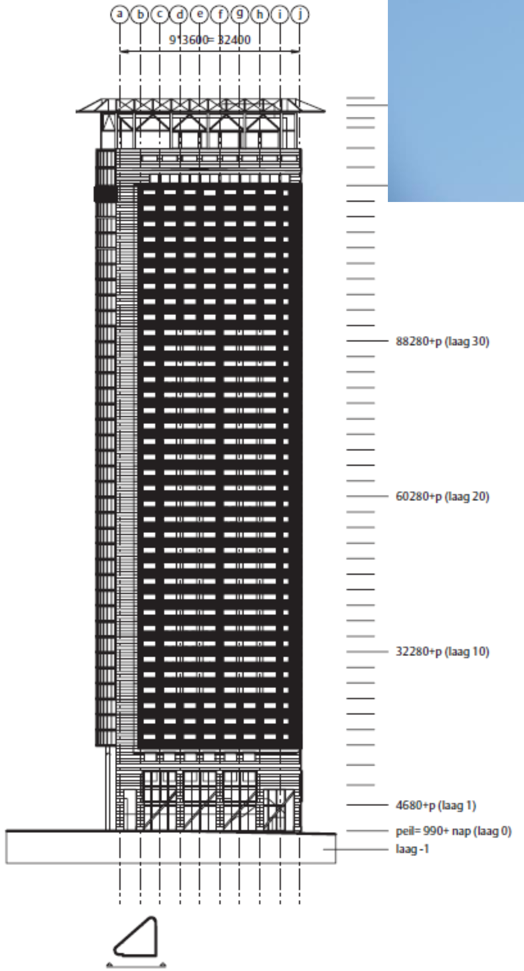
Shear walls

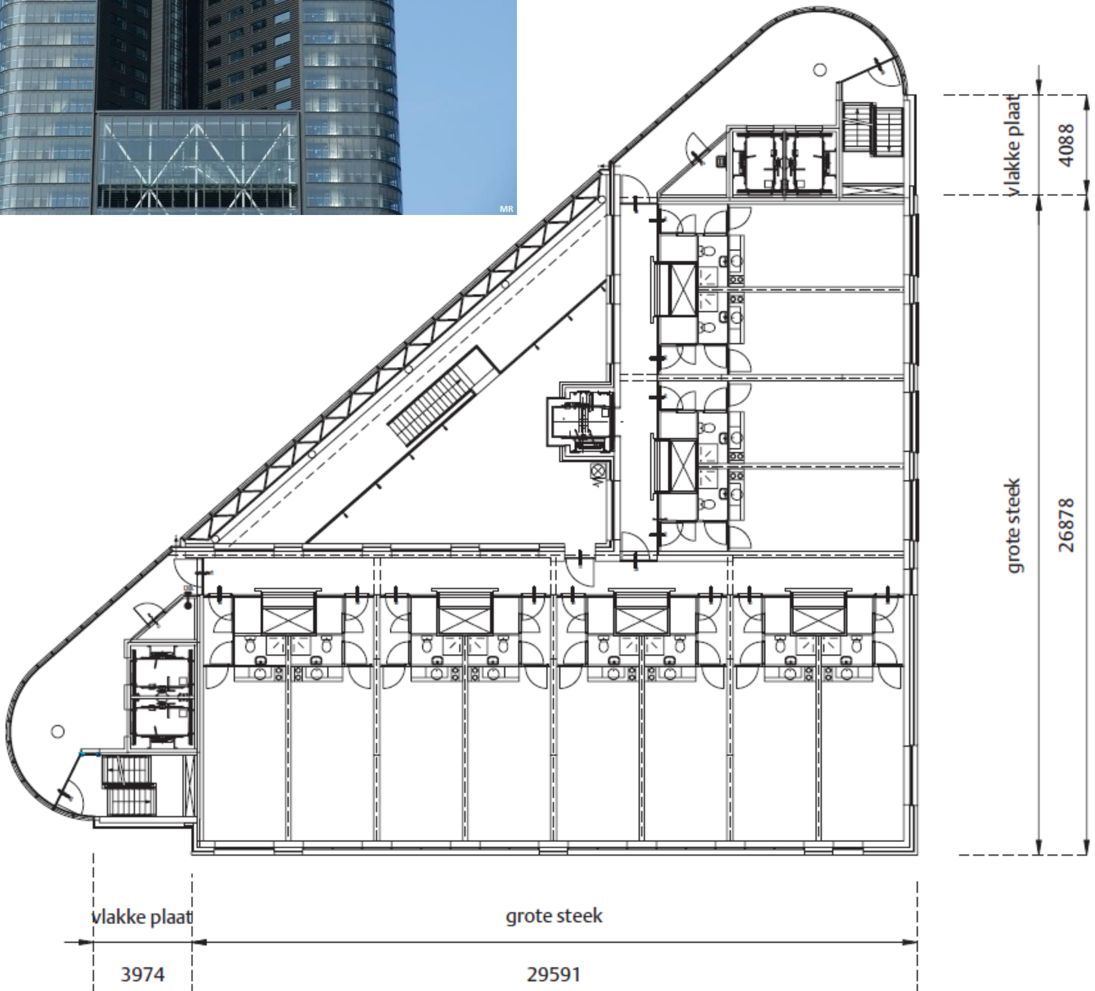
Cores / framed-tube

Design and Construction of high-rise buildings

Project: Het Strijkijzer – 135 meters

- Translation of the name: The Iron

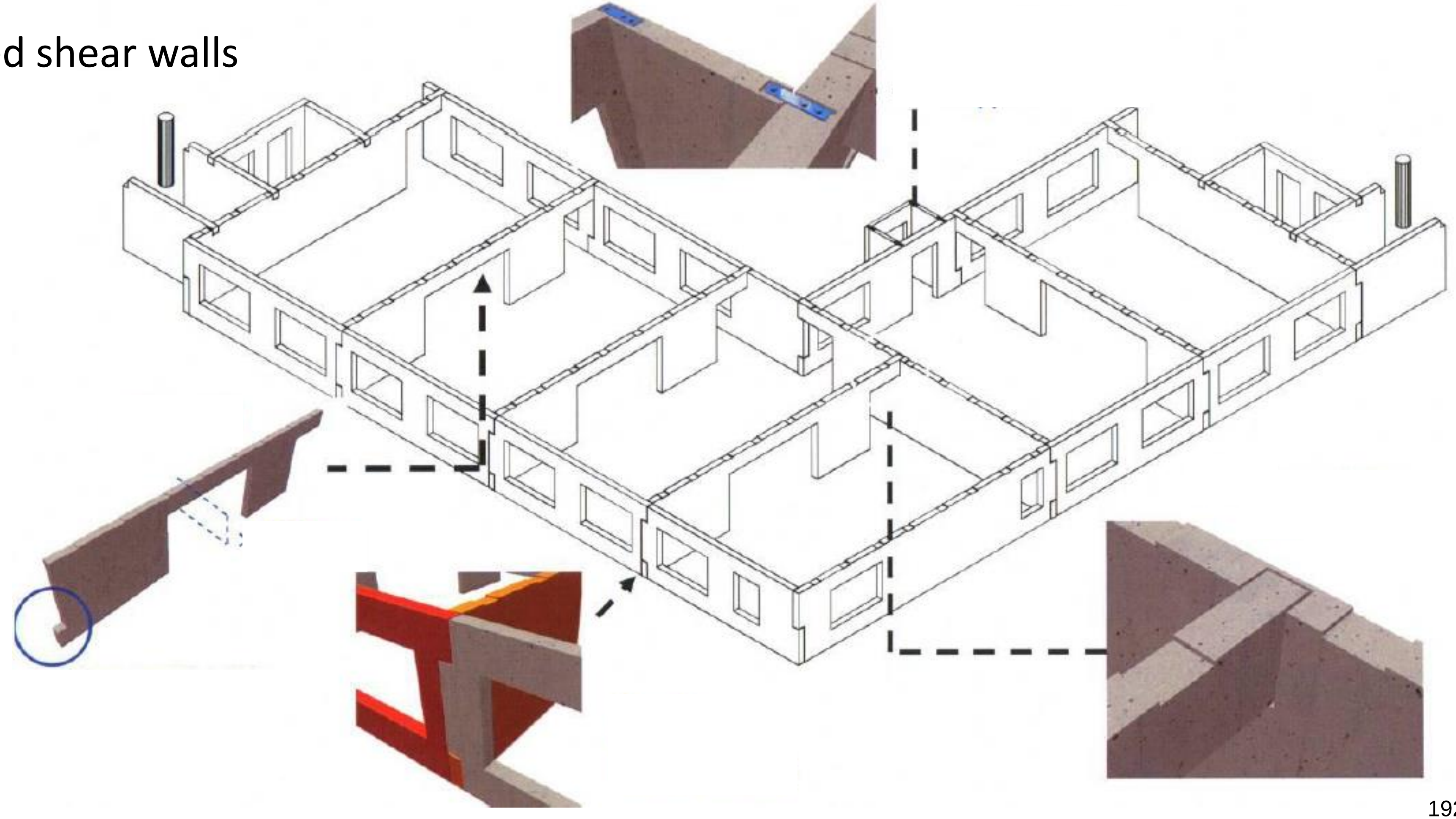




Design and Construction of high-rise buildings

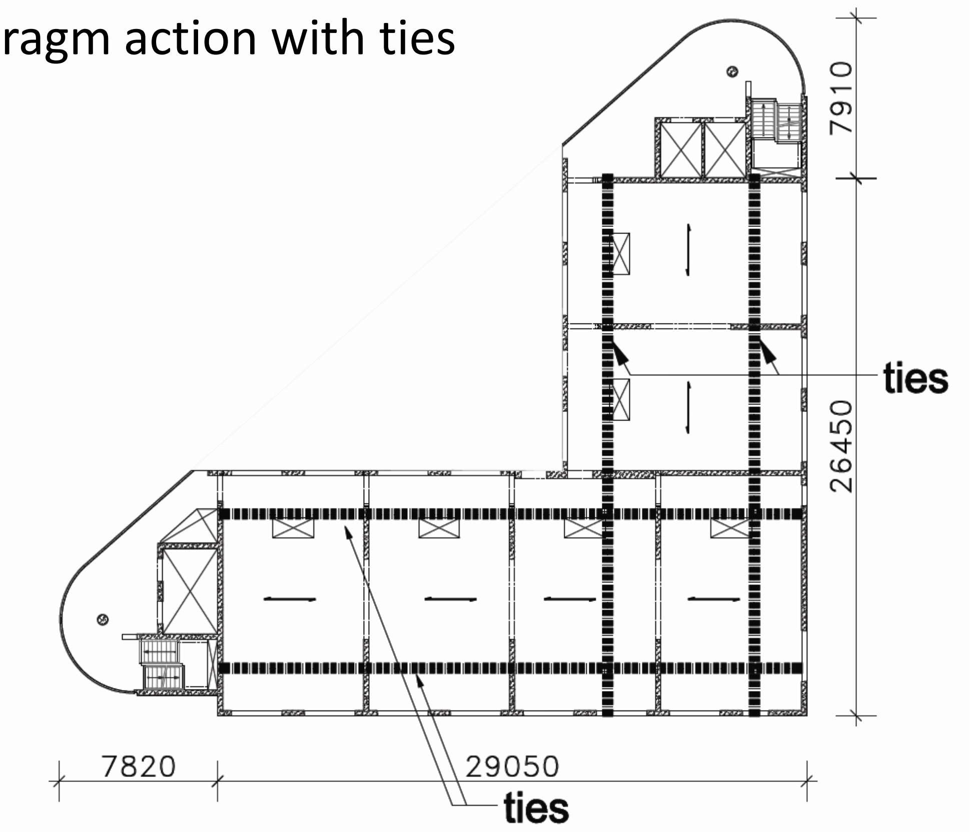
Project: Het Strijkijzer – 135 meters

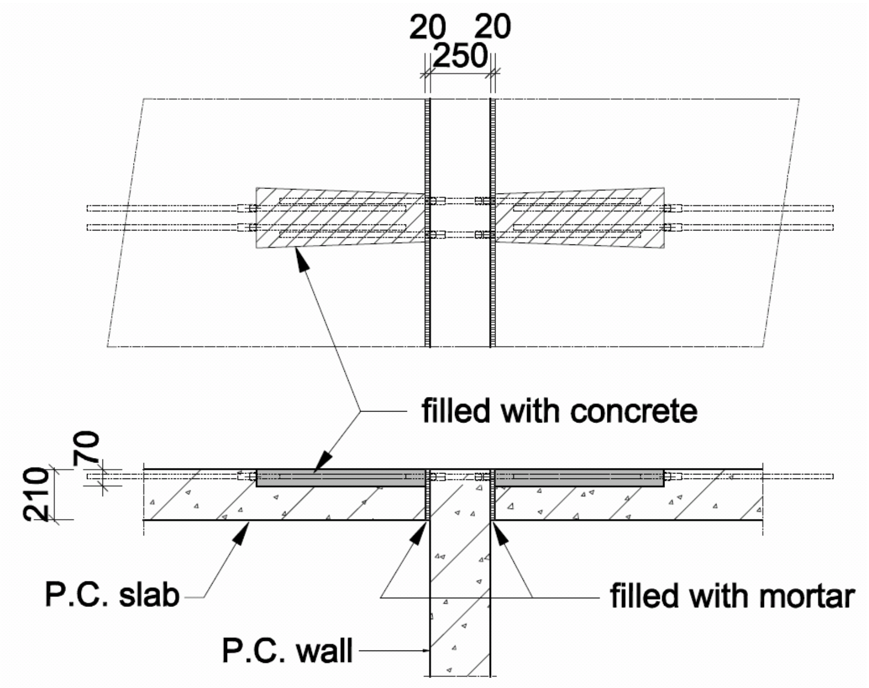
- Precast concrete design
- Framed Tube with Coupled shear walls



Project: Het Strijkijzer – 135 meters

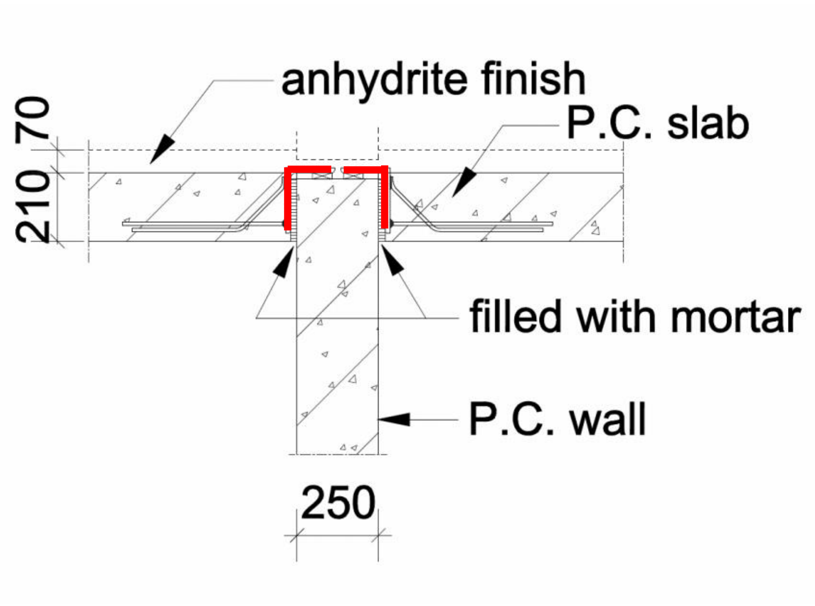
- Precast concrete slab elements
- Diaphragm action with ties





Project: Het Strijkijzer – 135 meters

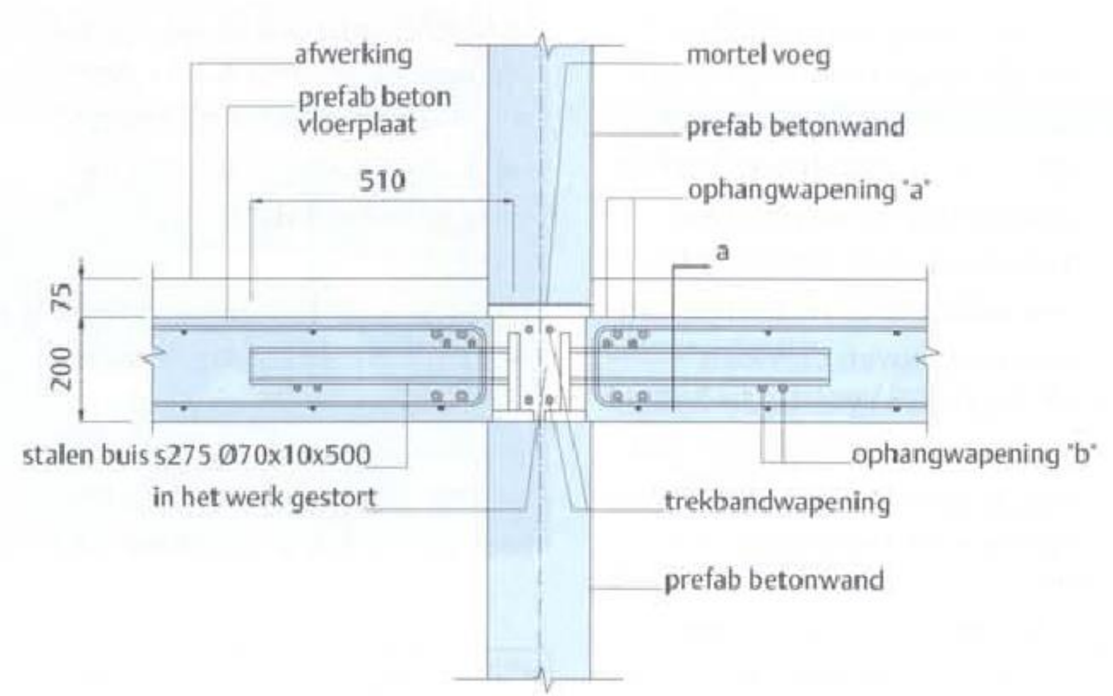
- Precast concrete slab with steel support



Design and Construction of high-rise buildings

Project: Het Strijkijzer – 135 meters

- Alternative steel support



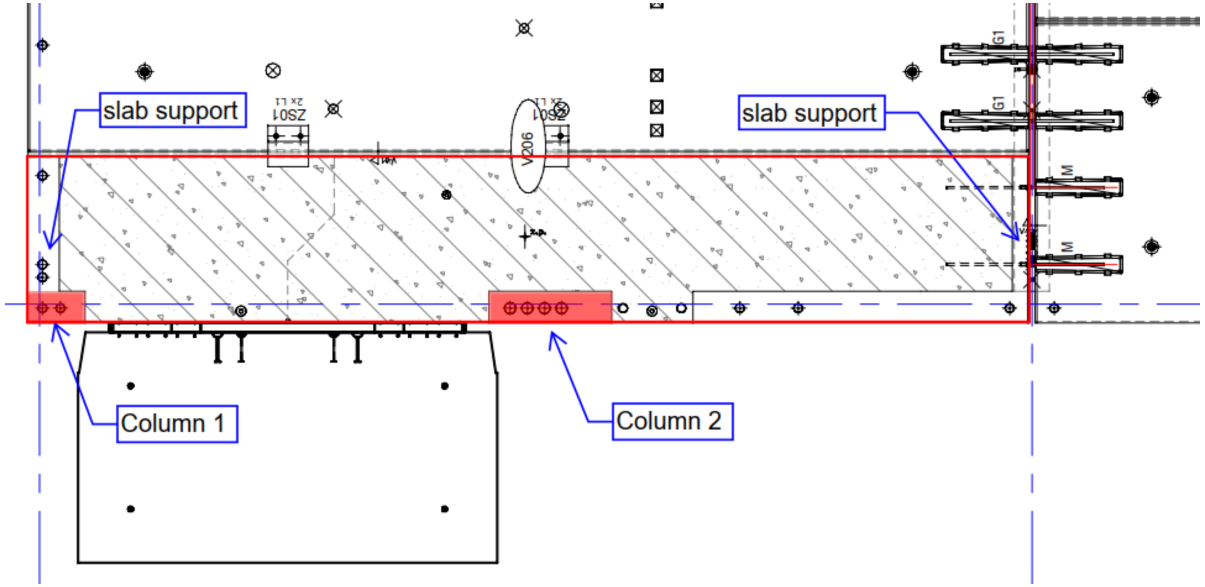
Design and Construction of high-rise buildings

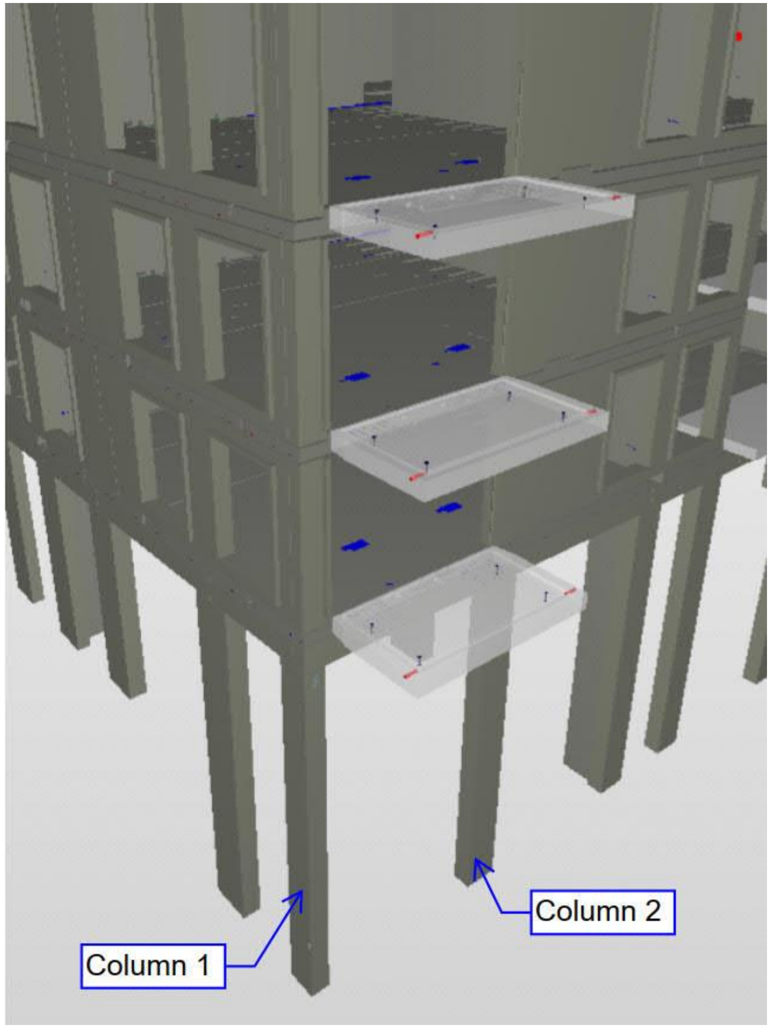
Project: Het Strijkijzer

- Construction

Project: Maanplein 110

- Slab + Balcony assembly hoisted as one component
- Laid down on ‘slab supports’ (column 2 not a support for Self Weight)
- Column 2 become active only for additional (e.g. finish and live) loads
- Column 1 loaded with higher normal force than probably expected due to precast concrete construction
- Normal forces in columns change by creep and displacements effects

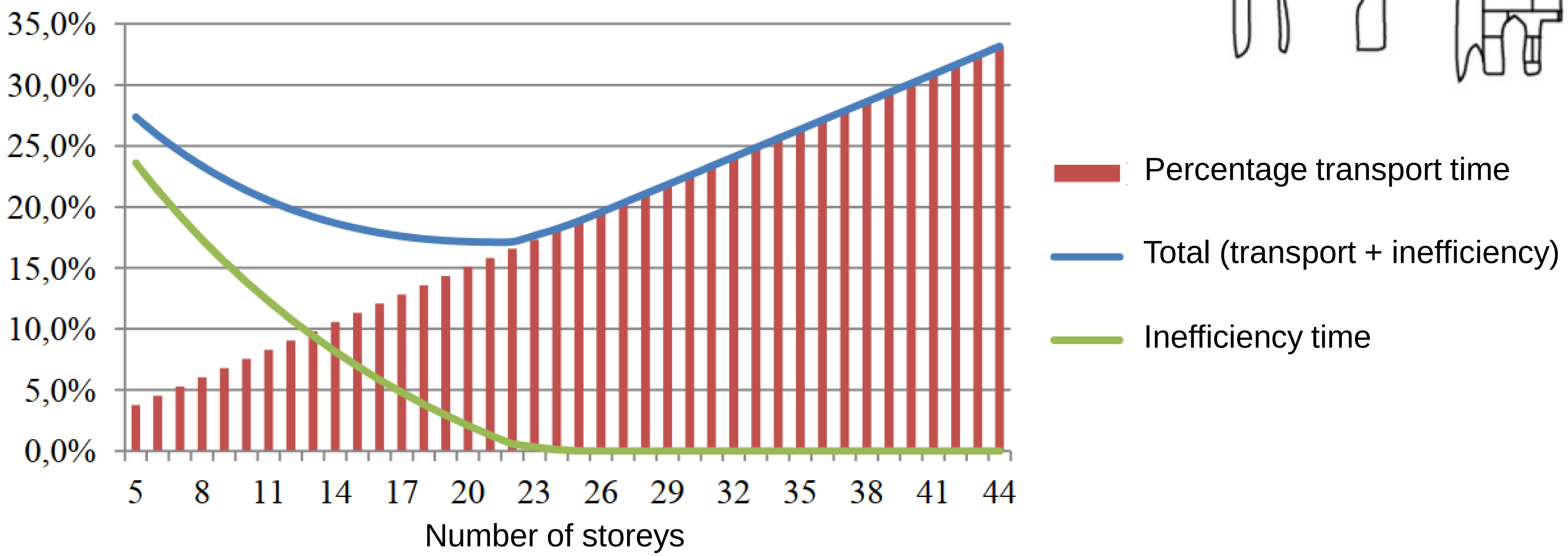


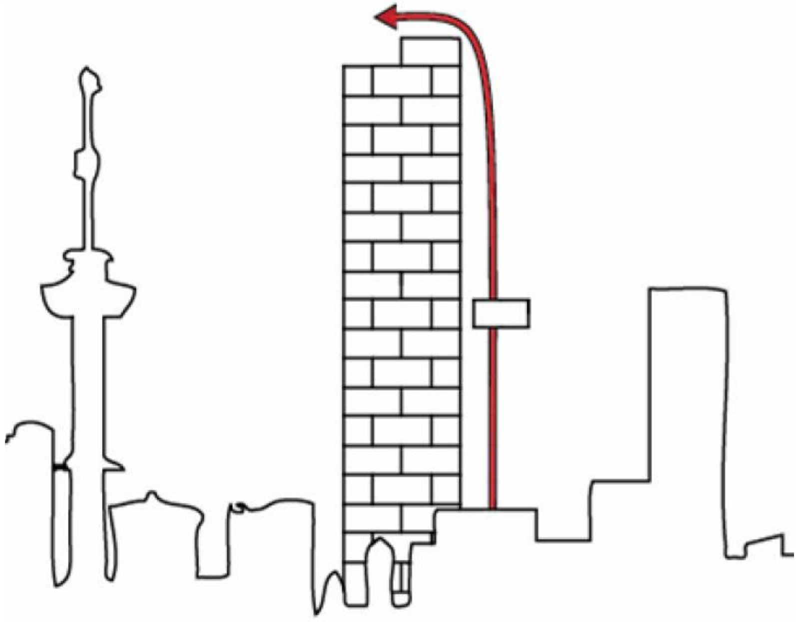


Design and Construction of high-rise buildings

Precast concrete construction

- Logistics control vital for the success of a business case
- Small construction site
- Increase of vertical transportation time
- Development of ‘Standard Time’ elements Het Strijkijzer:

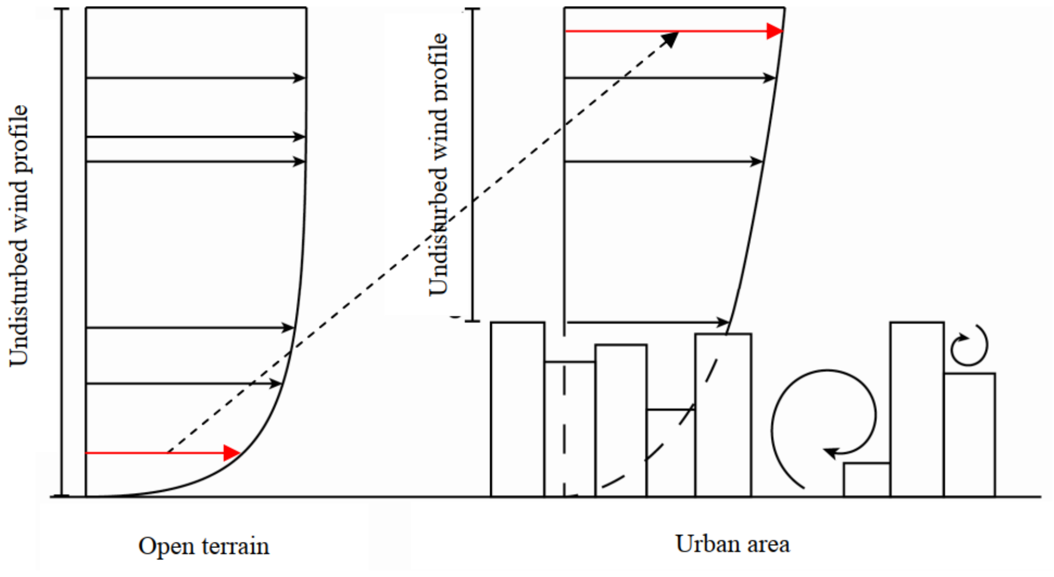




Design and Construction of high-rise buildings

Precast concrete construction:

- Weather conditions: effect of wind and height
- Critical wind speed of 11 m/s
- Forecast calender with an increasing building height
- Yearly reduction of workable days: 6,8 days at 50 m. and 37,5 days at 200 m.



Number of days wind speed above 11 m/s [3]

		Open terrain (roughness length 0.05 meter)					Urban area (roughness length 0.5 meter)				
	[1]	10	50	100	150	200	10	50	100	150	200
Forecast calendar	[2]	1.0	3.6	5.0	5.8	6.3	0.0	0.4	1.0	1.5	2.0
Jan	1	1.0	3.6	5.0	5.8	6.3	0.0	0.4	1.0	1.5	2.0
Feb	1	1.0	3.6	5.0	5.8	6.3	0.0	0.4	1.0	1.5	2.0
March	2	2.0	7.3	10.0	11.5	12.7	0.0	0.7	2.0	3.1	3.9
Apr	2	2.0	7.3	10.0	11.5	12.7	0.0	0.7	2.0	3.1	3.9
May	1	1.0	3.6	5.0	5.8	6.3	0.0	0.4	1.0	1.5	2.0
June	1	1.0	3.6	5.0	5.8	6.3	0.0	0.4	1.0	1.5	2.0
Jul/Aug	2	2.0	7.3	10.0	11.5	12.7	0.0	0.7	2.0	3.1	3.9
Sep	2	2.0	7.3	10.0	11.5	12.7	0.0	0.7	2.0	3.1	3.9
Oct	2	2.0	7.3	10.0	11.5	12.7	0.0	0.7	2.0	3.1	3.9
Nov	3	3.0	10.9	15.0	17.3	19.0	0.0	1.1	3.0	4.6	5.9
Dec	2	2.0	7.3	10.0	11.5	12.7	0.0	0.7	2.0	3.1	3.9
	19	19.0	68.9	94.8	109.6	120.2	0.0	6.8	19.0	29.3	37.5

1 = Height [m]
2 = In ratio to 10 meter above open terrain
3 = Of the 191 workable days in the Dutch coastal provinces

Agenda/ 14:10 – 16:00

- Robustness requirements and solutions:
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  - Key-elements
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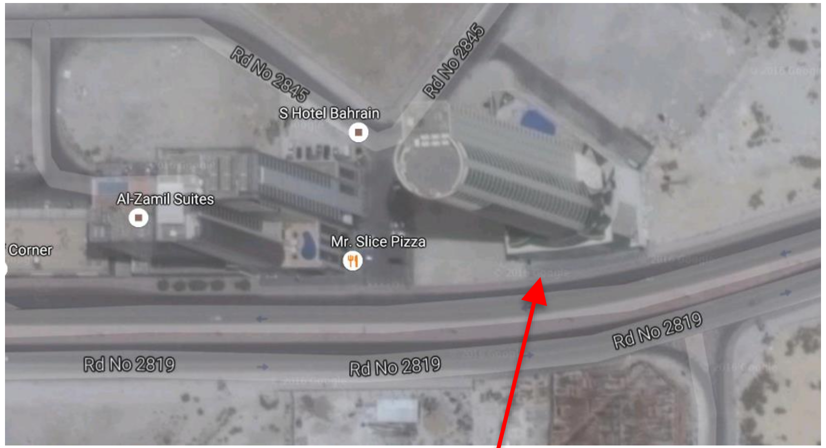
- Case studies of high-rise buildings:
  - Breaker Tower Bahrain (165 m)
  - Zalmhaven Tower Rotterdam (215 m)
- Q&A session

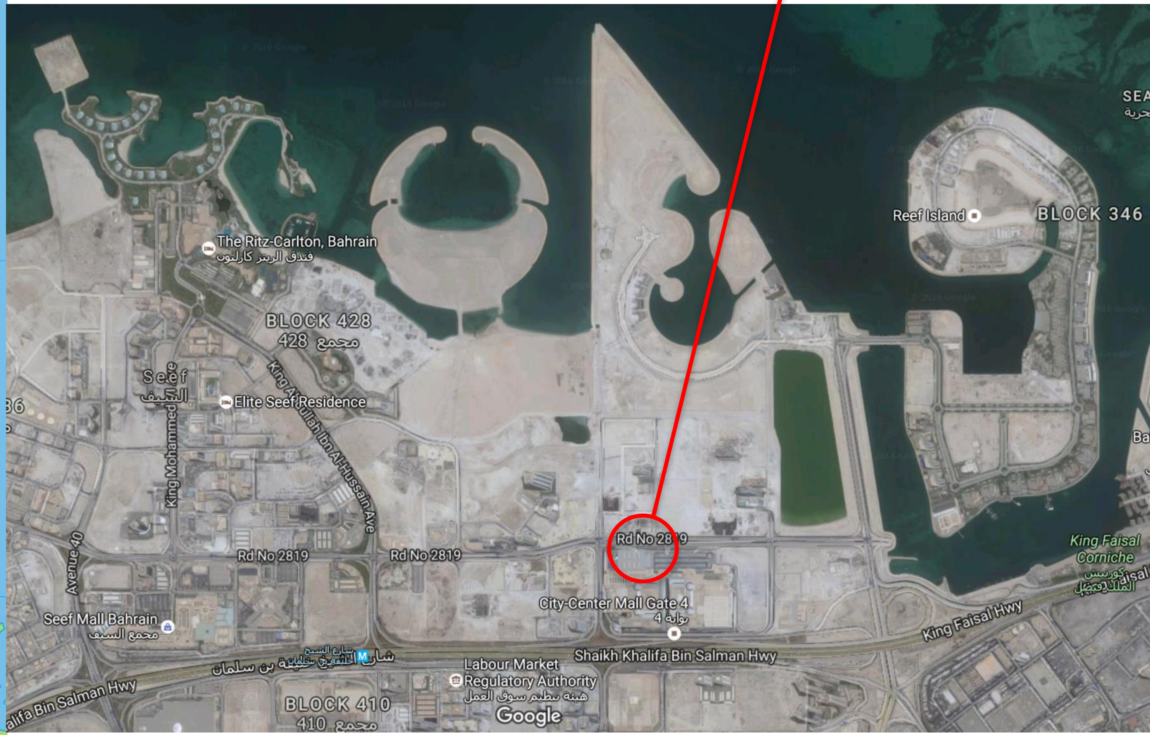
Case study Breaker Tower Bahrain

- 165 meters
- 35 floors
- 29,800 m2 floor surface
- Completely precast concrete
- Completed 2014

Case study Breaker Tower Bahrain

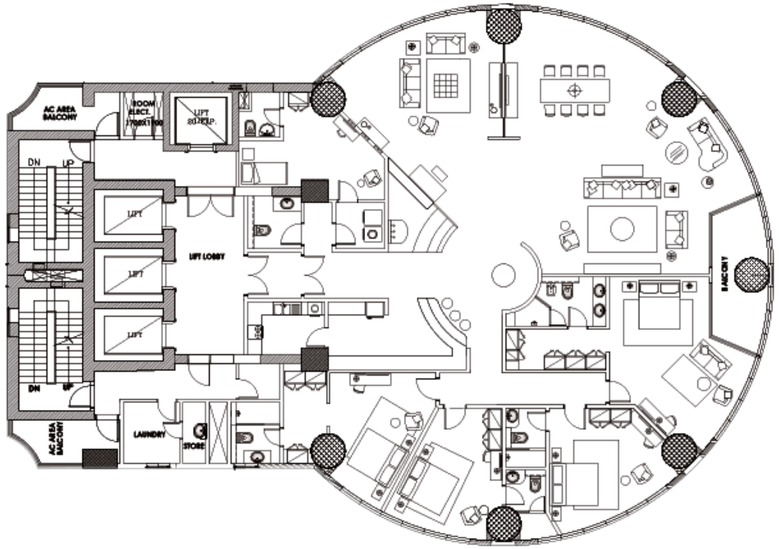
- Seef district – Manama – Kingdom of Bahrain

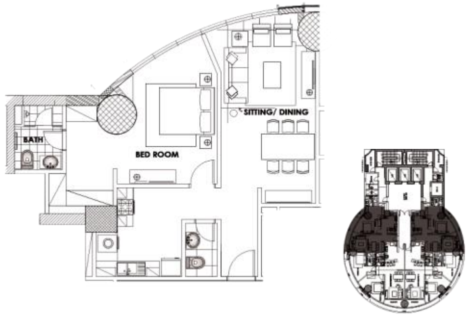


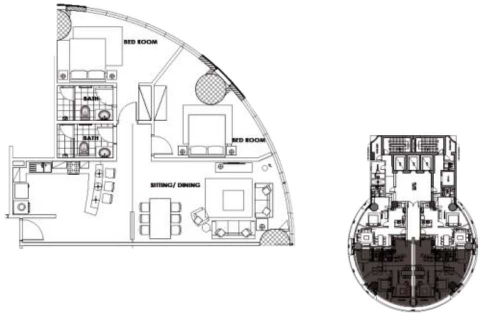


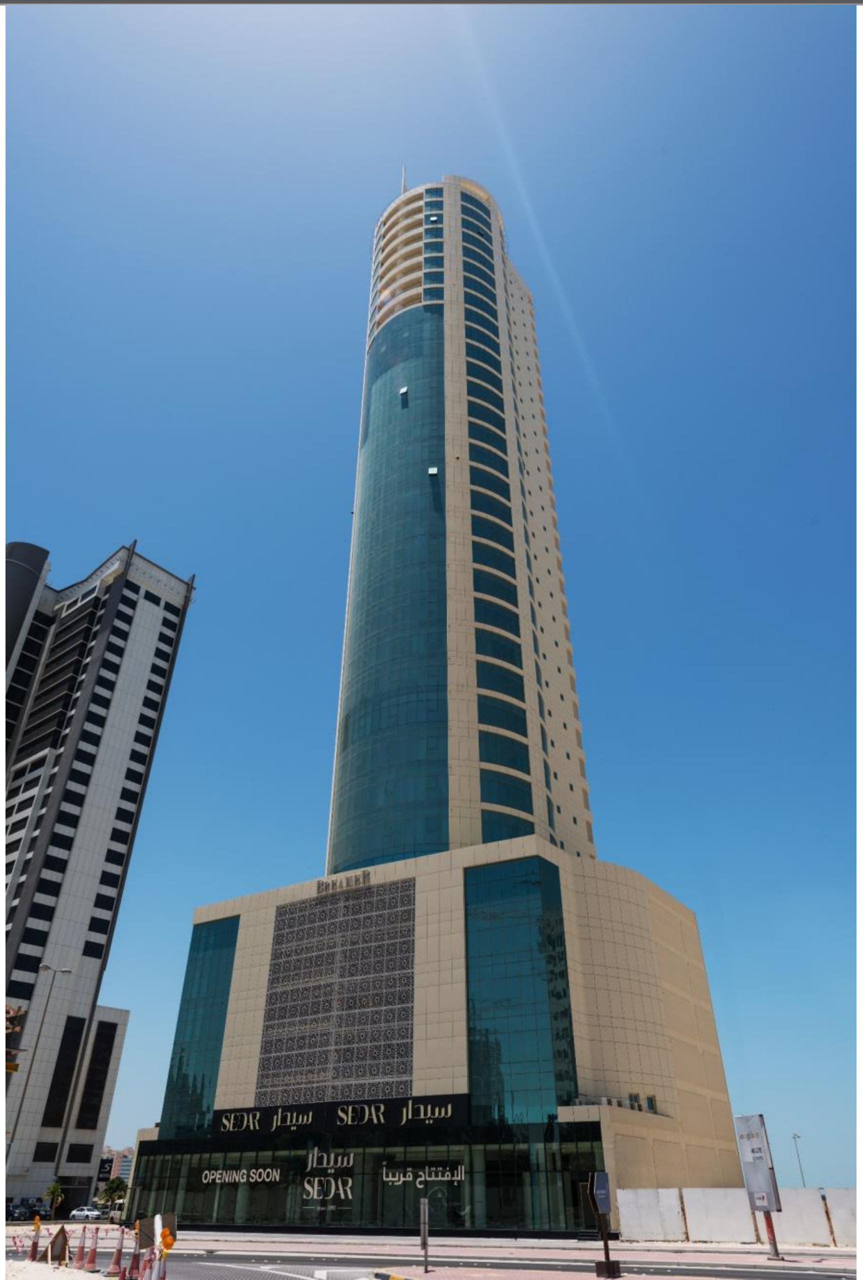
Case study Breaker Tower Bahrain

- 4 storeys with penthouses
- 25 storeys with 4 apartments / floor
- Luxurious style with 4,2 free storey height !
- Showroom at groudfloor







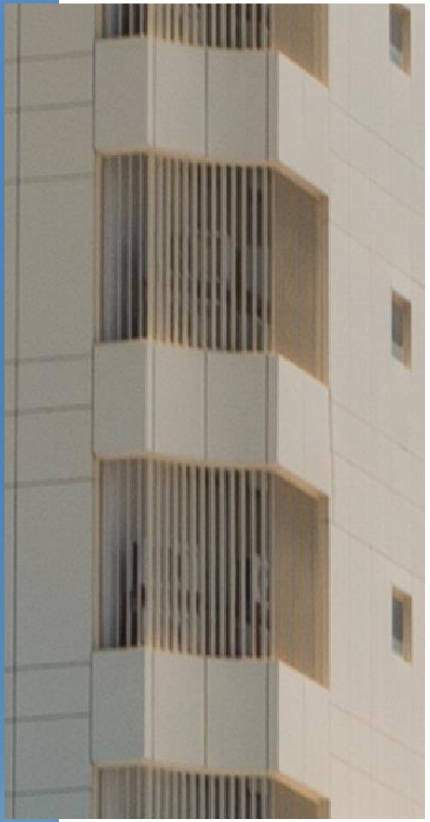


Case study Breaker Tower Bahrain

- 5-storey parking garage for over 200 cars
- Entrance at the back side:

- Climate:

	wo 07 sep	do 08 sep	vr 09 sep	za 10 sep	zo 11 sep	ma 12 sep	di 13 sep	wo 14 sep	do 15 sep	vr 16 sep	za 17 sep	zo 18 sep	ma 19 sep	di 20 sep
Min (°C)	34°	33°	31°	30°	30°	30°	31°	31°	31°	31°	31°	31°	31°	31°
Max (°C)	39°	38°	39°	38°	37°	38°	38°	38°	38°	37°	38°	38°	38°	38°



AC hidden inside the building

Case study Breaker Tower Bahrain

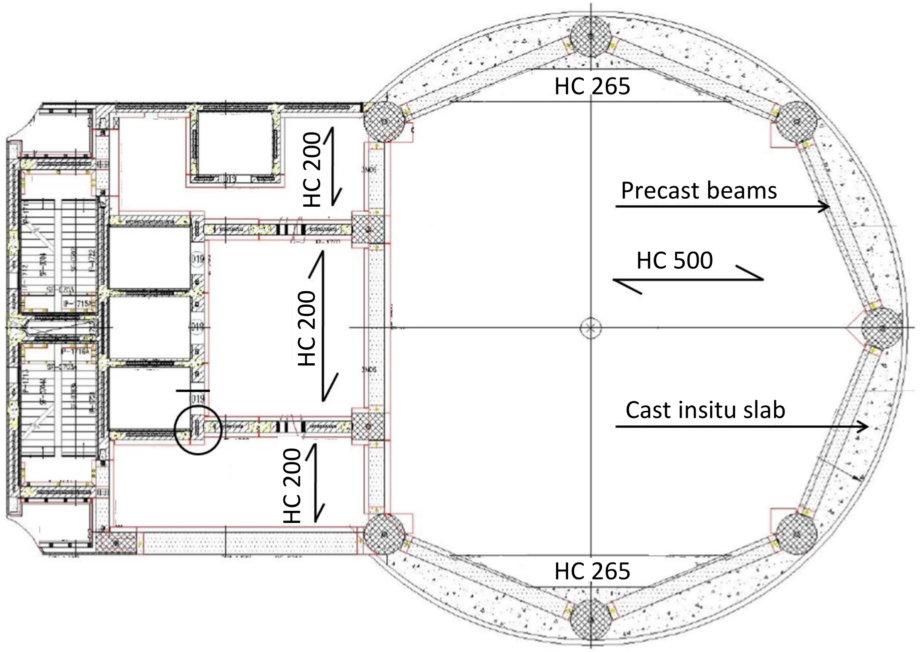
Precast concrete design low-rise:

- Parking garage
- Framed structure with HC slabs

Case study Breaker Tower Bahrain

Precast concrete design high-rise:

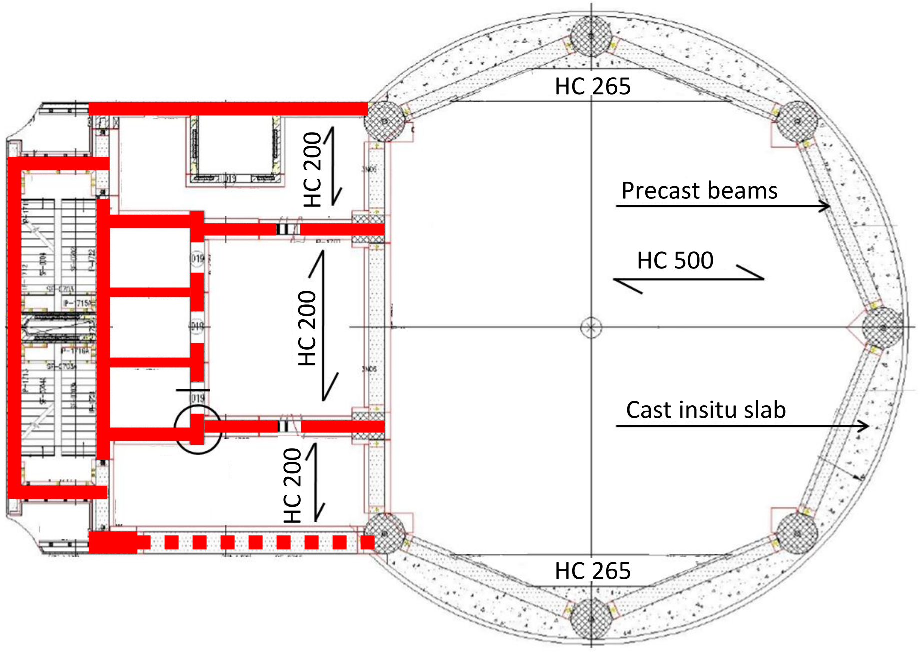
- Floor slab composition
- Beams
- Columns ø 1500 mm, C55/67



Case study Breaker Tower Bahrain

Precast concrete design high-rise:

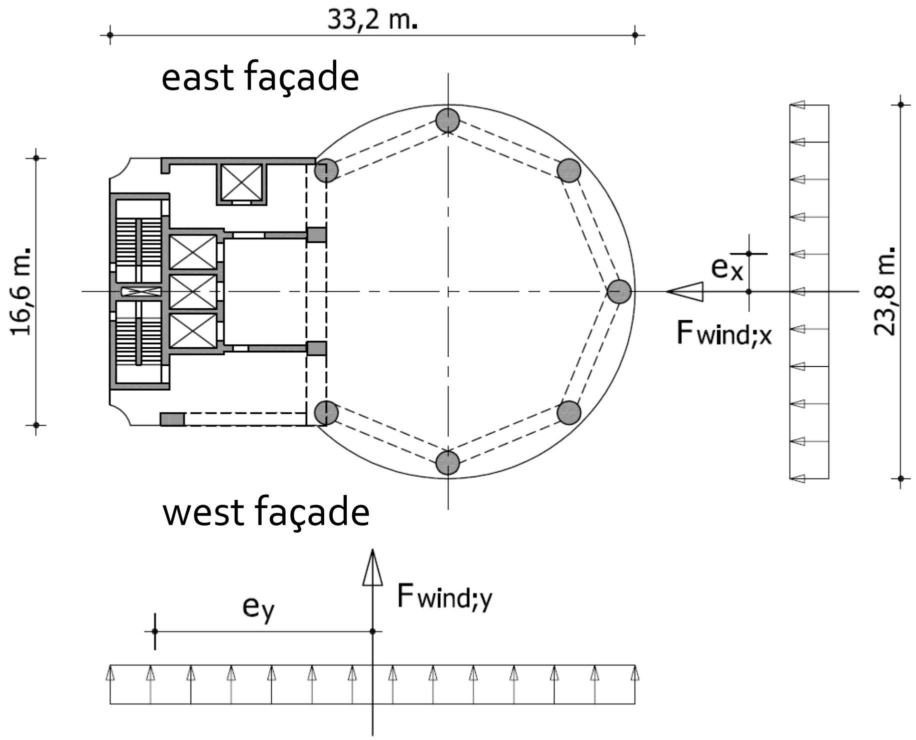
- Cantilever shear wall structures
- Core structures

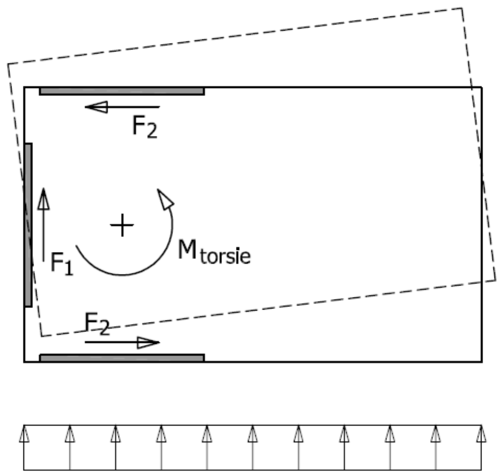


Case study Breaker Tower Bahrain

Structural design high-rise:

- Bending and Torsion caused by excentric lateral loads

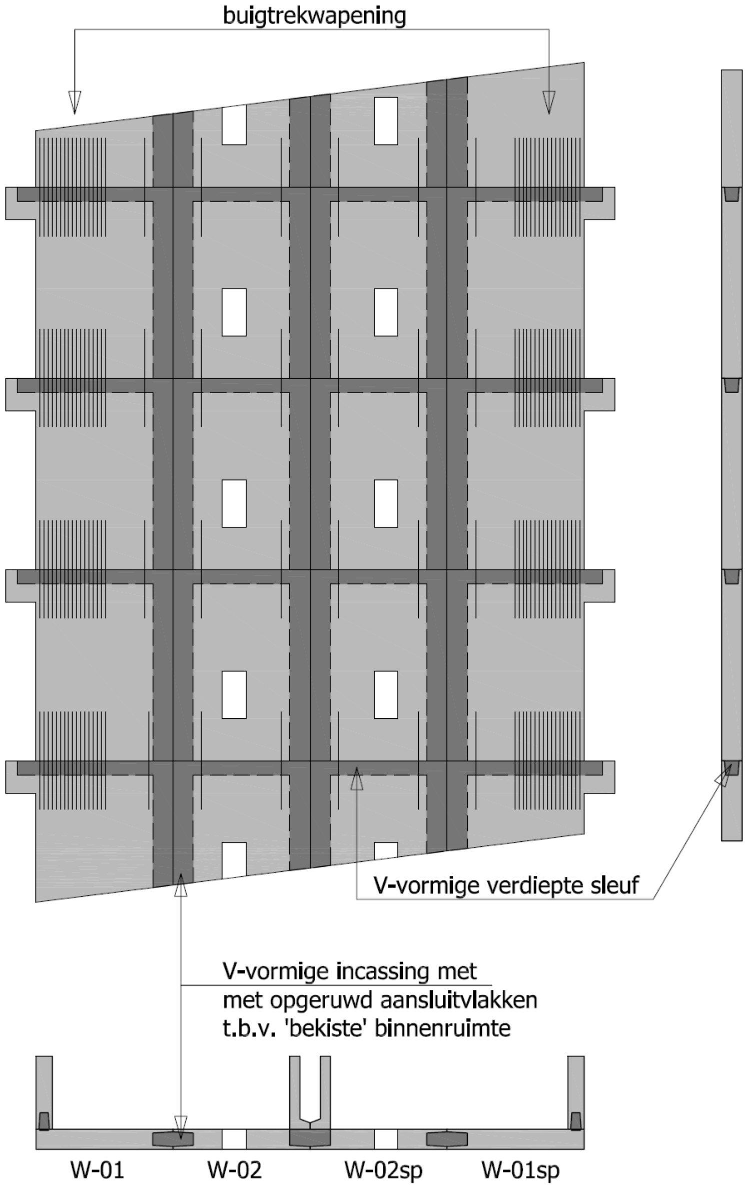




Case study Breaker Tower Bahrain

Structural design high-rise:

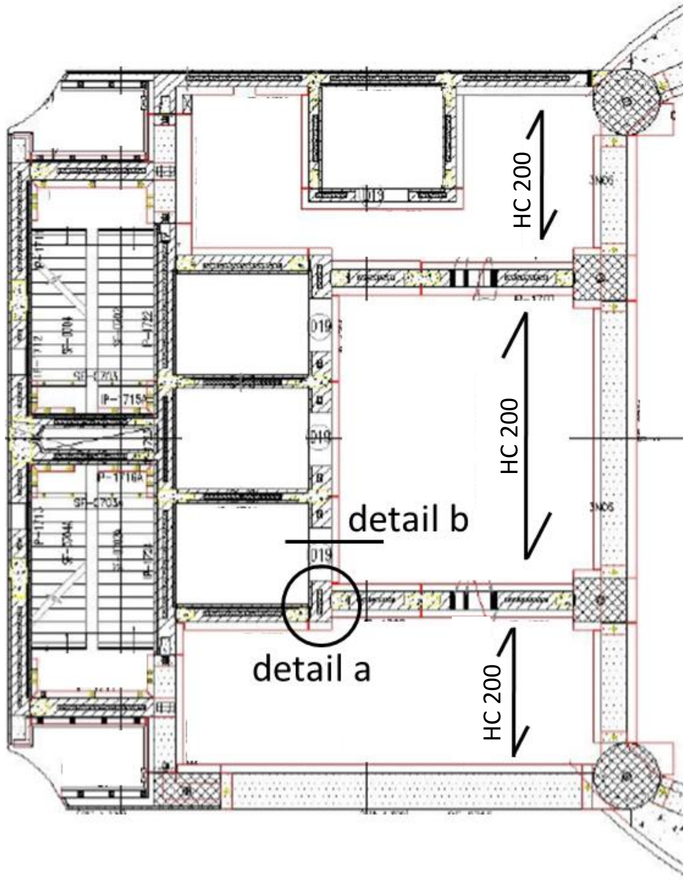
- Precast concrete shear wall connections

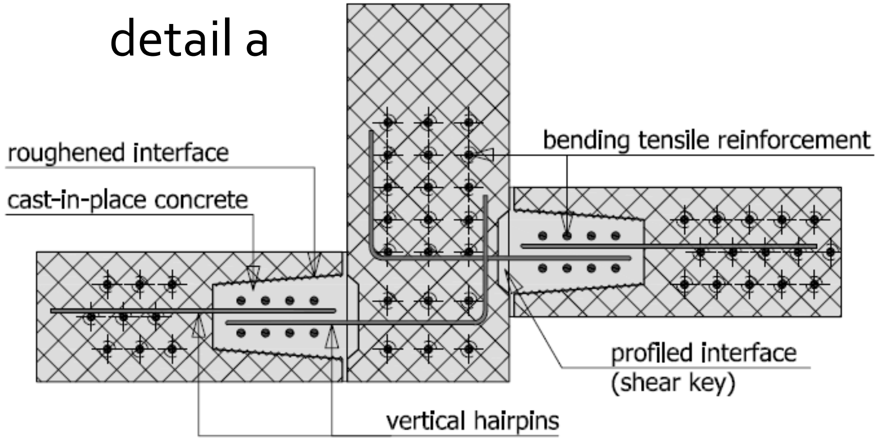


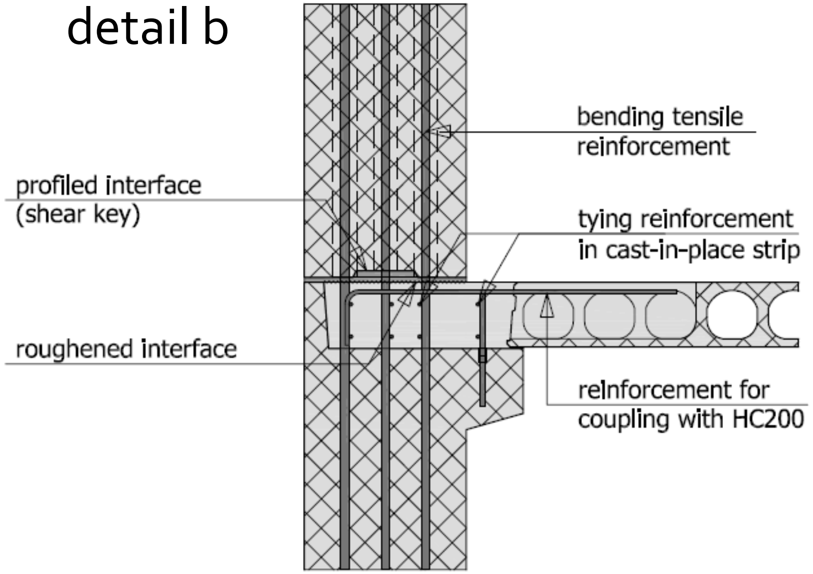
Case study Breaker Tower Bahrain

Structural design high-rise:

- Precast concrete connections



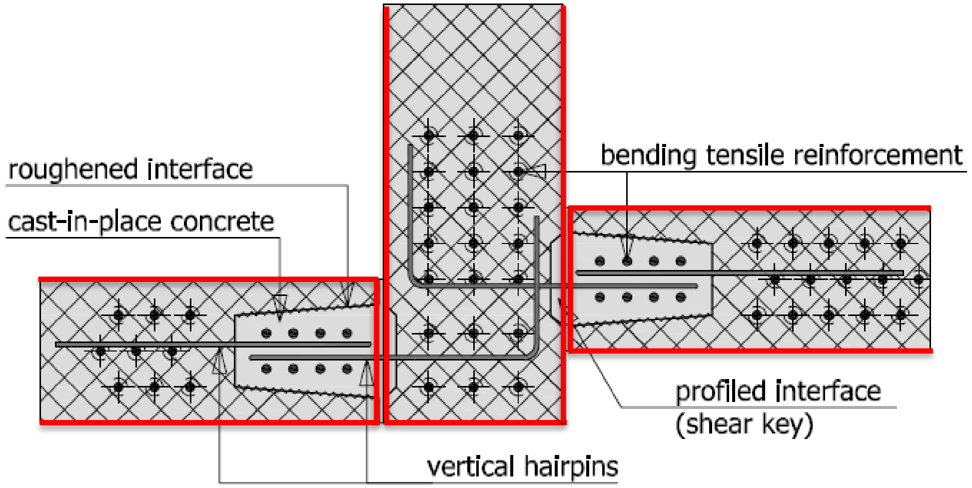


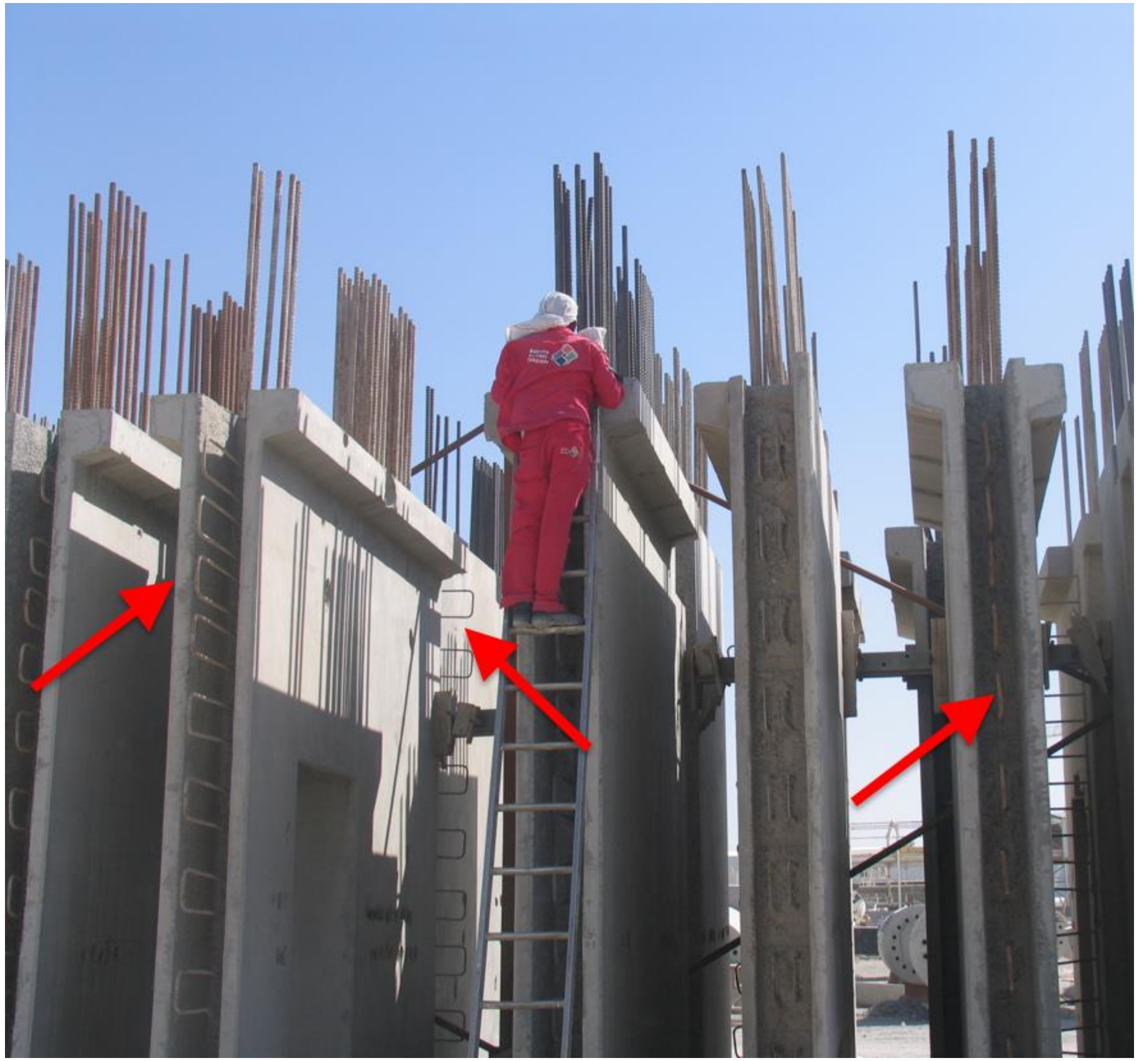


Case study Breaker Tower Bahrain

Structural design high-rise:

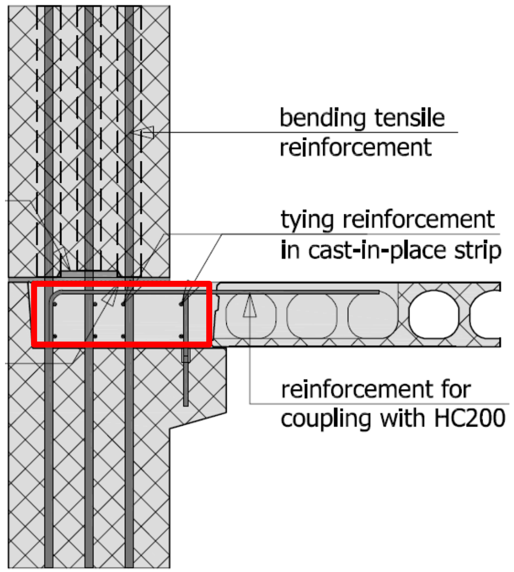
- Connections with vertical hairpins

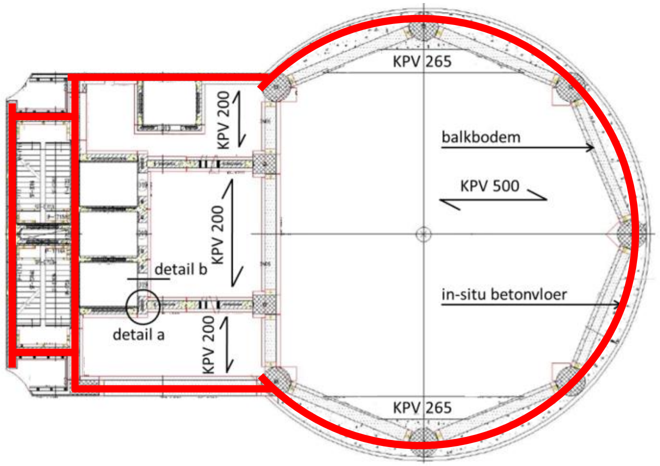


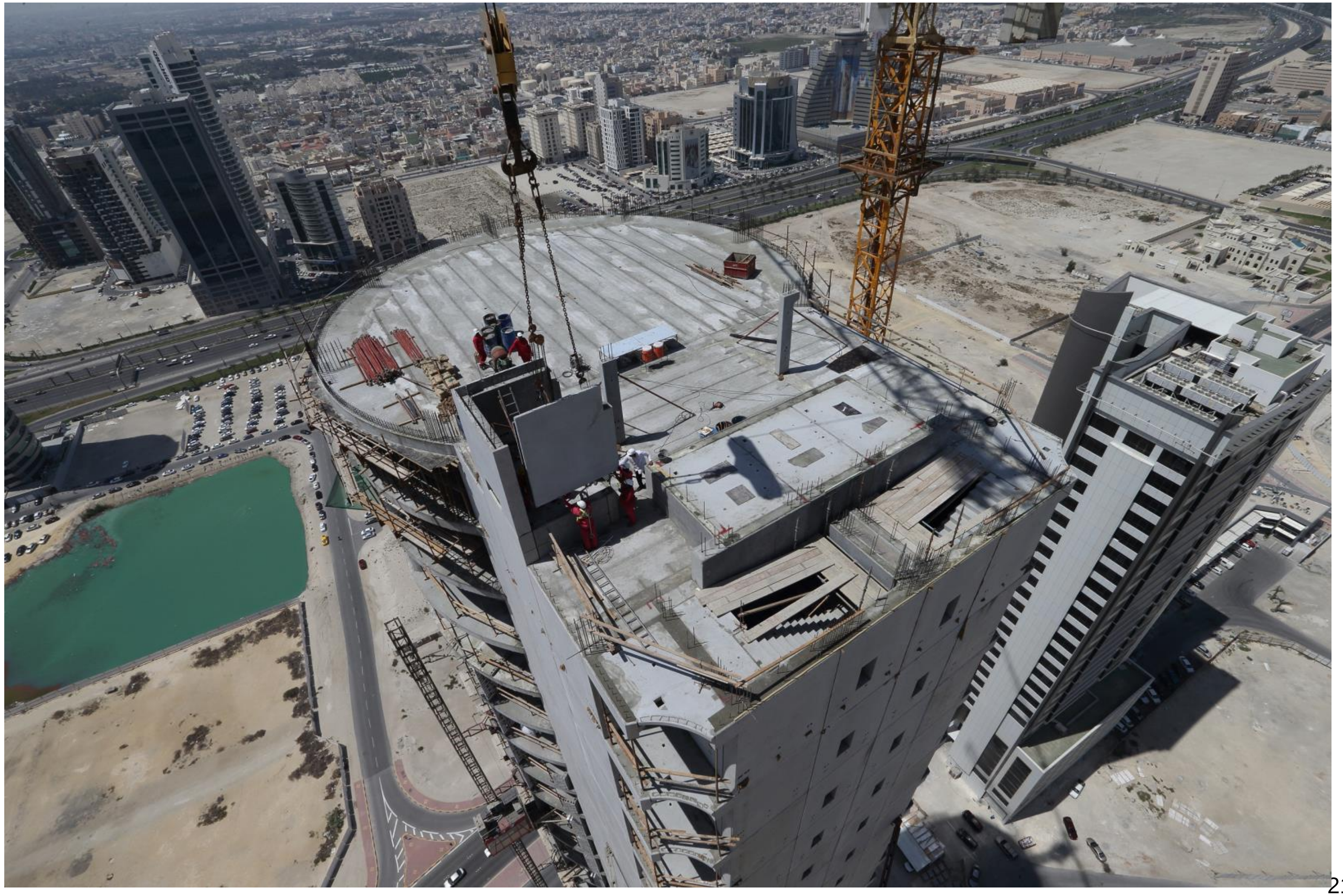


Case study Breaker Tower Bahrain

- Precast concrete diaphragm design with tying reinforcement



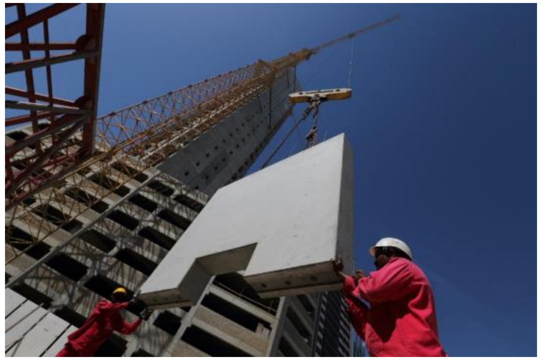




Case study Breaker Tower Bahrain

Precast concrete construction:

- 2 floors ahead produced in factory
- Construction 1 storey / 13 days:
  - 7 days hollow core slabs, beams, stairs
  - 6 days columns and shear wall panels
- Largest tower crane of the region:
  - Capacity 24 ton on 43 meters
  - 100 m free above highest fastening point
  - 168 m high tower crane



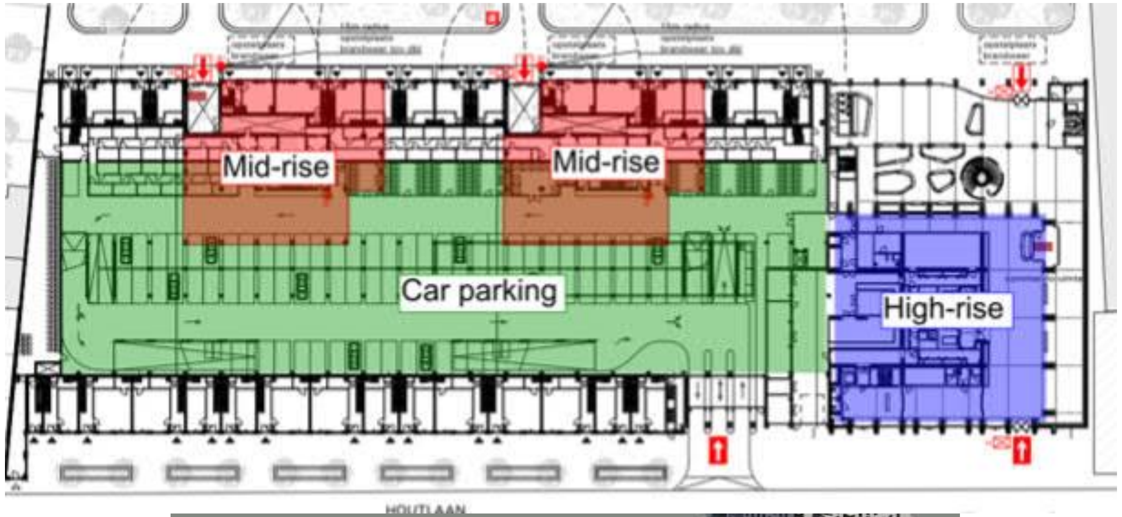
Agenda/ 14:10 – 16:00

- Robustness requirements and solutions:
  - Interpretation of Eurocode provisions
  - Tyings and alternative load paths
  - Key-elements
- Issues related to the design and construction of precast concrete high-rise buildings
- Case studies of high-rise buildings:
  - Breaker Tower Bahrain (165 m)
  - Zalmhaven Tower Rotterdam (215 m)
- Q&A session

Case study Zalmhaven Tower Rotterdam

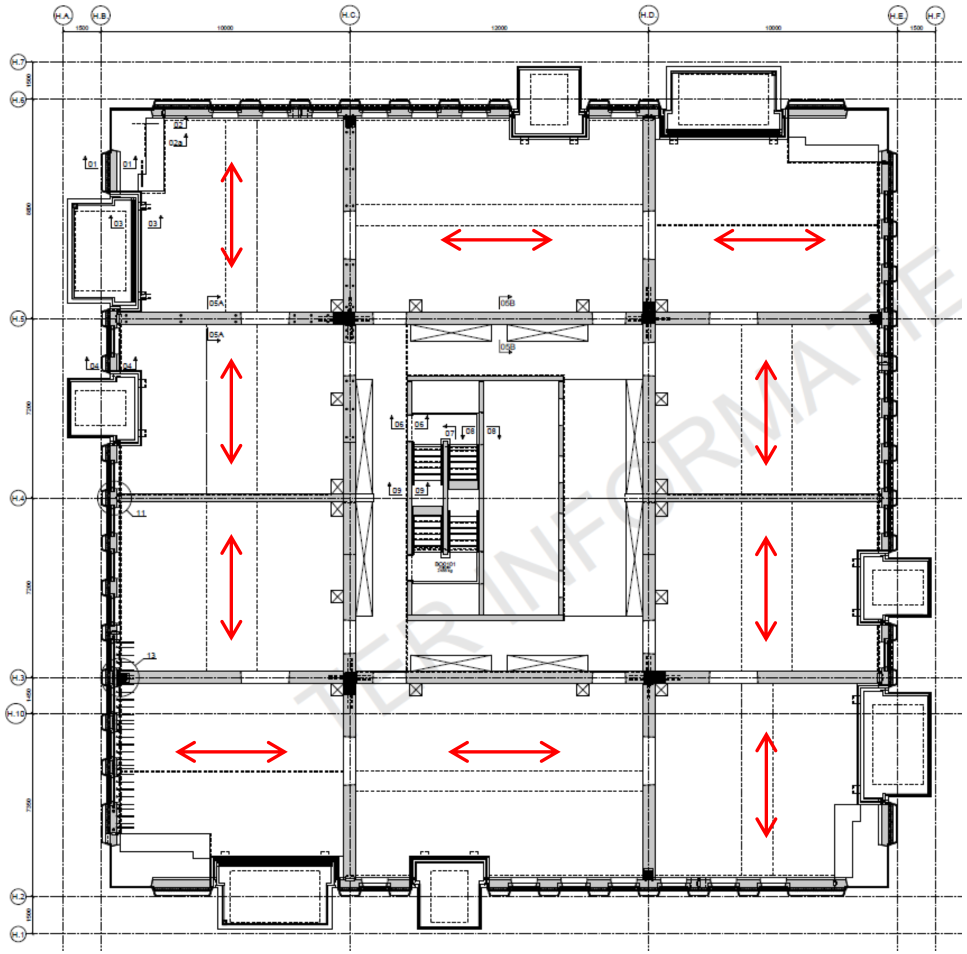
- 215 meter high / 59 storeys
- 242 apartments / 1 restaurant
- Tallest building fully constructed with precast concrete
- Completed in 2022

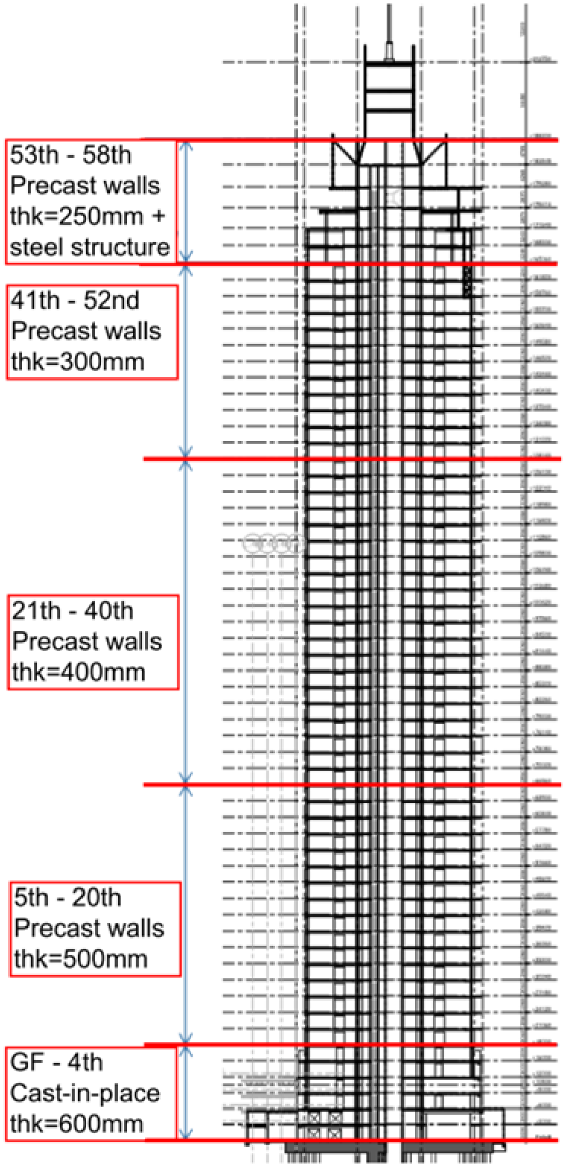
- Overview of the project



Case study Zalmhaven Tower Rotterdam

- Structural plan

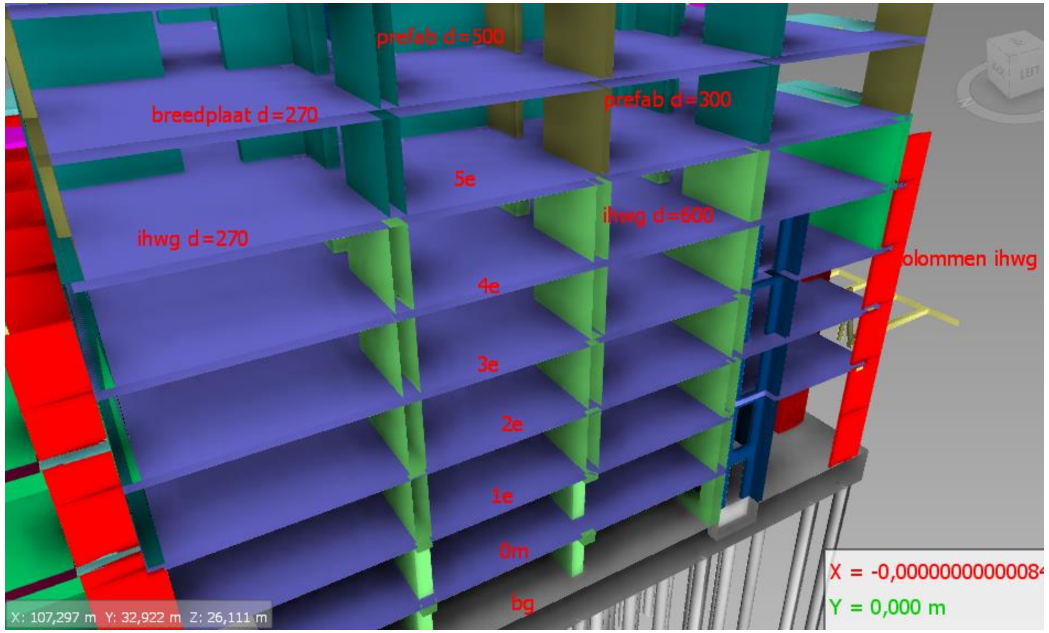


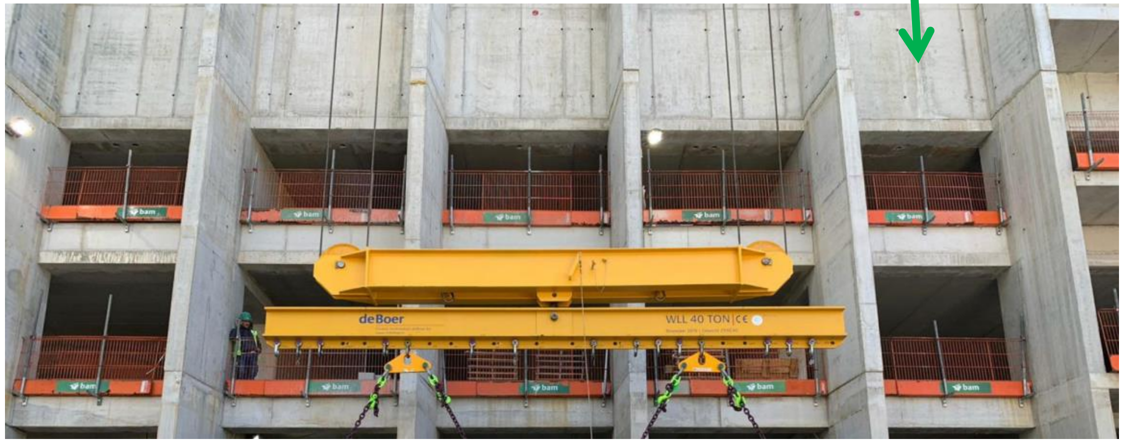


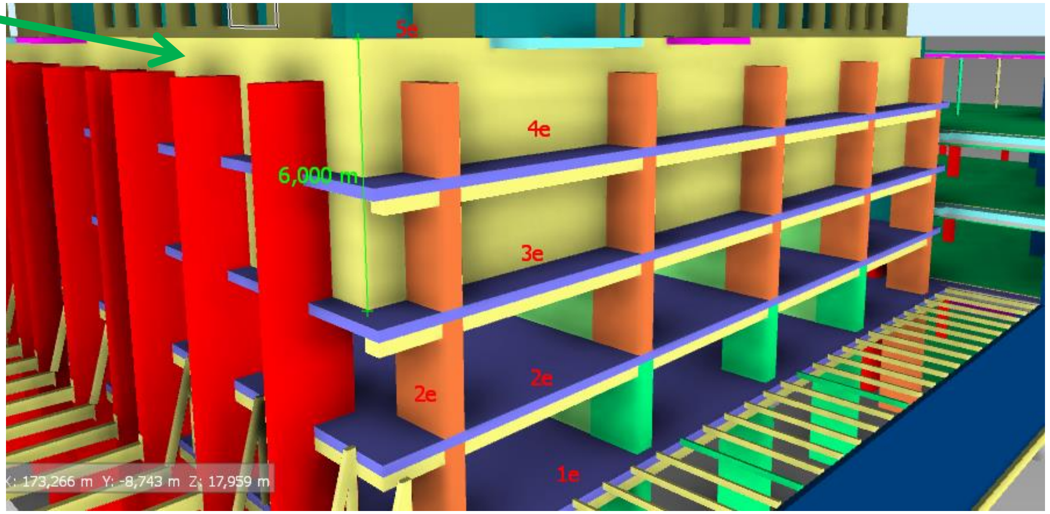
Case study Zalmhaven Tower Rotterdam

Ground – 5th storey

- Cast-in-place concrete
- Climbing formwork
- Walls, thickness 600 mm – C55/67
- Columns C80/95, Slabs – C55/67
- 2-storey concrete transitions walls, thickness 500 mm, C55/67

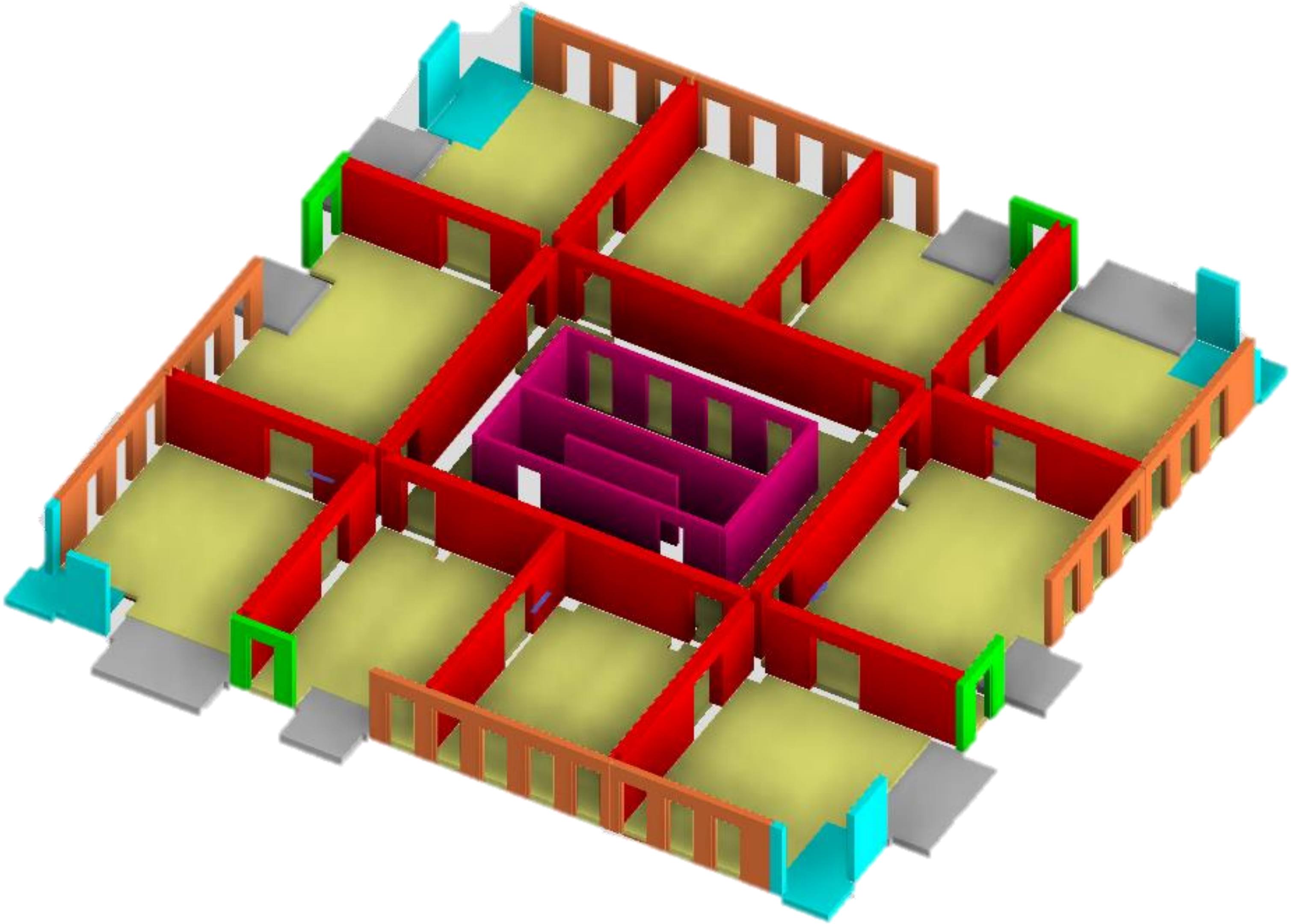






Case study Zalmhaven Tower Rotterdam

- 5th – 54th storey: Precast concrete superstructure



Stability:

Red = Precast concrete inner walls
thickness: 300, 400, 500mm

Orange = precast façade elements
thickness 300 and 400mm

Remaining:

Green = precast façade elements

Purple = lift and staircase walls

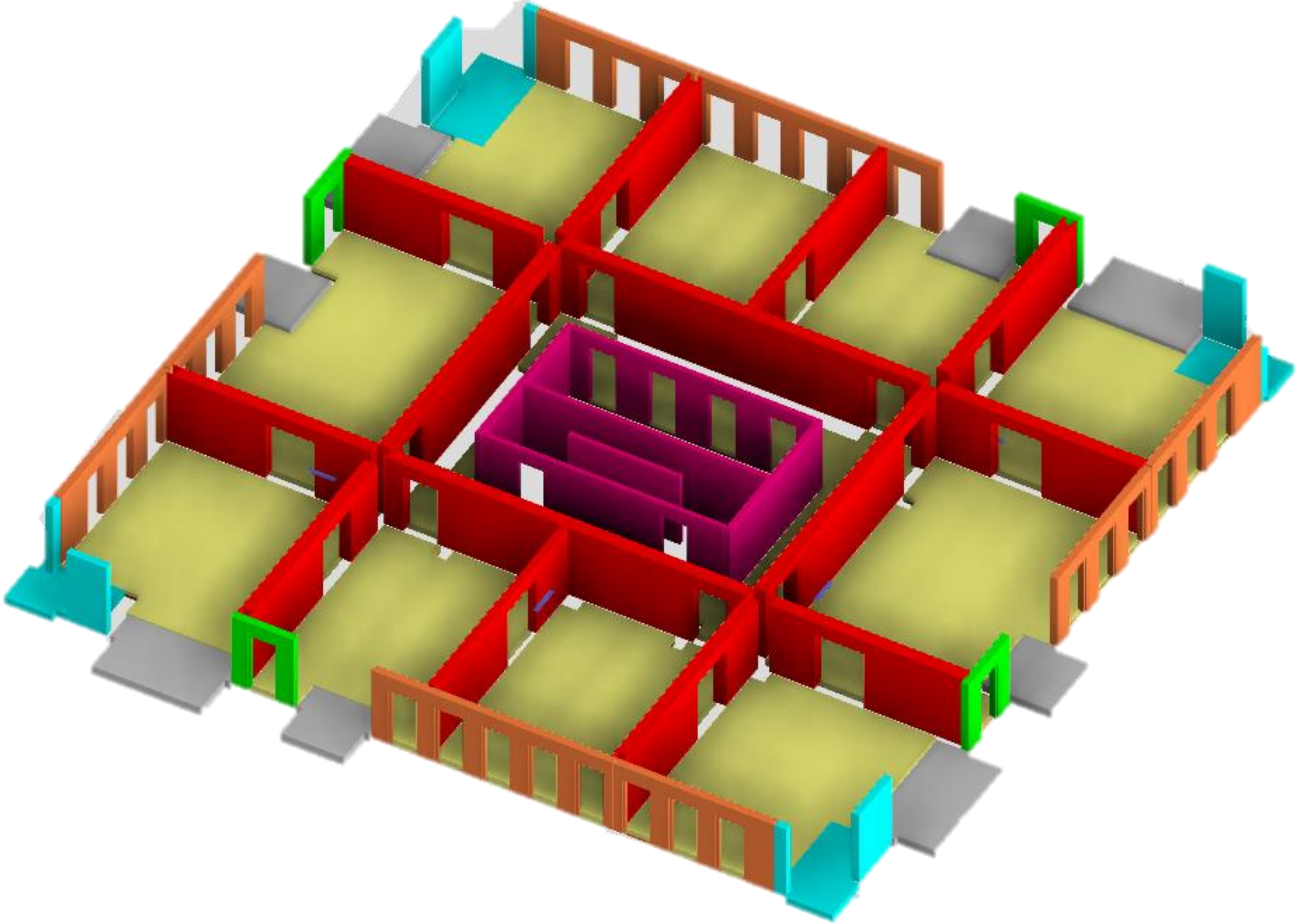
Grey = precast balcony elements

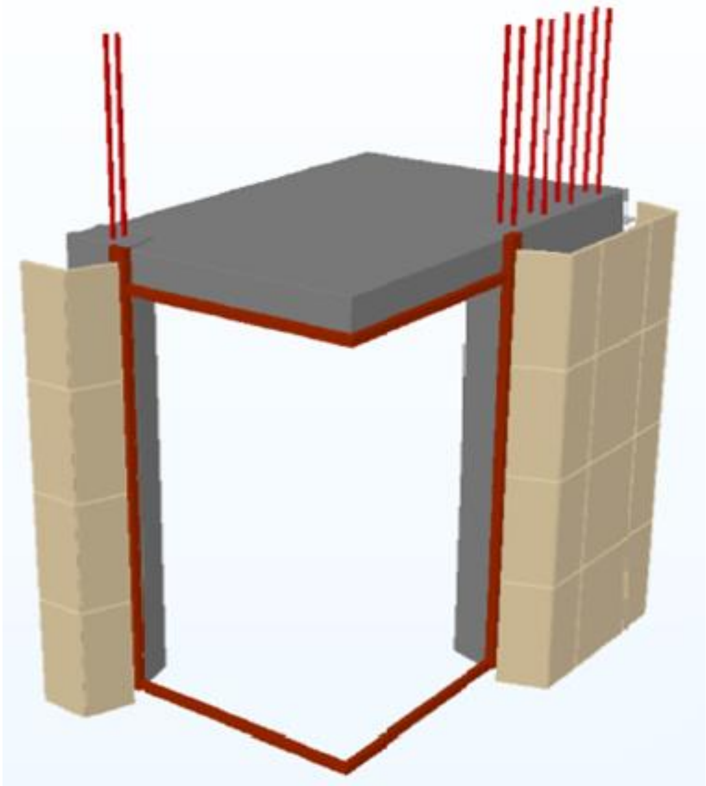
Yellow = floor slabs 270mm
(100 precast +170 in-place)

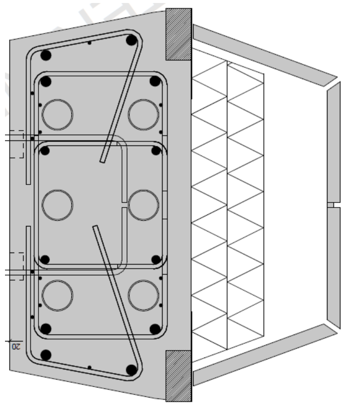
Blue = 3d element (façade + slab)

Case study Zalmhaven Tower Rotterdam

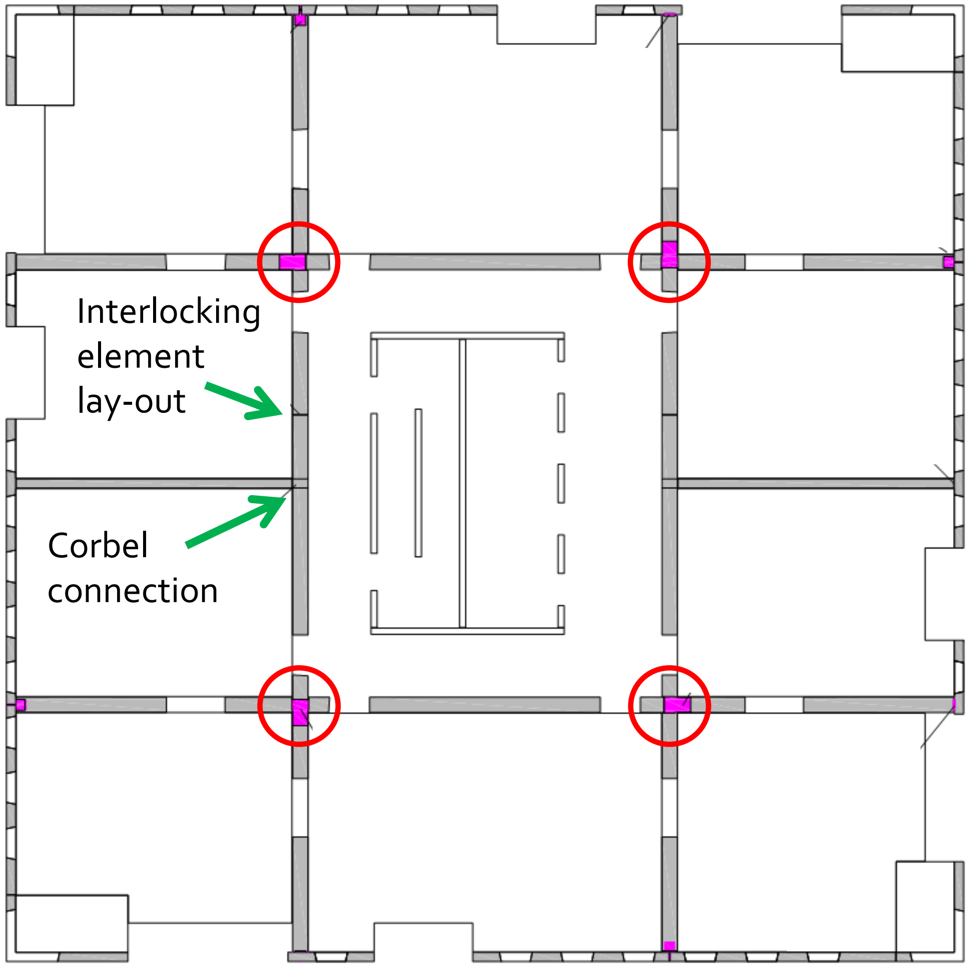
- 5th – 54th storey: Precast concrete superstructure

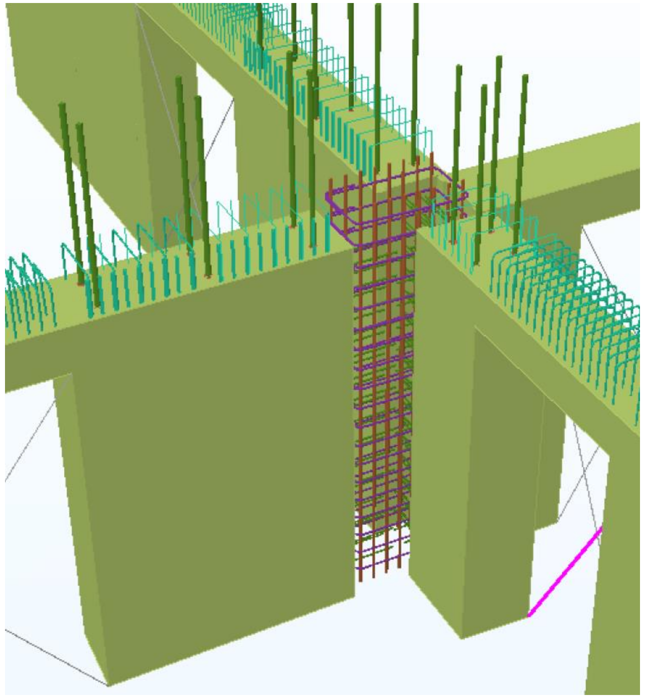


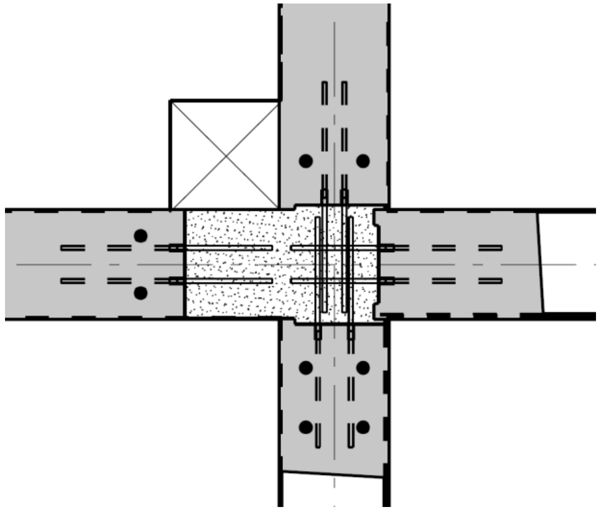




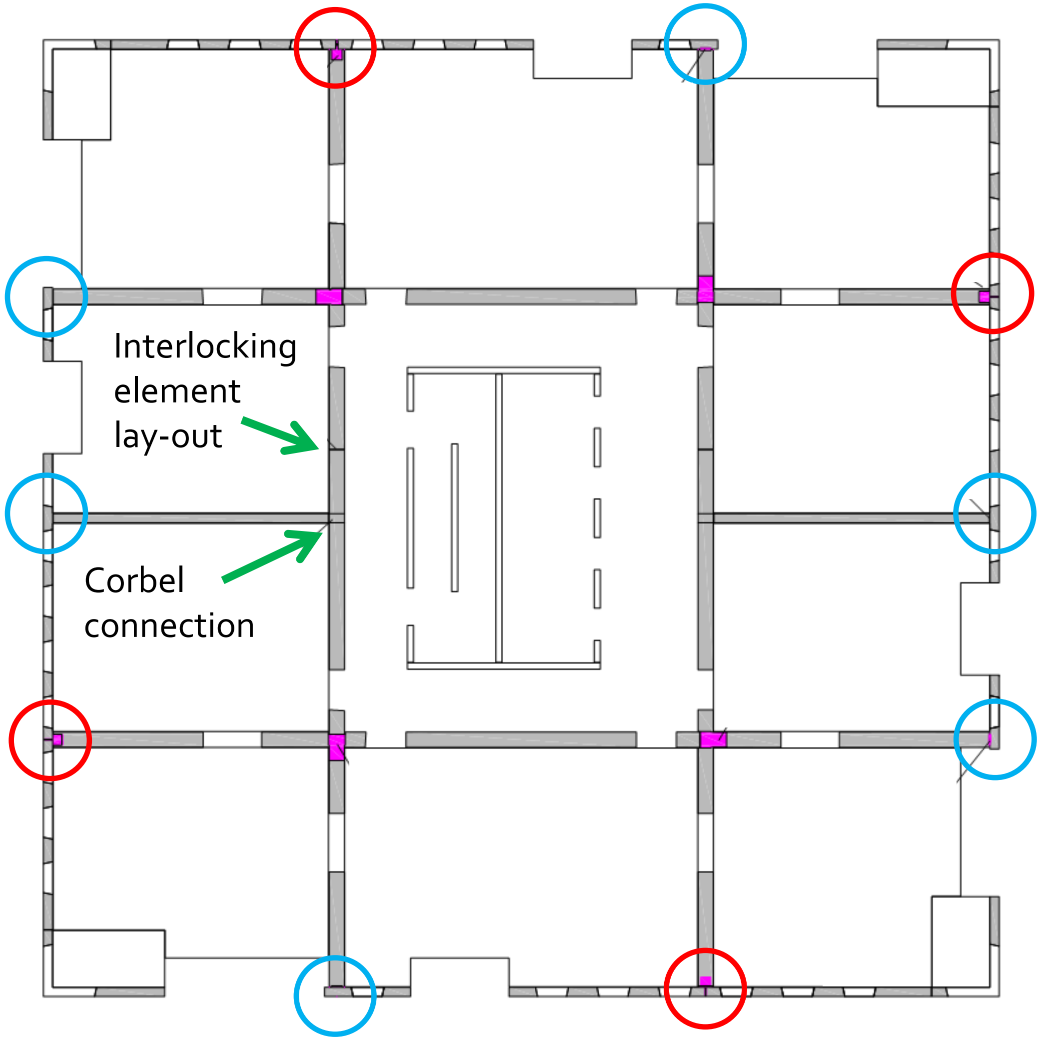
Case study Zalmhaven Tower Rotterdam

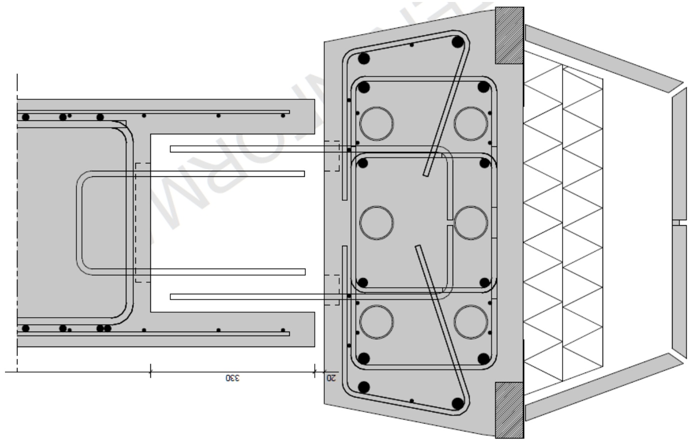




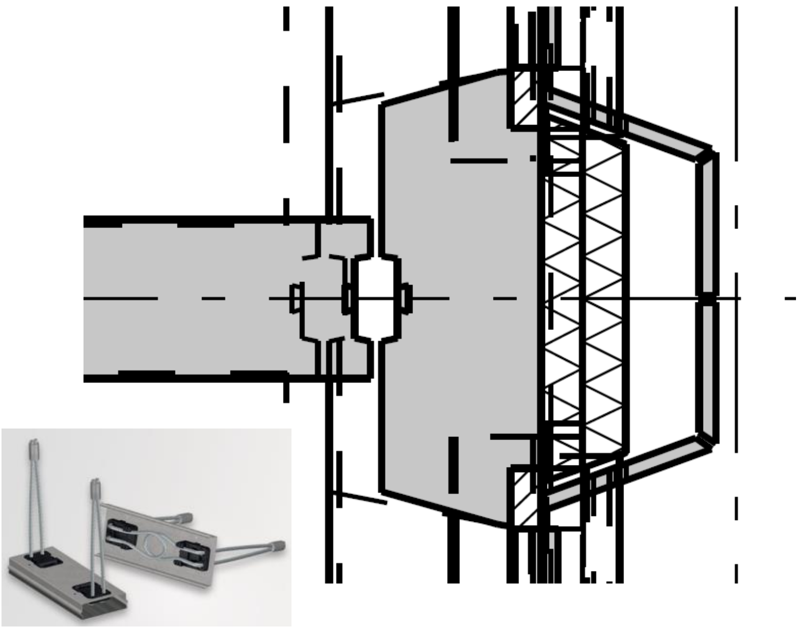


Case study Zalmhaven Tower Rotterdam





Precast connection



Philipp loop connection

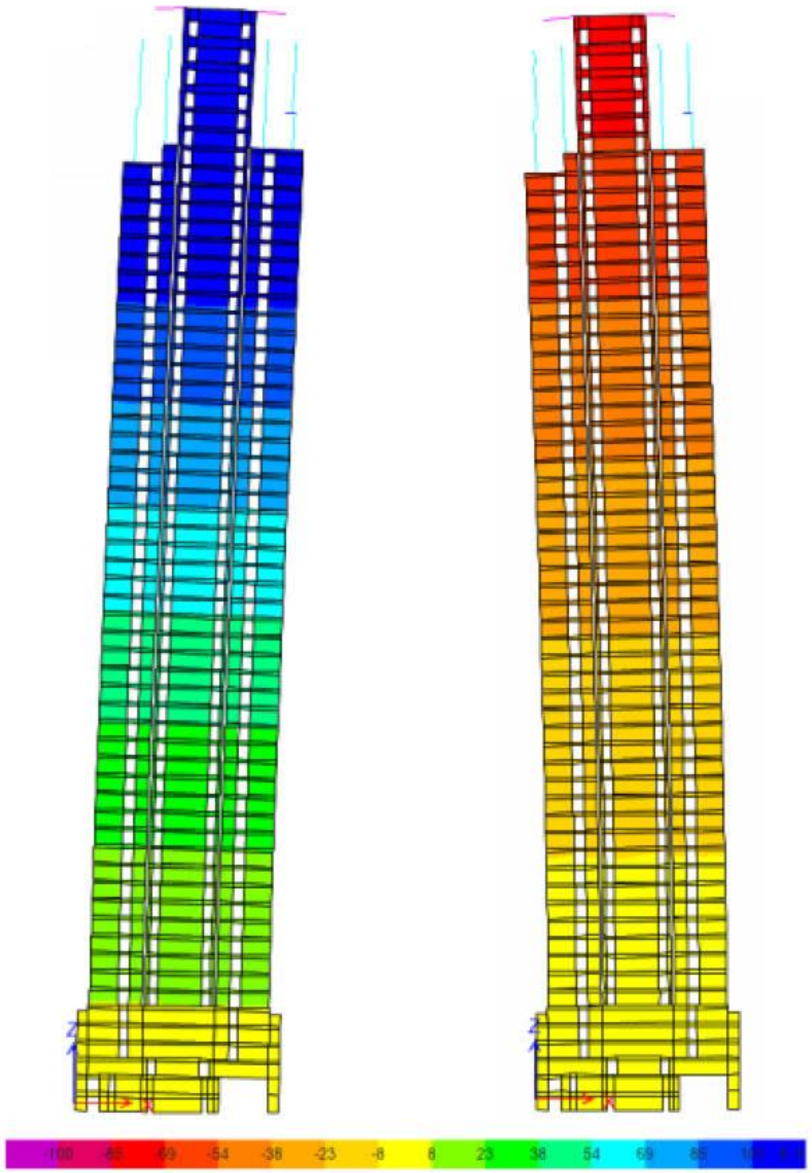
Case study Zalmhaven Tower Rotterdam

Stiffness

- Max permissible lateral deflection = h/500
- Deflection max. 120 mm
- 206/500 = 412 mm

Slenderness:

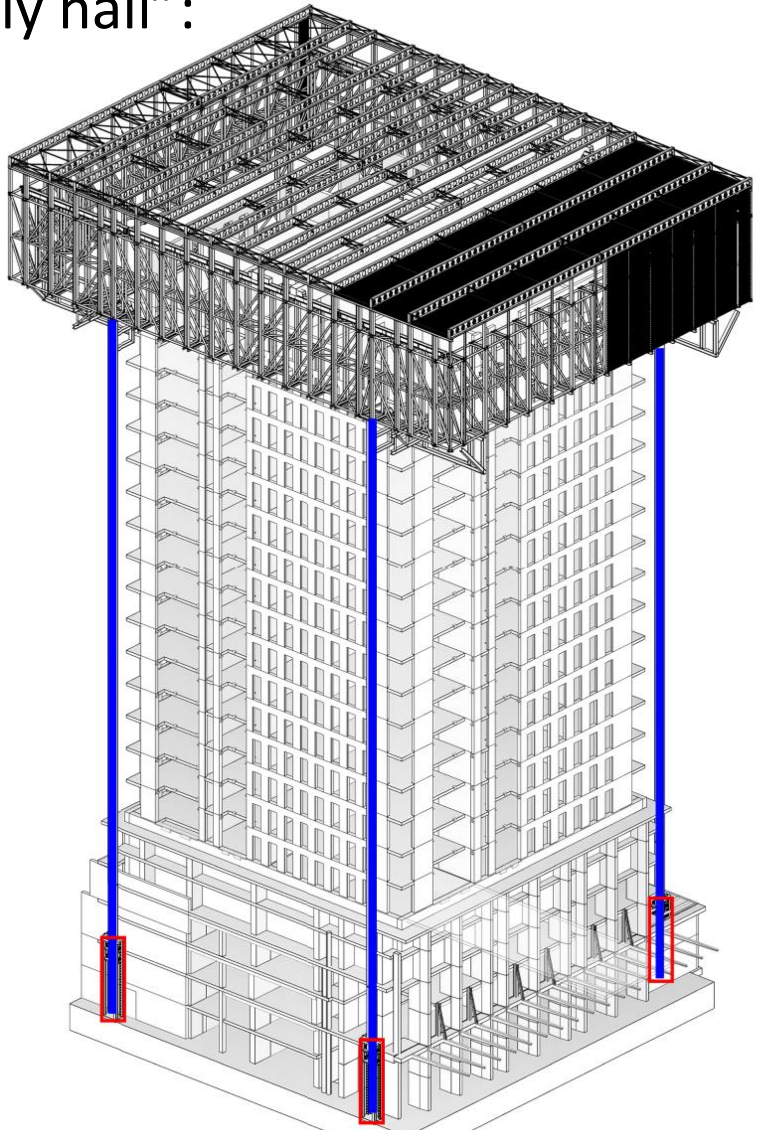
- Height/with ratio
- 206/31,3 = 6,6
- Not really slender



Case study Zalmhaven Tower Rotterdam

Precast concrete construction with self-climbing “Assembly hall”:

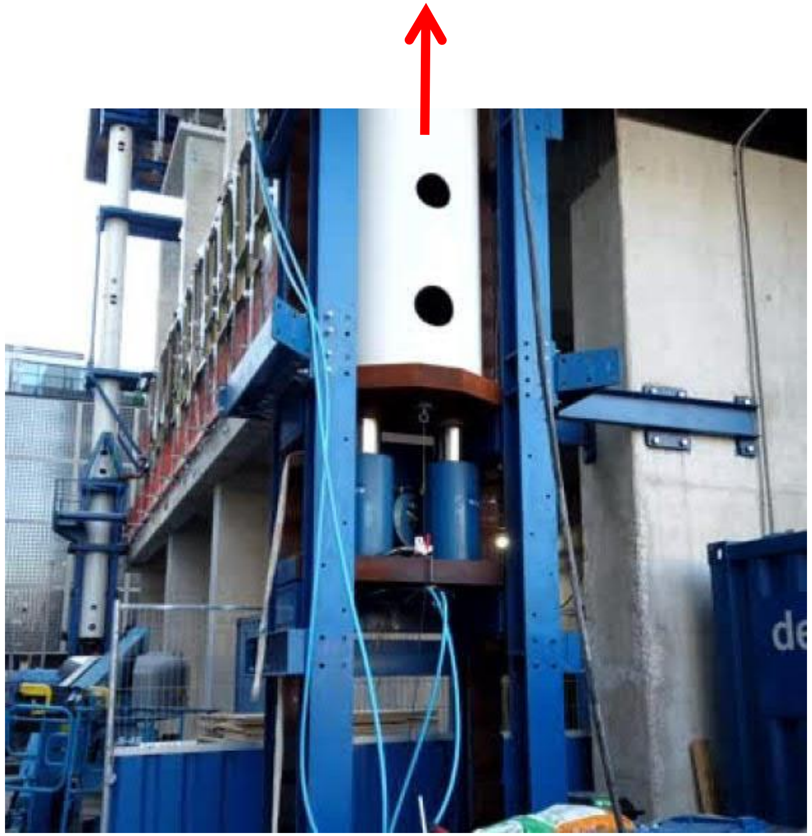
- 4 independent columns (blue)
- Hydraulic jacks at ground floor (red)

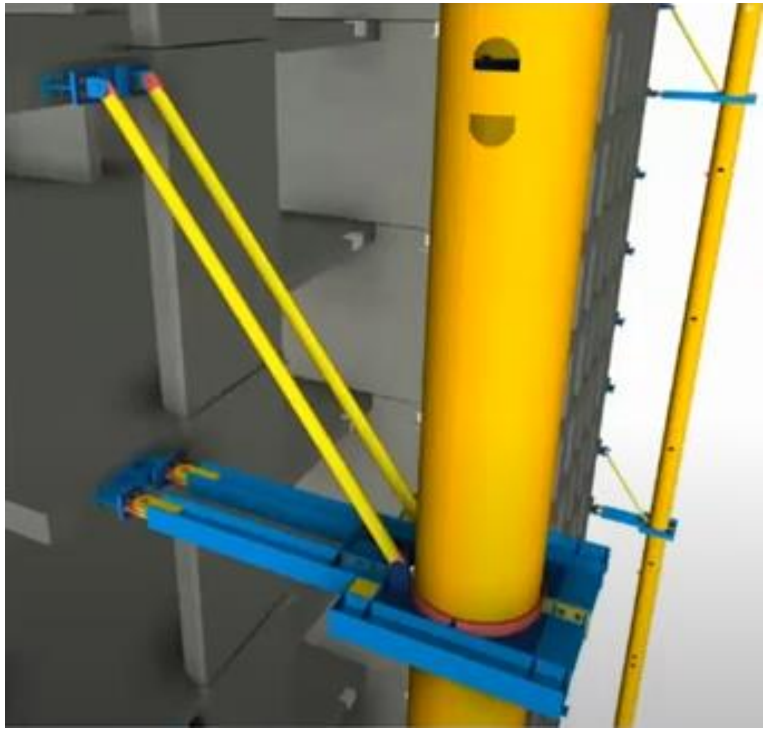


Case study Zalmhaven Tower Rotterdam

Precast concrete construction with “Assembly hall”:

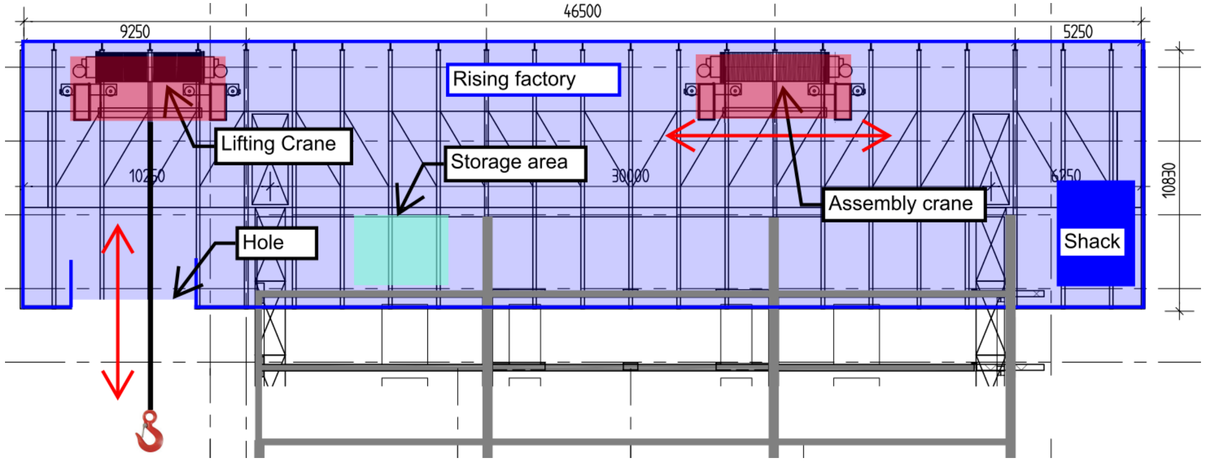
- Hydraulic jacks
- Lateral supports for the “columns”

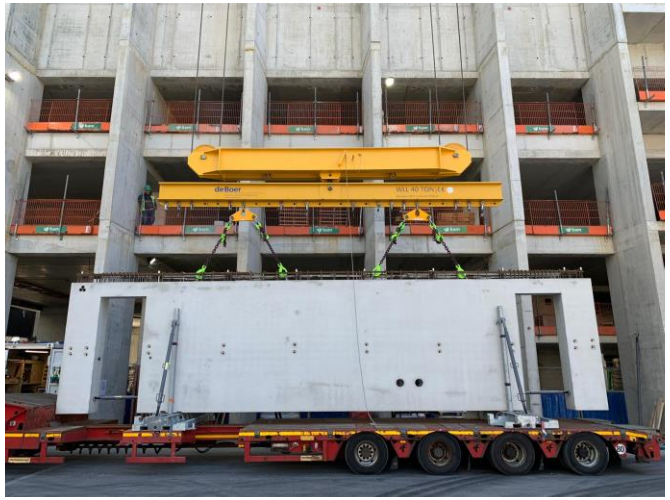




Case study Zalmhaven Tower Rotterdam

Precast concrete construction with “Assembly hall”:



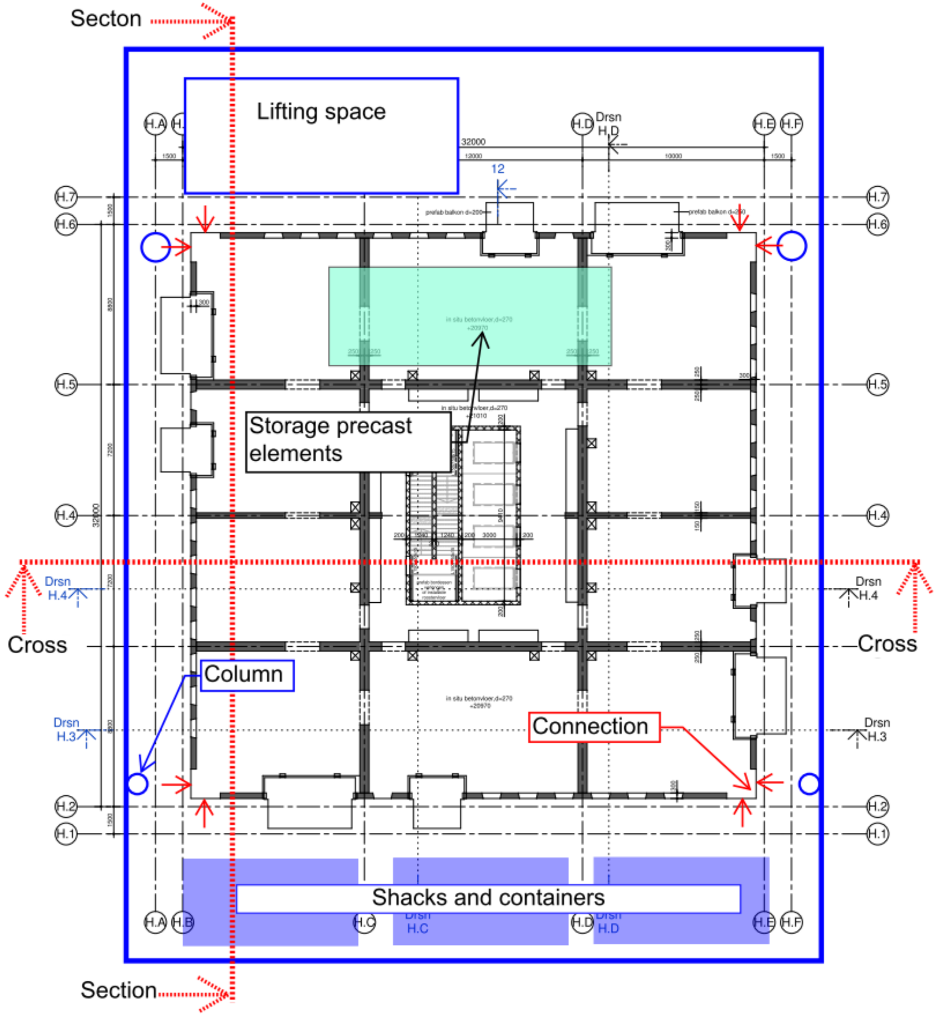


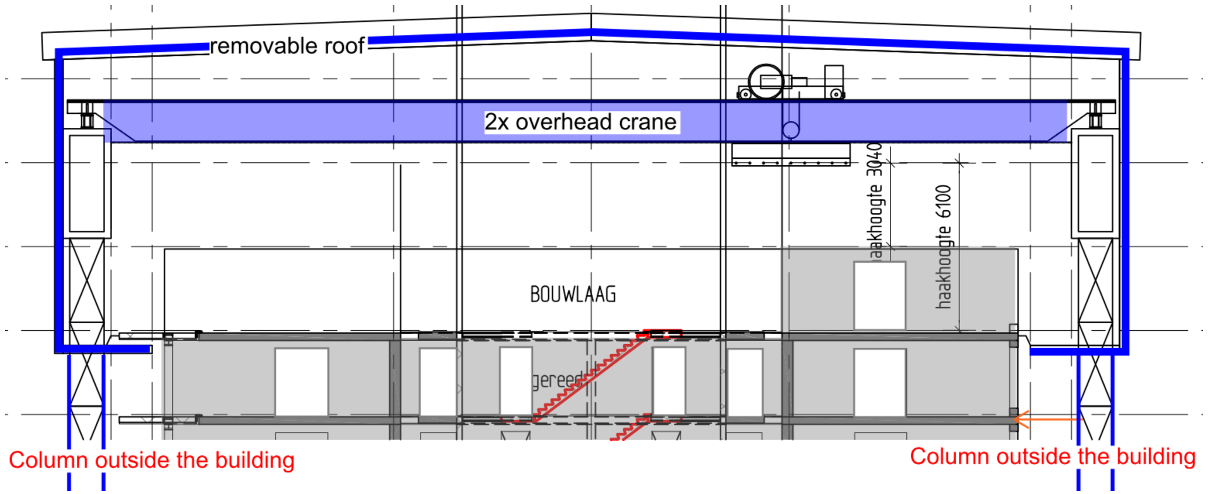
Case study Zalmhaven Tower Rotterdam

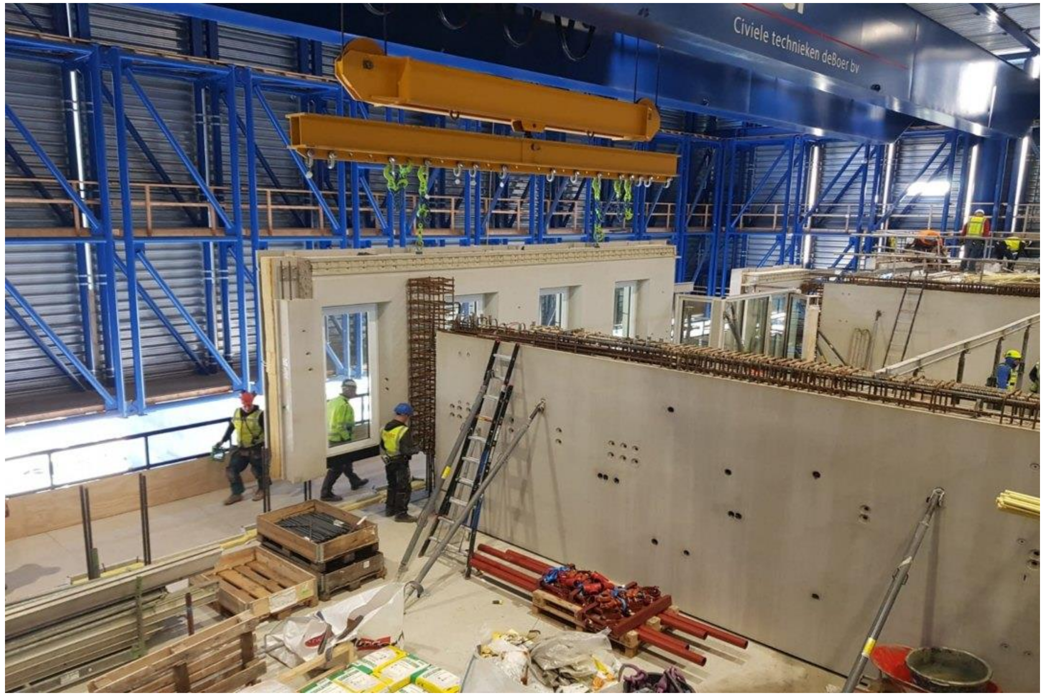
Precast concrete construction inside "Assembly hall":

Case study Zalmhaven Tower Rotterdam

- Inside Assembly hall

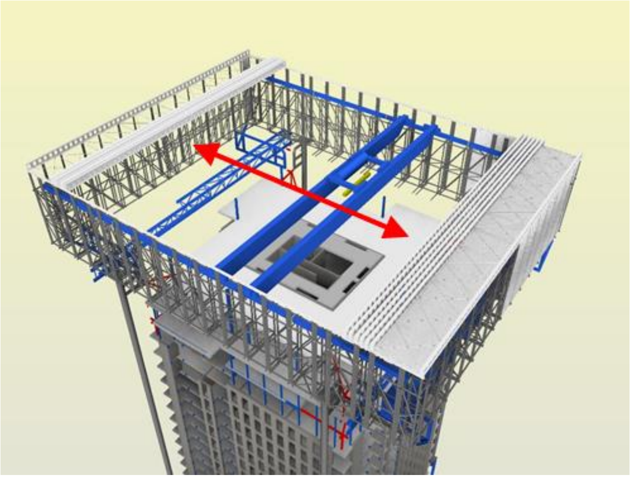


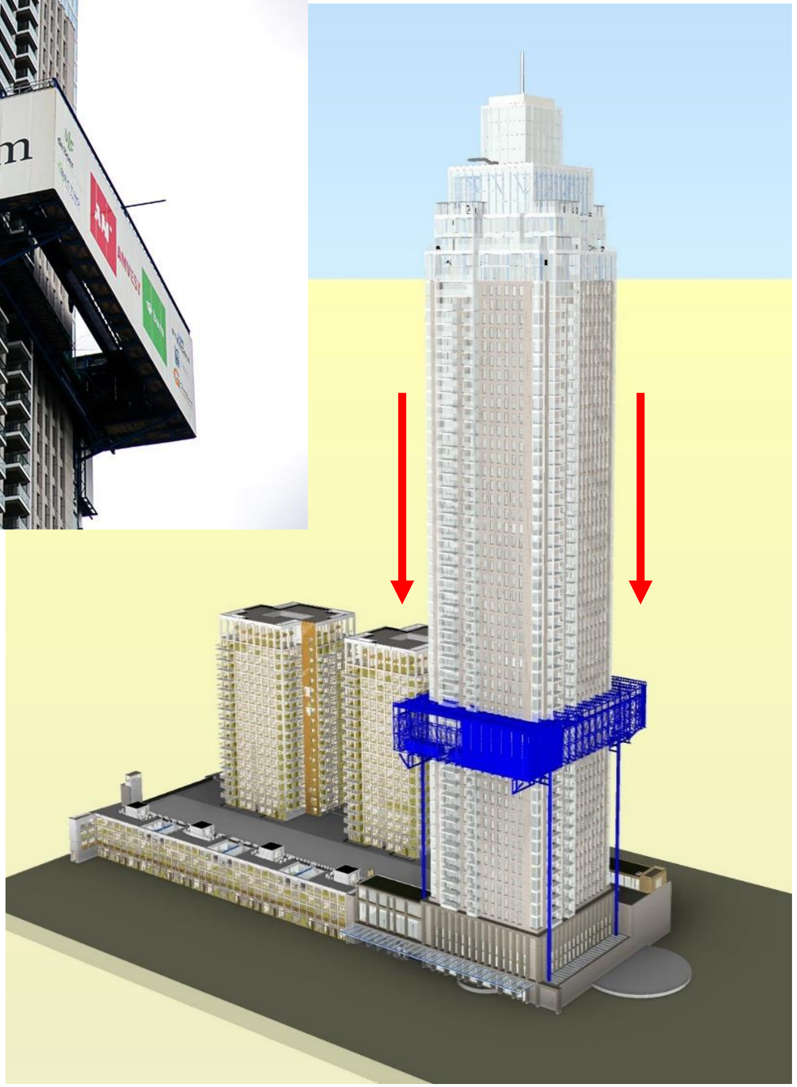


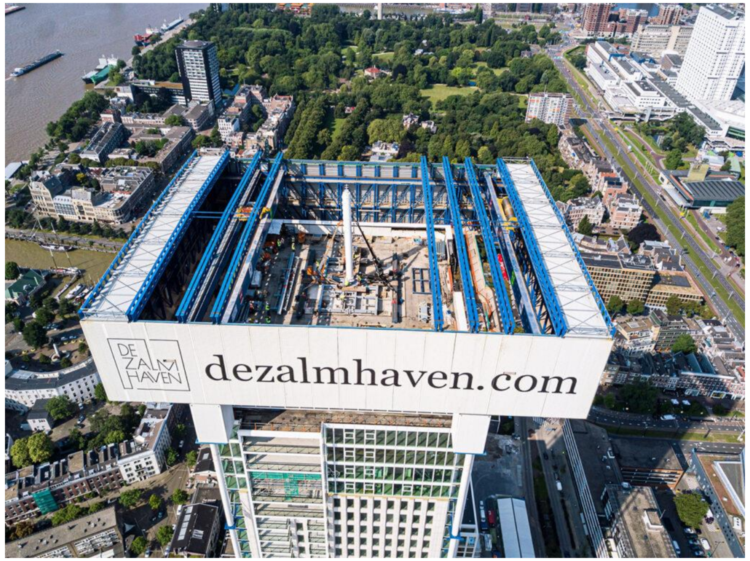


Case study Zalmhaven Tower Rotterdam

- Dismantling Assembly hall







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Ekonomikas ministrija

INŽENIERU KOMPETENCES CENTRS

Thank you for your attention!

Dick van Keulen un Gabriel Tarta

ID Nr. EM 2023/28
Riga, 2023