Multi storey buildings

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European Erasmus Mundus Master Course

Sustainable Constructions
under Natural Hazards and Catastrophic Events

520121-1-2011-1-CZ-ERA MUNDUS-EMMC

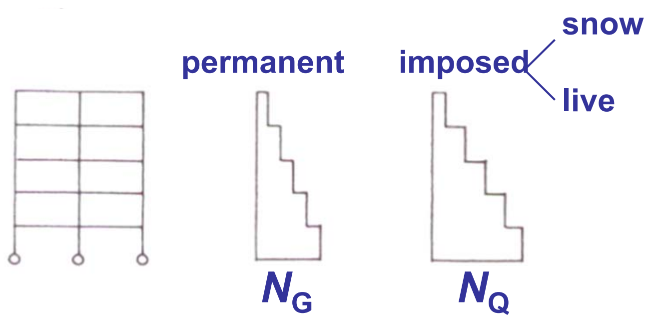
Universitatea Politehnica Timişoara

Part II – Multistorey buildings

- Columns: columns, site and shop splices, column bases, anchoring types.
- Bracings: classification (first and second order) frames, distribution of forces into vertical bracings, detailing of diagonal and frame bracings.

Columns

a) Columns which are not part of the bracing system



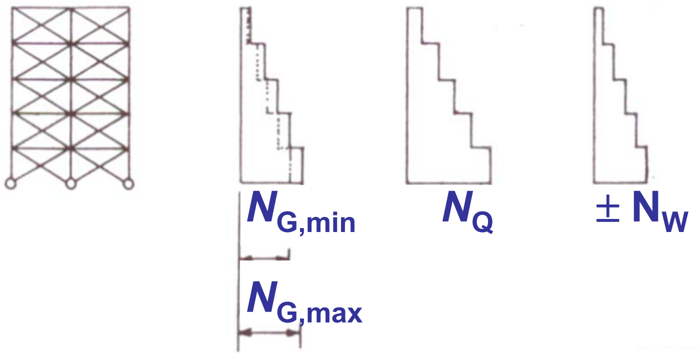
EN 1991-1-1: (6.2)
Reduction due to the number of storeys ($n \geq 3$) by coefficient α_n

$$\alpha_n = \frac{2 + (n-2)}{n}\psi_{0,i}$$

$$N = \sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

b) Column as members of a braced bay frame

permanent — imposed (snow / live) — imposed (wind / possible sway imperfections)



imperfection load caused by gravity load (partly permanent)

$$N_{max} = \gamma_G G_k + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

Minimal (in some cases tension may result):

$$N_{min} = \gamma_{G,inf} G_k - \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

c) Columns as member of a continuous frame

Internal forces: *N*, *M*, *V*.

Column cross-sections

1. Axial force only (compression) (advantageous $i_y \approx i_z$)

Tension only:

2. Axial force and bending moment N, M_y or N, M_y, M_z

(biaxial bending)

Steel column at “Freedom Tower” building, New York

Column splices:

Position limited by:

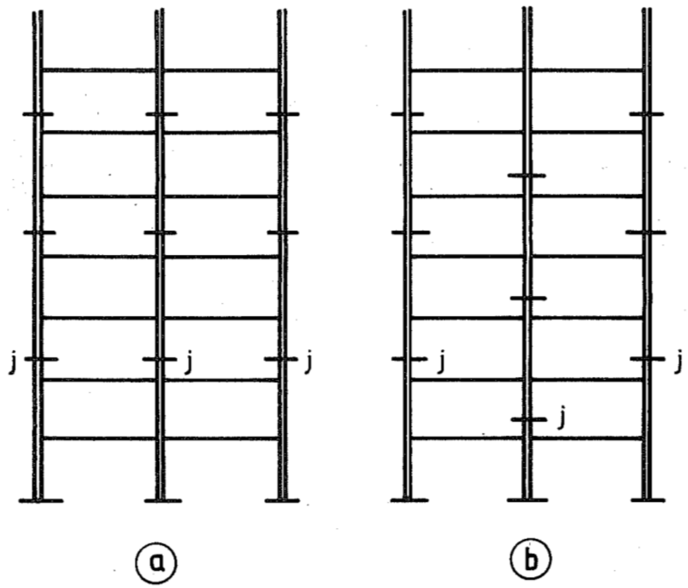
a) **Maximal member length:** 2 ÷ 3 storeys (bar length 12 m, max. 18 m)

b) **Easy assembly:** about 1/3rd above the beam

c) **Column buckling:** within the central half of the length ($\chi \approx 1$)

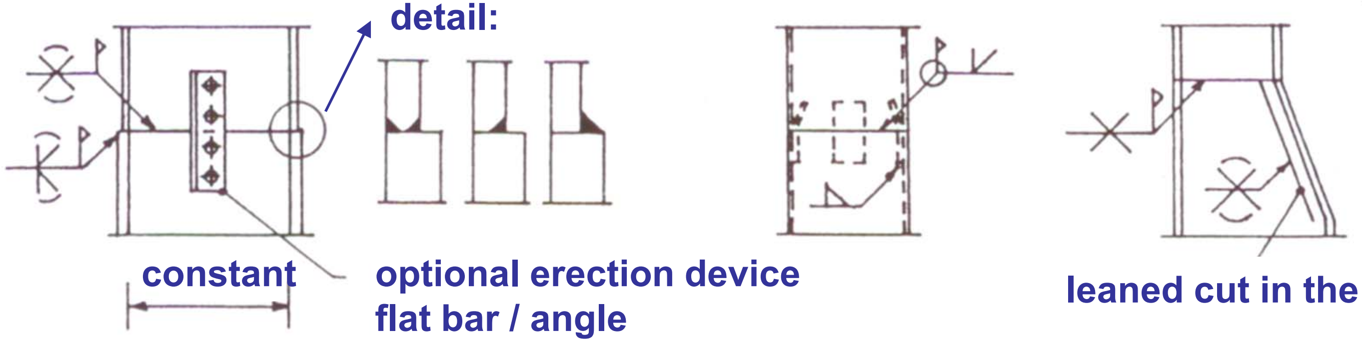
d) **Change of the section size (may be within the bolted splice).**

- Columns are delivered at the construction site in sections
- Sections are jointed on site by means of welded or bolted connections
- Splice connections can be at same level (a) or staggered (b)



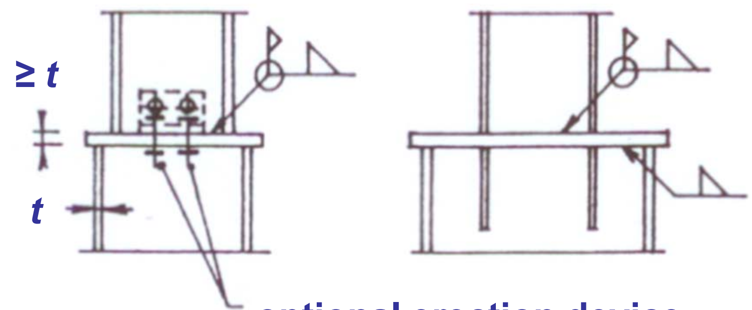
Welded splice (field weld, cut normal to the member length, eventually frontal milling)

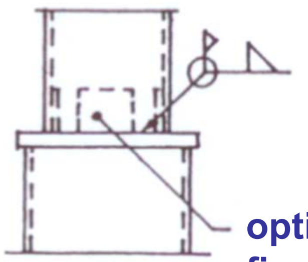
1. Butt weld



2. Fillet weld, flange plate splice

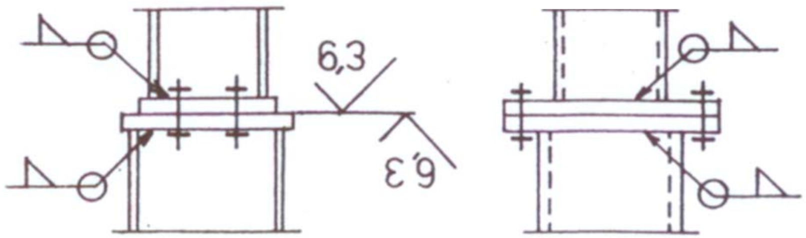
(not suitable for columns in tension – lamellar tearing)





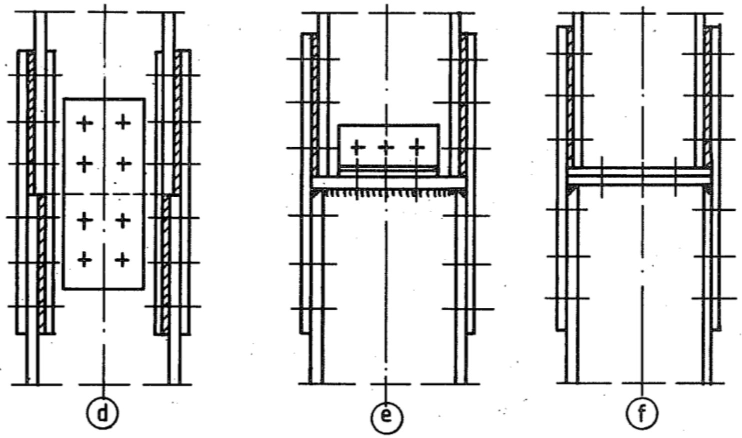
3. Bolted, flange plate splice

hollow section



Columns sections are different

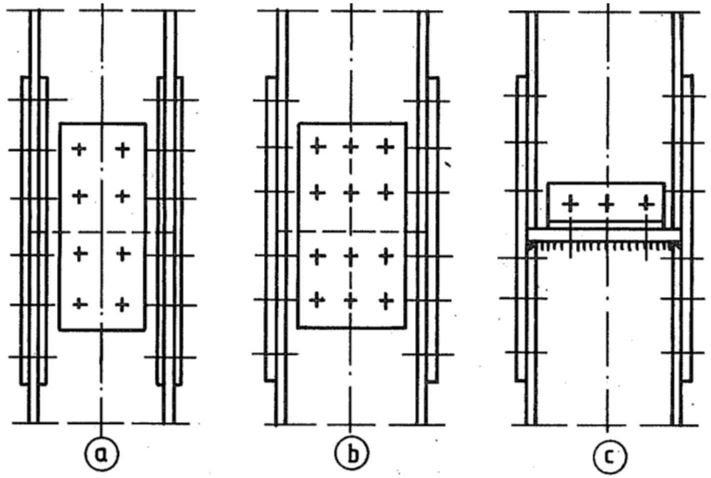
d) double cleat bolted connection
e), f) single cleat and cap plate bolted connection



Bolted spliced connection

Columns sections are identical

a) double cleat bolted connection
b) single cleat bolted connection
c) single cleat and cap plate bolted connection



Splice connection verification

1) **Connections usually designed for contact bearing only. Where:**

a) **connection within the mid half-height:**

b) **small slenderness** ($\lambda = L_{cr} / i < 80$),

c) **small eccentricity** (bending moment respectively):

$$M_{\mathrm{Ed}} < \frac{N_{\mathrm{Ed}} h}{2}$$

where *h* is taken conservatively as the overall depth of the smaller column

Contact bearing connection is designed (eg. weld or bolts) for shear only (if any)

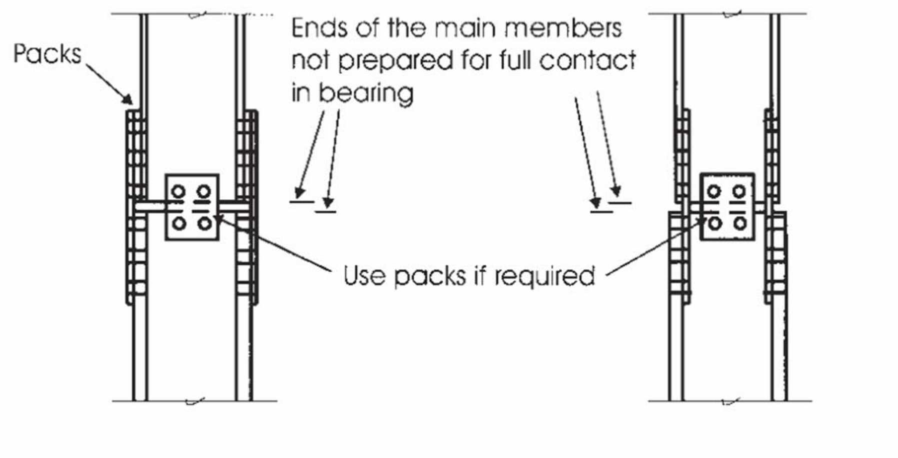
2) **Resistance of connections in tensions or with larger moments must be verified.**

Eg. for splice the flange covers plates and their fasteners should be checked for:

$$F_{Ed} = \frac{M_{\mathrm{Ed}}}{h} - \frac{N_{\mathrm{Ed.compressive}}}{2}$$

Or column splices with ends not prepared for bearing - figure

$$F_{Ed} = \frac{M_{\mathrm{Ed}}}{h} + \frac{N_{\mathrm{Ed.compressive}}}{2}$$

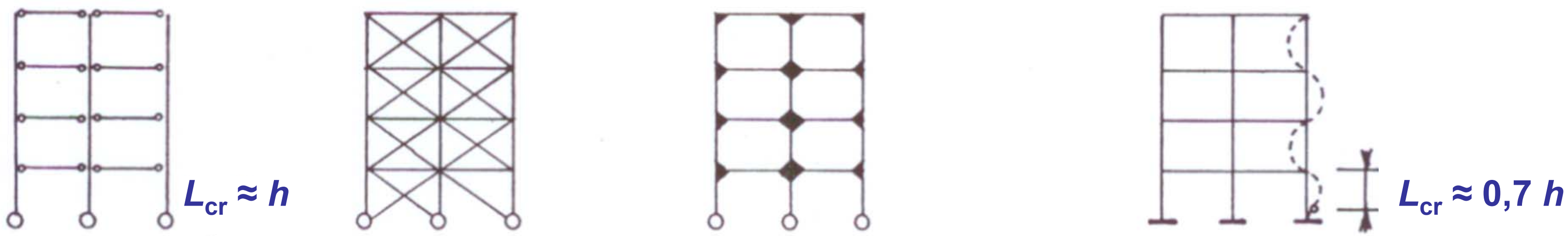


External flange cover plates   Internal flange cover plates

Column base

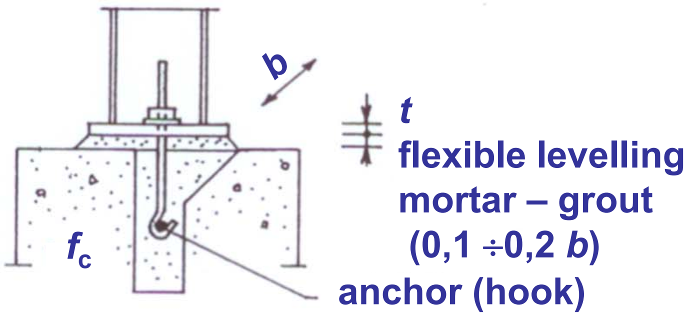
Base plates transfers the load to a concrete block.

Base plates
- **pinned base - plain bases (free rotation)**
- **fixed base - stiffened or gusseted base (moment bearing)**



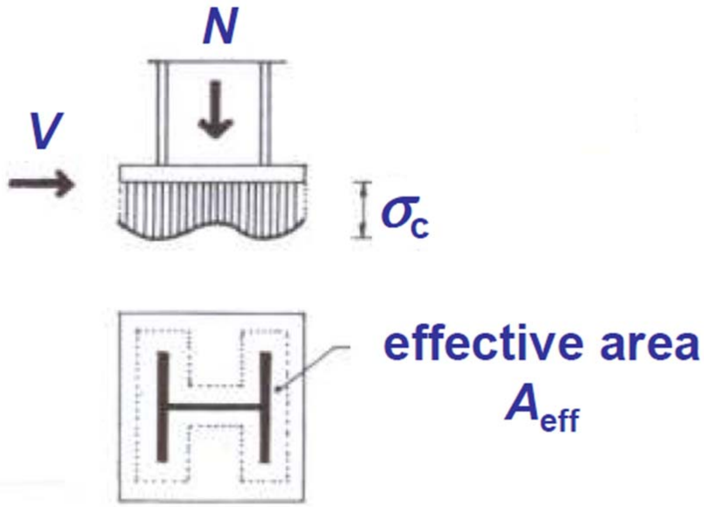
Pinned base

Usually not completely free rotation (small rotations assumed).



fixed for assembly

Design and resistance verification



Eurocode procedure:
Effective area A_{eff} and strength of concrete under concentrated compression including grout f_{jd}.

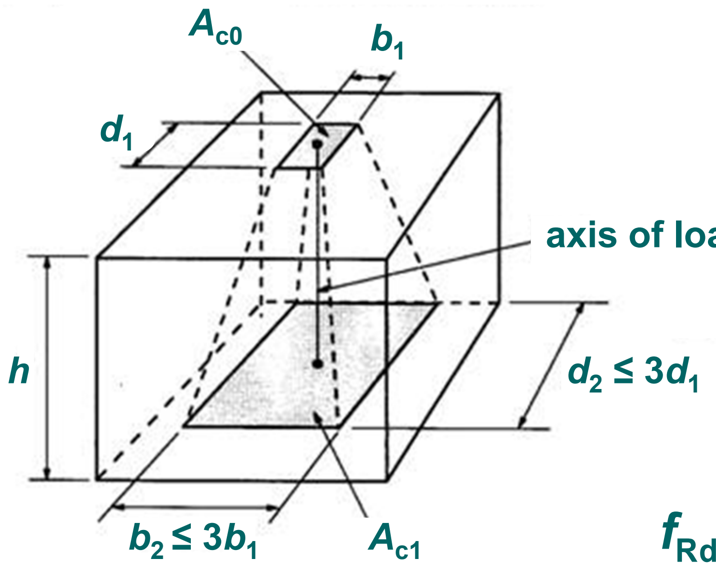
Minimal effective area: $A_{eff} \geq \dfrac{N_{Ed}}{f_{jd}}$

Design bearing strength of concrete: $f_{jd} = \beta_j \, f_{Rdu}$

concentrated design strength of concrete (EN 1992) (refers to f_{Rdu})

Foundation joint material coefficient β_j = 2/3 if:
- grout thickness ≤ 0,2 b
- f_{ck} grout ≥ 0,2 f_{ck} concrete

Concentrated design strength of concrete f_{Rdu} (EN 1992-1-1):



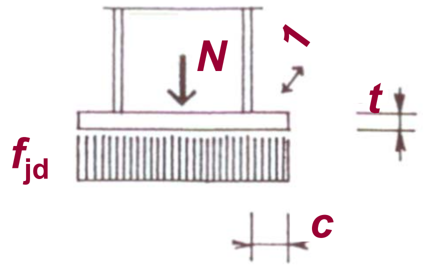
$$f_{Rdu} = f_{cd}\sqrt{A_{c1} / A_{c0}} \leq 3,0\ f_{cd}$$

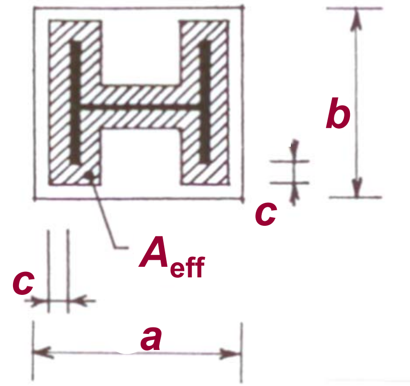
A_{c0} - loading area (A_{eff}, may be considered by A - the base plate area)

A_{c1} - maximal load delivery area of shape similar to A_{c0} at depth h

$h \geq (b_2 - b_1)$
$h \geq (d_2 - d_1)$
} means load delivery at 45º

Effective area





For elastic behaviour of the base plate:

bending moment: $m = \frac{1}{2} f_{jd} c^2$

bending resist.: $m = \frac{t^2}{6} f_y / \gamma_{M0}$

results in:

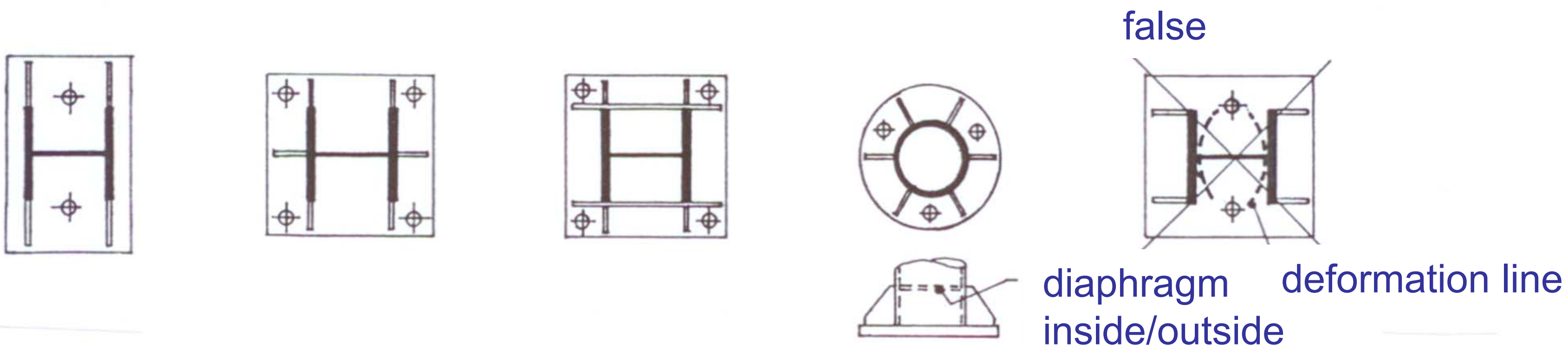
$$c = t \sqrt{\frac{f_y}{3\, f_{jd}\, \gamma_{M0}}}$$

The verification procedure is iterative:

1. **choosing of base plate dimensions *a* x *b*** (eg. $A \approx N_{Ed}/f_{cd}$),
2. **design bearing strength of concrete** f_{jd},
3. **choosing of plate thickness *t* → determining of *c* (section expand),**
4. **necessary effective area verification:** $A_{eff} \geq N_{Ed}/f_{jd}$,
5. **possible refinement of plate dimensions *a* x *b* or thickness *t*. → 2.**

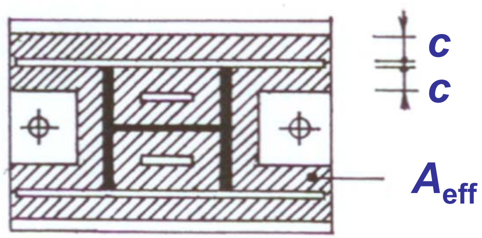
Stiffened base

Base plate thickness usually under $t \leq 50$ mm. If not satisfactory, stiffeners are used:



Design of the base plate:

- effective area procedure, extension *c* of the stiffeners area included:

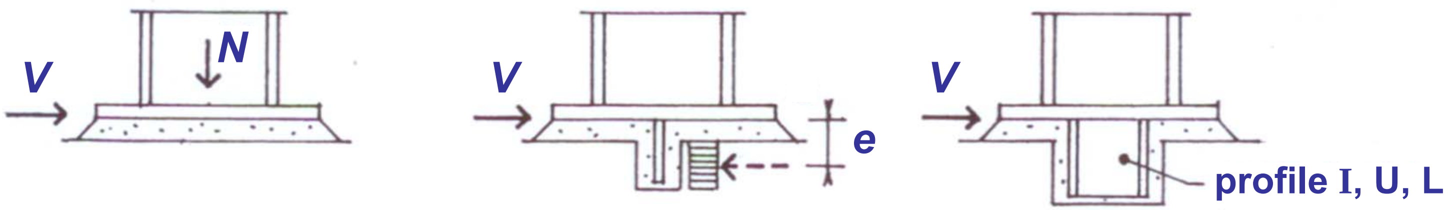


extension of the section + stiffeners area

Note.: Stiffeners loaded by *M*, *V*. (at the connection to the column)

Shear resistance for base plates

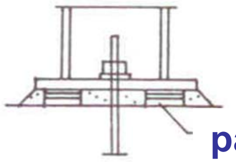
- **friction** (compressive reaction): $V \leq C_{f,d} N_{c,Ed}$ **(friction coefficient $C_{f,d}$ = 0,2)**
- **shear in the anchor bolt:**
  may be designed as additional to the friction
  EN 1993-1-8 gives resistance for *n* bolts = $n\,F_{vb,Rd}$
  (where $F_{vb,Rd}$ is reduced resistance in shear and bearing)
  in cases where the bolt holes are not oversized
- **block or bar shear connector:**
  may be designed as additional to the friction
  verification of internal forces (V, $M = V\,e$)



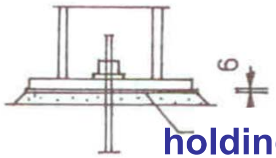
Column assembly at the concrete block

- **packs** (packs made of plate or preferably flat bar),
- **holding plate** ($t \approx 6$ mm, for base plates less than ≈ 500 mm, placed together with the anchor bolts embedded to the concrete, holes D+5 mm),
- **anchor bolt nut** (for smaller base plate dimensions).

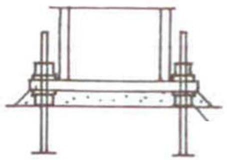
Grout $p \approx 0{,}1b$ filled form plate sides or using a hole in the base plate (diameter min. 70 mm).



packs



holding plate



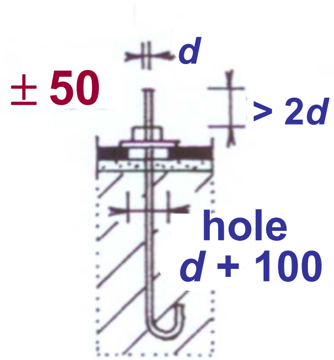
anchor bolt nut

Anchor bolts

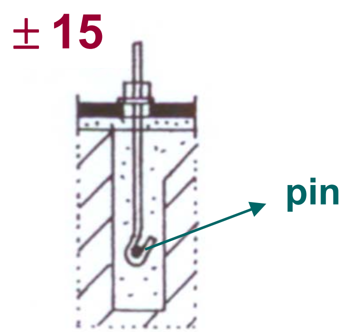
a) **Non-structural (no significant load, resp. compression only):** common for typical columns of multi-storey buildings.

b) **Load-bearing:** tension mostly, by bending or for tensile columns.

Non-structural (smaller load):
diameters M16 ÷ M30



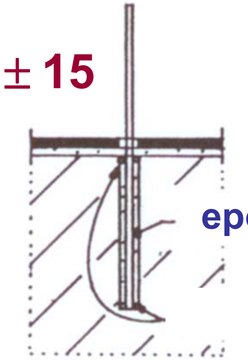
a) Anchor bolts embedded in concrete
- tolerances ± 50 mm
(for bolts interconnected by plate tolerance ± 15 mm)



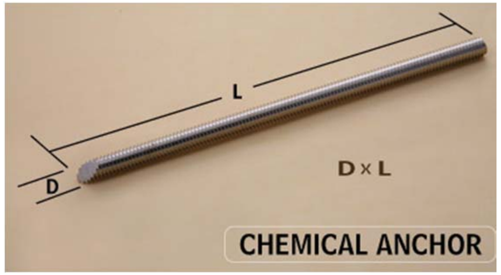
b) Additionally encased anchor bolts
- tolerance ± 15 mm

c) Bonded anchor bolt - chemical (not suitable for permanent tension – relaxation of stress)

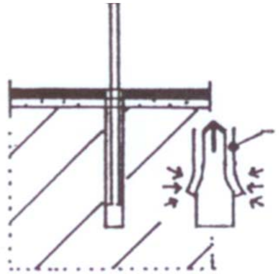
± 15



epoxide



d) Mechanical anchor

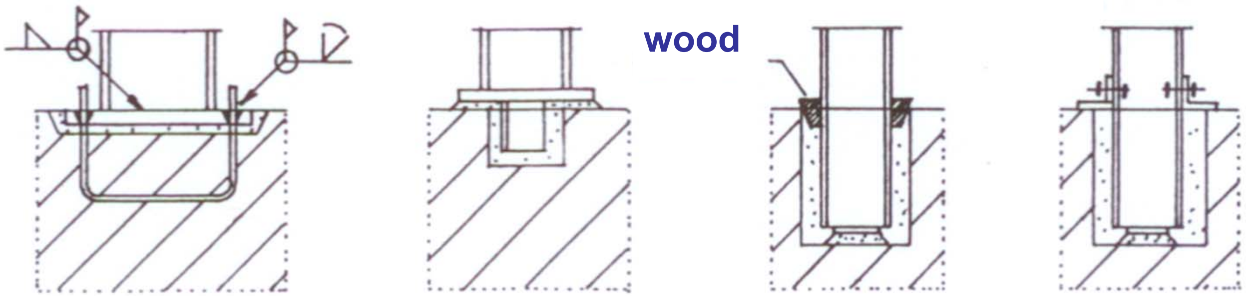


anchor unit opened

e) Other: to steel reinforcement, without bolts, pocket base

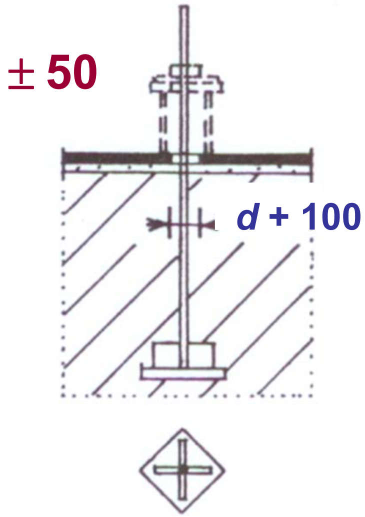
rarely

wood

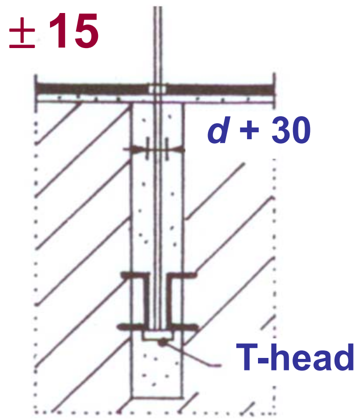


Load-bearing: bolts M30 ÷ M100

a) Anchor bolts embedded in concrete
- **tolerances ± 50 mm**
  (for plate interconnected bolts ± 15 mm)



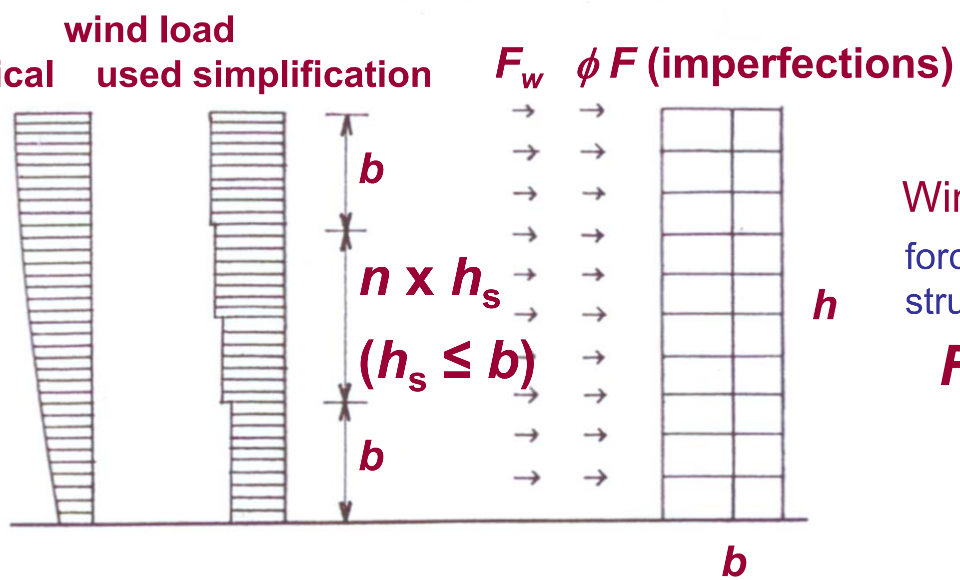
b) Additionally encased anchor bolts with T head
- **tolerances ± 15 mm**



Note: For small load previously presented anchor may be used as well.

Bracings

Horizontal load: **- wind,**
- sway imperfections,
- seismicity.



Wind pressure acting on area A_{ref}:

force coefficient for the structure.

peak velocity pressure

$$F_w = c_s c_d c_f q_{p(Z)} A_{ref}$$

structural factor ($c_s c_d$)

For framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth, the value of $c_s c_d$ may be taken as 1. (otherwise natural frequency is expected to be low and resonate with some part of the wind spectra - simplified procedure in EN 1991-1-4).

Bracings

Structural systems used to resist lateral loads:

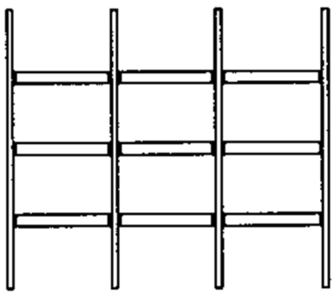
a) continuous or wind-moment frames,

b) shear walls,

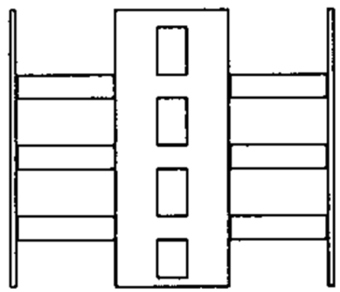
c) braced-bay frames.

Combinations of these systems may also be used (dual frame structures).

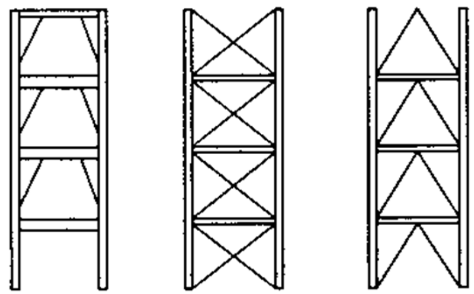
(a)



(b)



(c)



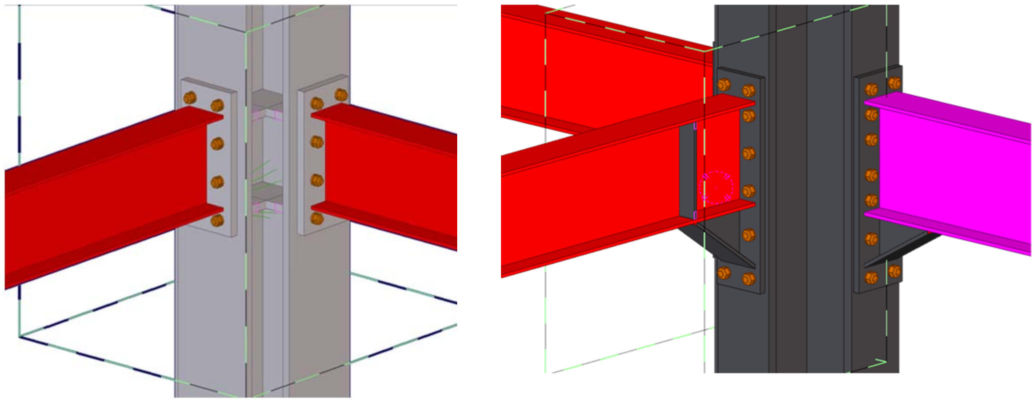
Eccentric **X braced** **Inverted V**

Continuous or wind-moment frames

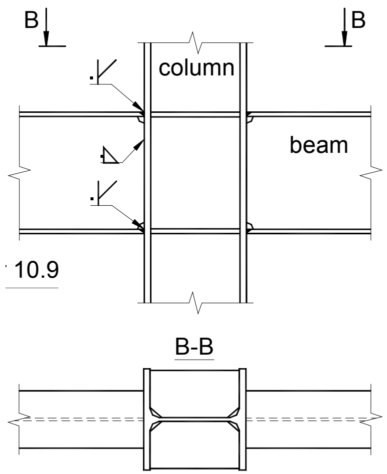
- With rigid moment-resisting connections between beam and columns.
- Only sufficient frames to satisfy the performance requirements.
- Advantage: internal adaptability
- Disadvantage:
  - Generally, less stiff (or more frames needed) than other bracing systems.
  - Increased fabrication for complex framing connections
  - Increased site connection work, particularly if connections are welded, limited in application.
  - Columns are larger to resist bending moments.

Beam – column connection:

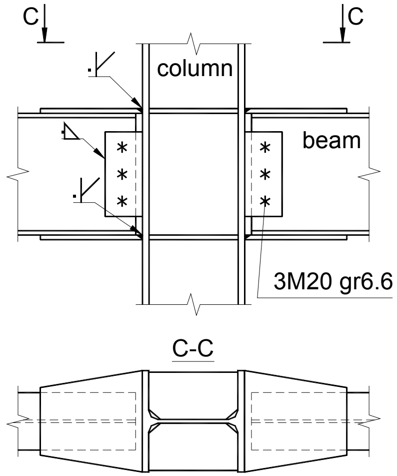
- Bolted (see beam connection detailing)
- Welded



Bolted joint

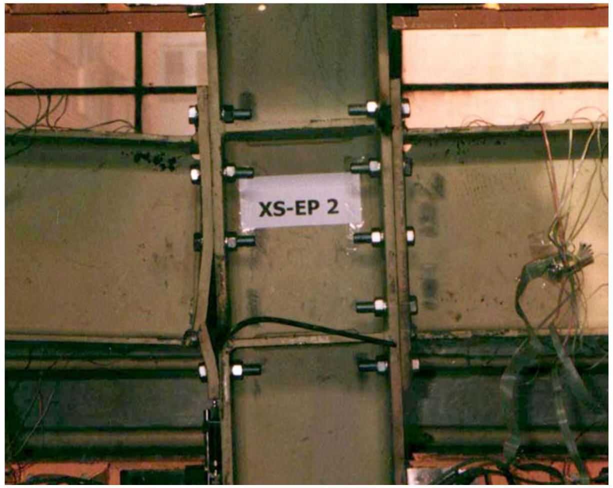


Welded joint



Cover plates welded joint

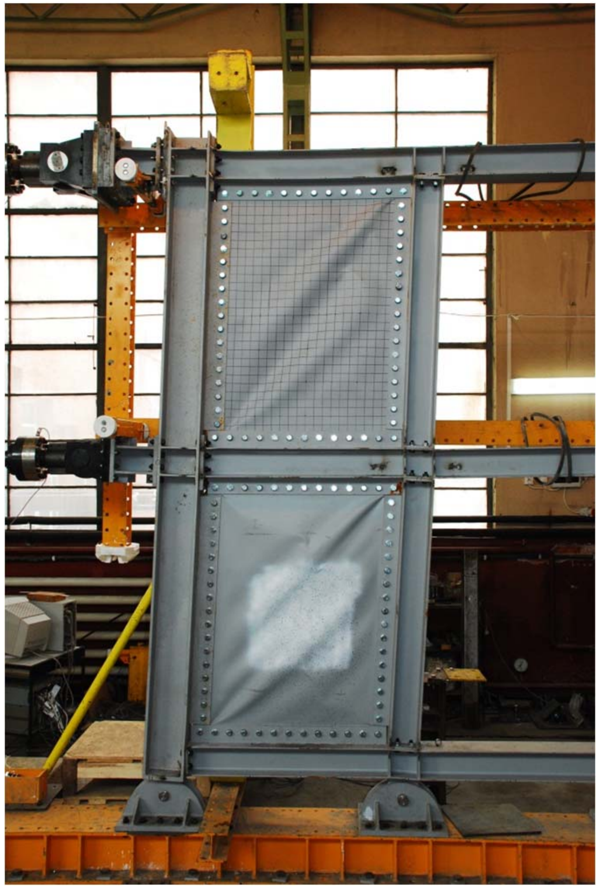
Example of beam-to-column joints – failure modes



Shear walls (steel, concrete)

Reinforced concrete walls

- Constructed to enclose lift, stair and service cores
- Generally possess sufficient strength and stiffness to resist the lateral loading.
- Advantages:
  - Very rigid and highly effective.
  - Act as fire compartment walls.
- Disadvantages:
  - The construction is slow and less accurate than steelwork.
  - Difficult to modify in the future.
  - Difficult to provide connections between steel and concrete to transfer the large forces generated.



Steel shear walls

- May be used instead of concrete walls
- High initial stiffness and good ductility (high dissipative seismic system)
- May be integrated in the steel framing system
- Disadvantageous behaviour in case of fire

Braced-bay frames

Act as vertical trusses which resist the wind and seismic loads by cantilever action.

Advantages:

- **All beam-to-column connections are simple.**
- **The braced bays are concentrated in location on plan.**
- **The system is adjustable if building modifications are required in the future.**
- **Bracing can be arranged to accommodate doors and openings for services.**
- **Bracing members can be concealed in partition walls.**
- **They provide an efficient bracing system.**

Disadvantages:

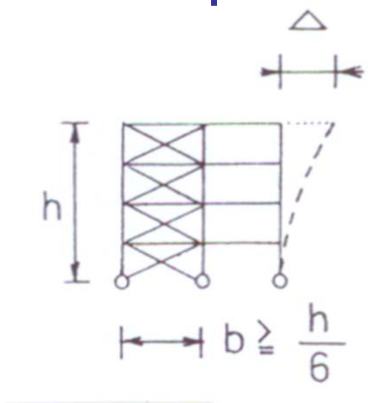
- **Diagonal members with fire proofing can take up considerable space.**

Design of the bracing:

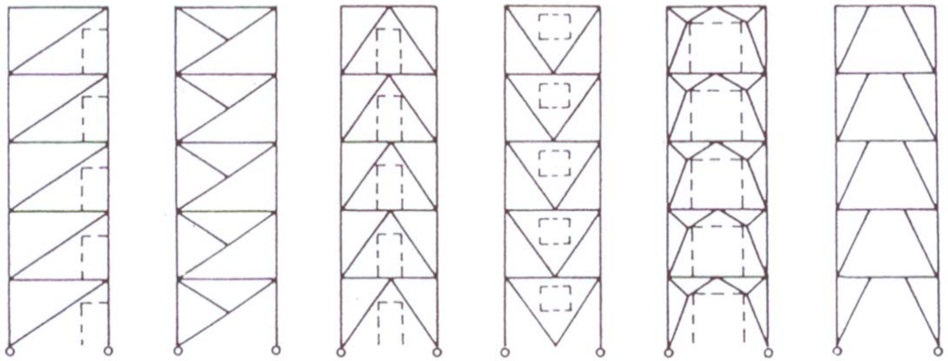
- **ULS – columns, diagonals, connections.**
- **SLS – horizontal deflection**

$$\Delta \leq \frac{h}{500}$$

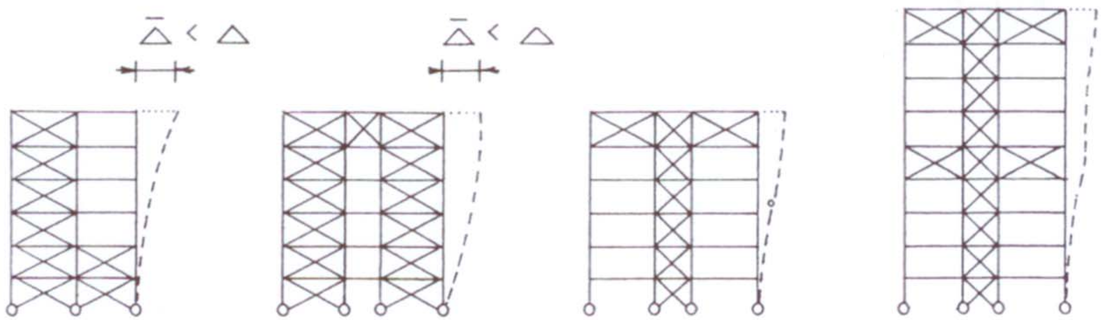
General limitation for wind condition



Possible geometry:



Increased stiffness geometry:



frame like deflection

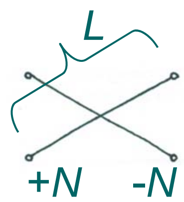
'S' shape

Diagonal member cross-section:

tensile

compressed

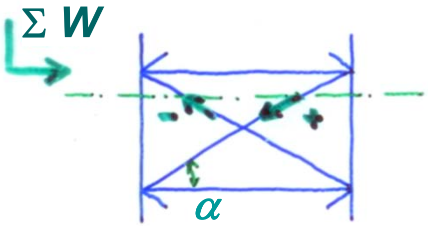
For connected diagonals:



L_{cr} in-plane = $L/2$

L_{cr} out-of-plane = $L/2$

Internal forces
(symmetric geometry, antisymmetric load, considering the horizontal load only):

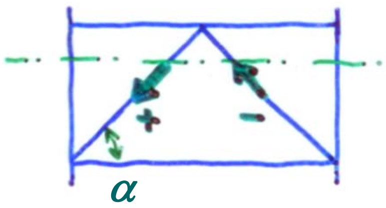


$$D \cong \pm \frac{1}{2} \frac{\sum W}{\cos \alpha}$$

(axial deformation neglected)

For low buildings compressed diagonal member may be neglected (lower stiffness – bigger deflection):

$$D = + \frac{\sum W}{\cos \alpha}$$

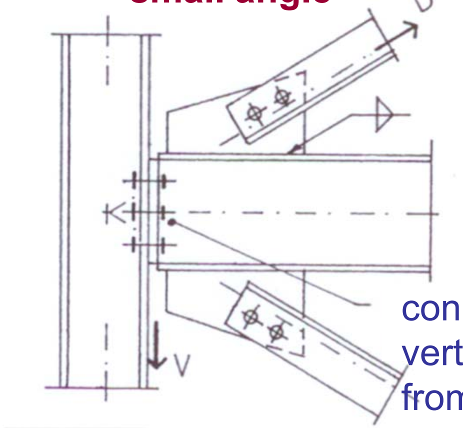


$$D \cong \pm \frac{1}{2} \frac{\sum W}{\cos \alpha}$$

Special requirements - to be used as part of the seismic resistant system – see EN 1998-1

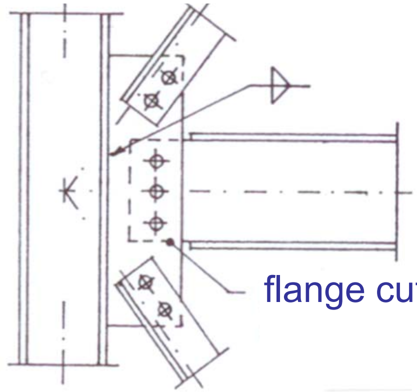
Detailing:

small angle



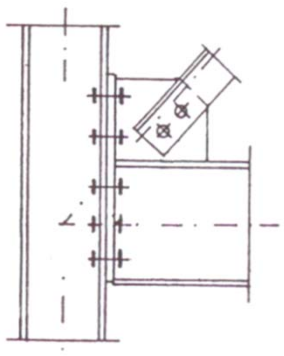
connection carries also the vertical component of reactions from diagonals

bigger angle

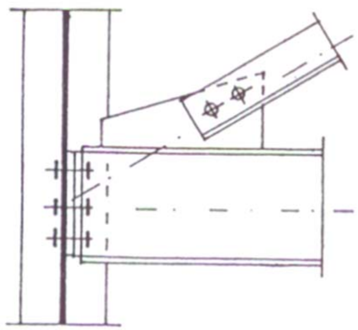


flange cutt

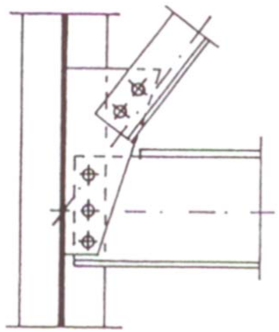
analogically for V - brace



semi-rigid connection



small angle



bigger angle

Global analysis - effects of deformed geometry of the structure

1. first-order analysis, using the initial geometry of the structure

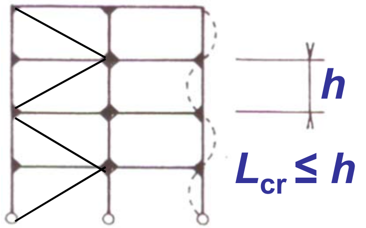
$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$$

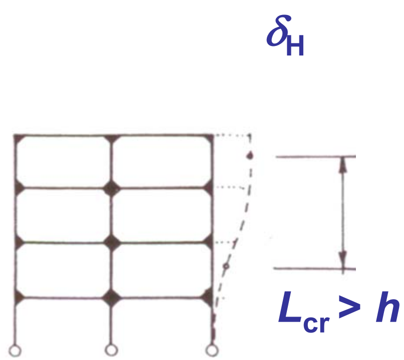
 for members:

- $N_{cr}/(\gamma_M N_{Ed}) \geq 25$ no buckling.
- Buckling length taken system length (distance between the nodes/supports is generally conservative).

2. second-order analysis, taking into account the influence of the deformation of the structure

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} < 10$$





or equivalent column method:

Possible buckling consideration directly from LBA:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{Af_y}{\alpha_{cr} N_{Ed}}} \rightarrow \chi$$

(existing tables and expressions)

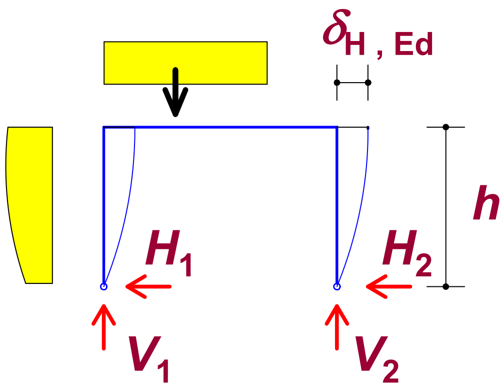
Global analysis of frames and complex multistorey structures

Global analysis depends on both geometry and loadings → different for each loading combination !!

1. First-order analysis structures ($\alpha_{cr} > 10$):

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$$

Note: For given loading F_{Ed} the α_{cr} results from FEM.



For sway mode failure approximately (simple frames):

$$\alpha_{cr} = \left(\frac{\sum H_{Ed}}{\sum V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$$

At the same time the slenderness of all members must fulfil:

$$\bar{\lambda} \geq 0{,}3\sqrt{\frac{A\,f_y}{N_{Ed}}}$$

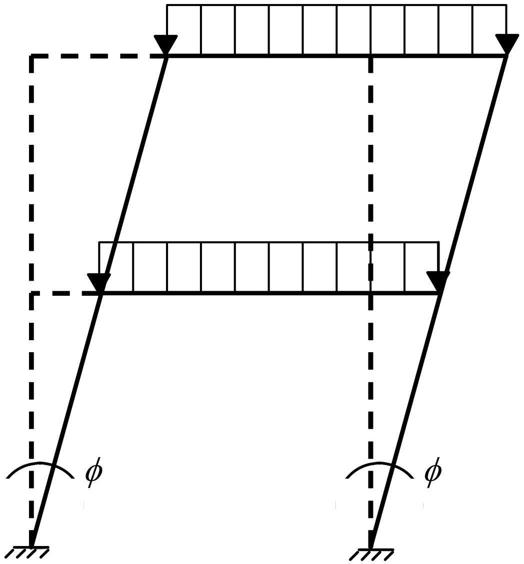
In plane frames this shall be applied at each floor level, the lowest value decides.

The check of all members with buckling length equal to the system length (between joints) is then conservative (acc. to Eurocode if $\alpha_{cr} > 25$ then $\chi = 1$).

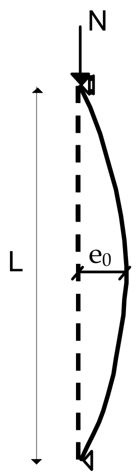
2. Second-order analysis structures (α_{cr} < 10):

In general three methods may be used:

a) Geometrical non-linear analysis with imperfections (GNIA).
Second order effects considering global and member imperfections are then included in resulting internal forces and moments. Check of individual members is done for simple compression or bending (without χ , χ_{LT}, no stability check is necessary). The solution is demanding on software, introduction of imperfections and evaluation of results.

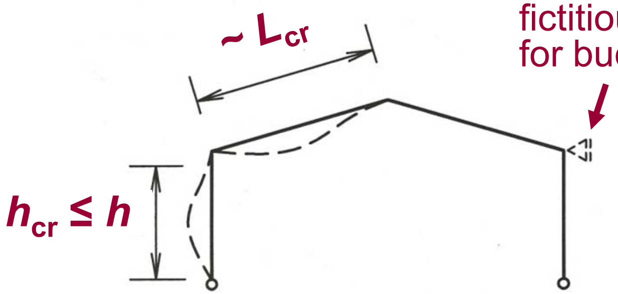


(a) Sway imperfections



(b) Member imperfections

b) **Geometrical non-linear analysis (GNIA) with global imperfection only (using frame sway or equivalent horizontal forces). Members shall be checked on buckling (i.e. 2nd order effect and influence of imperfections), taking the system length as buckling length (e.g. *h*, *L*/2).**



fictitious support for subsequence check of members for buckling

Note: for small slopes (up to 15º or flat rafters) the L_{cr} equals distance of columns.

If $3 \leq \alpha_{cr} < 10$ and sway buckling mode (corresponds to α_{cr} determined from approximate relation above) the 2nd order effects from sway may be evaluated approximately in accordance with following method:

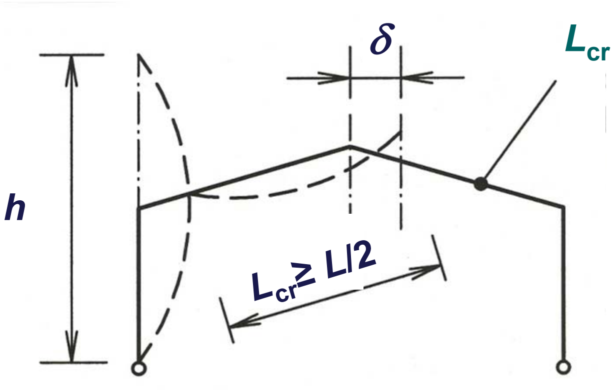
b1) Second order sway effects due to vertical loads may be calculated by increasing the horizontal loads H_{Ed} (e.g. wind) with an equivalent loads $V_{Ed}\,\phi$ due to imperfections and other possible sway effects according to first order theory by second order factor:

$$\frac{1}{1-\dfrac{1}{\alpha_{cr}}} \geq 1$$

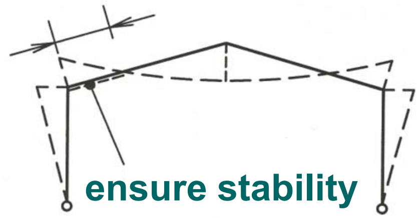
c) **Frequently (classical method) is used first order theory without any imperfections and members are checked with equivalent global buckling lengths (using relevant reduction coefficients χ):**

$h_{cr} = \beta h$

given in many references

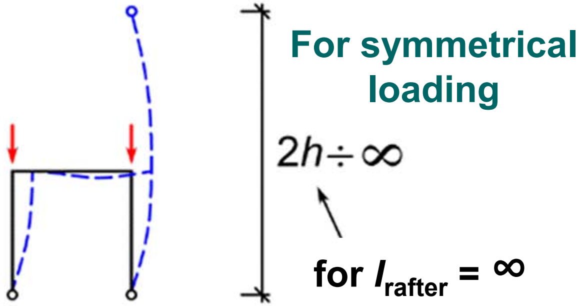


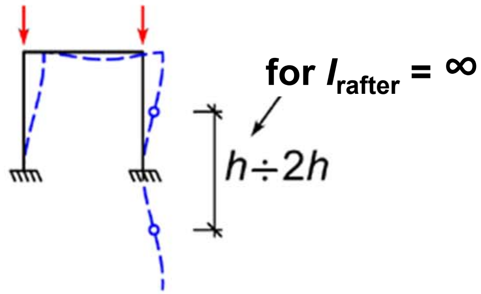
L_{cr} determined similarly as for columns or to use system length and increase moments from horizontal loadings by about ~ 20%.



ensure stability of free flange ! !

Typical global buckling lengths (for sway buckling mode):





Global buckling lengths are given in tables or formulas in literature.
They may be preferably determined from critical loading N_{cr} by common software of corresponding α_{cr} (corresponding to buckled member) as follows:

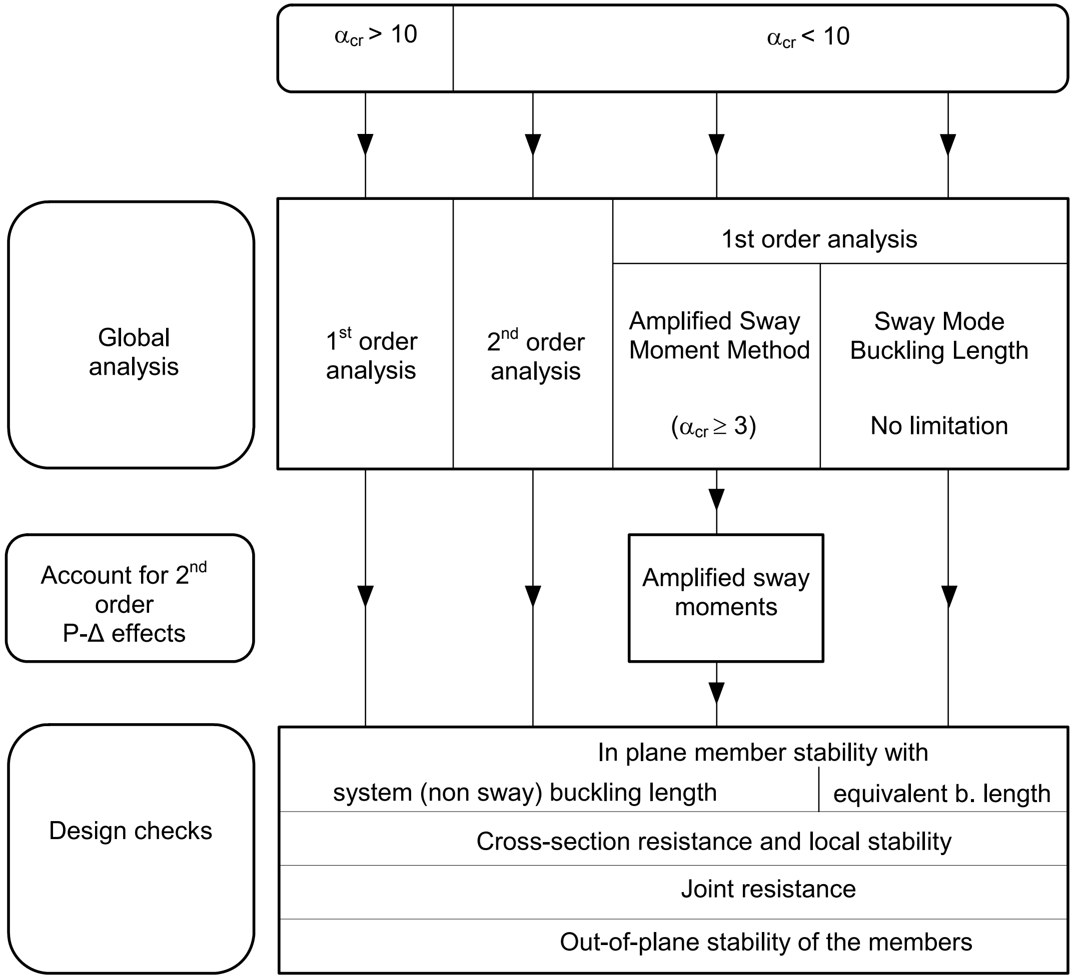
$$L_{cr} = \sqrt{\frac{\pi^2 E I}{N_{cr}}} = \sqrt{\frac{\pi^2 E I}{\alpha_{cr} N_{Ed}}}$$

Note:

1) Using α_{cr} from approximate formula (i.e. for sway buckling mode), the minimum buckling length equals the system length.
1) Mind the modification of cross sections after check:

results in different α_{cr} and hence also L_{cr}.

Global analysis - summary of possible approaches according to EN 1993-1-1

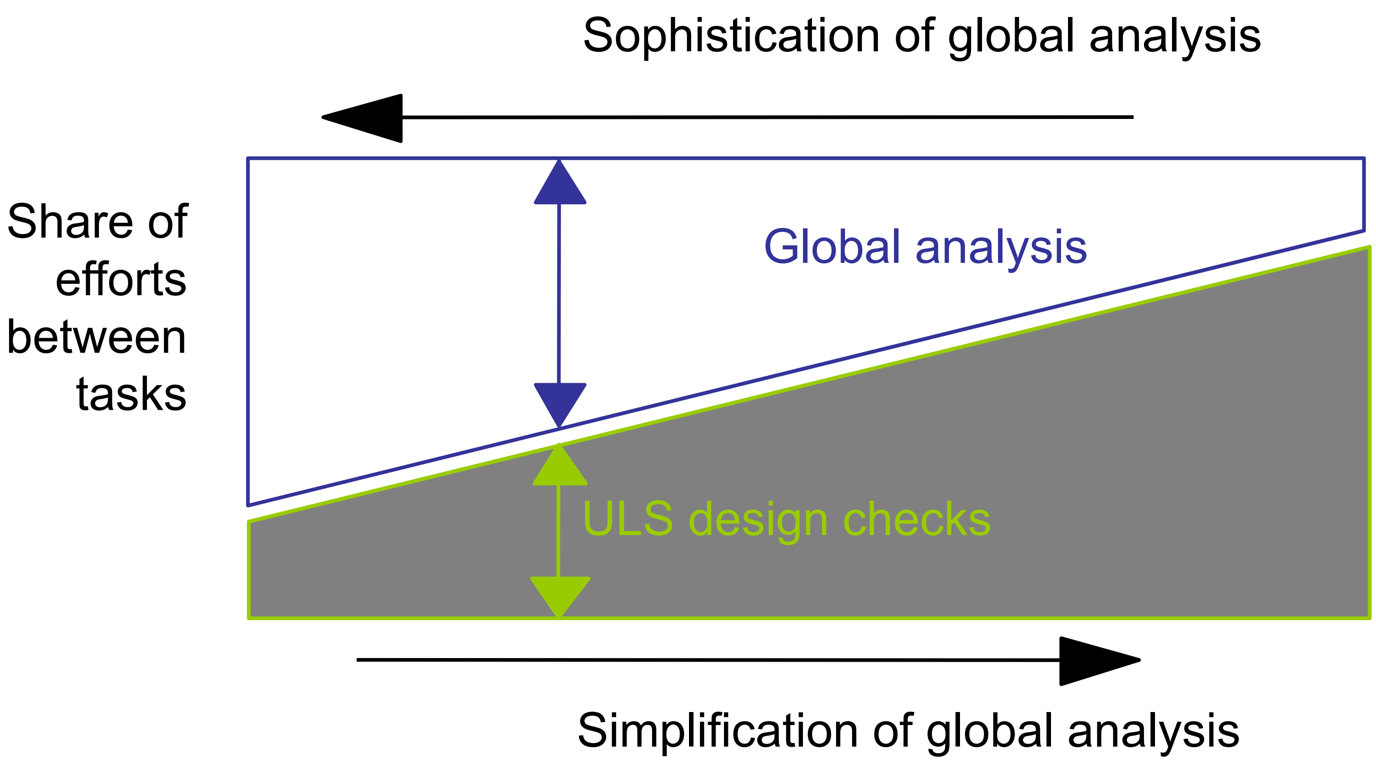


If GMNIA is used (both sway and member imperfections, no stability check (usually just in-plane) is necessary.

Global analysis - summary of possible approaches according to EN 1993-1-1

- Frame design consists in global frame analysis followed by a series of design checks.
- Due account shall be made of sway (frame) and member imperfections, where necessary.
- To which extent consideration of so-called second order effects (*P*-Δ effects) is required depends on sensitivity to second order effect (effect of deformed geometry). It is relative to the load combination in consideration.
- For $\alpha_{cr} > 10$, the P- Δ effects are negligible so that first order analysis suffices.
- For $\alpha_{cr} \leq 10$, the P- Δ effects need to be accounted for either by performing a second order analysis or by using simplified approaches referring to first order analysis and indirect approximate account for P- Δ effects.
- The most sophisticated the method of global analysis, the less the number and sophistication of the design checks still to be conducted.

Global analysis - summary of possible approaches according to EN 1993-1-1



Overall Design Task = Analysis + Design Checks

This lecture was prepared for the 1st Edition of SUSCOS (2012/14) by Prof. Josef Macháček (CTU) and Michal Jandera, PhD. (CTU).

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