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A P P U N T I

STUDENTE: Pirro Giulia

MATERIA: Theory an Design of Reinforced and Prestressed Concrete Structure - Prof. Fantilli

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REINFORCED AND PRE-STRESSED CONCRETE

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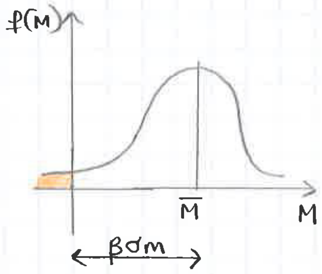

(●) probability of failure
The distance from the avarage is $\beta\sigma_M$
β : reliability index
σ_M : standard deviation of M

It's necessary to chose a suitable β to guarantee safety

⇒ Too difficult to find the real distribution of E and S so it's used a **SEMI-PROBABILISTIC METHOD** (Eurocode 2 or ACI 318 – USA –)

$E_d \leq R_d$

ACI - 318	Eurocode
$E_d = \mathcal{U}$	$E_d = \gamma_q \cdot Q$ → nominal value of loads
factoral loads	partial safety factor for loads
$R_d = \phi \cdot S_n$	$R_d = R_k / \gamma_m$
ϕ : reduction factor (<1)	R_k : characteristic value of strenght
S_n : nominal strenght	γ_m : partial safety factor for materials (>1)

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$\gamma_P \nearrow 1{,}3$ unfav. $\searrow 1$ fav.

$\gamma_q \nearrow 1{,}5$ unf. $\searrow 0$ fav.

All these coefficients could be different from country to country
↳ the max values is in Europe because of economy.
Higher is the value, high is the cost of the structure.
safety of the

ψ depends on the category of construction.
↳ There are tabs.

$G_1 + Q_1$ become $1{,}3\,G_1 + 1{,}5\,Q_1$ for example
↳ Similar to ACI 318:
$1{,}2\,G_1 + 1{,}6\,Q_1$

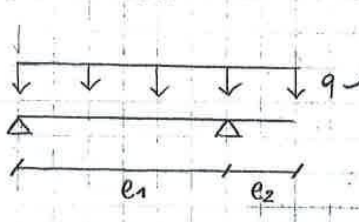
There are several combinations → 6 in European Codes
↓
For:
- Ultimate Limit States → Fundamental, Accidental, Seismic Combinations
- Serviceability Limit States → characteristic, Frequent, Quasi-Permanent combinations

Other loads:
- Eurocode 1-1.1 densities, self-weight and imposed loads
- " 1-1.2 fire actions
- " 1-1.3 snow loads
- " 1-1.4 wind loads

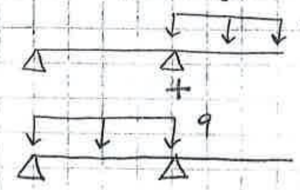
LOAD ARRANGEMENTS → we can fix the values of loads

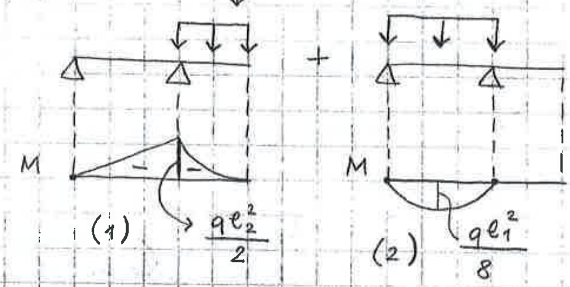
In the combinations, loads are contemporaneity with maximum effect

Simple structure:

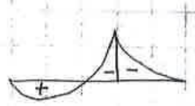


→ distributed load → its effect is the same of the effect of 2 loads:





When we put together these diagrams, we have 2 possibilities:

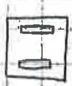
1. 

(1) + (2)

2. 

in the section we there are 2 moments:
- positive M
- negative M

→ M can change the part



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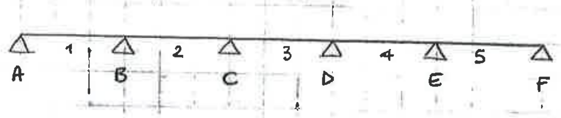
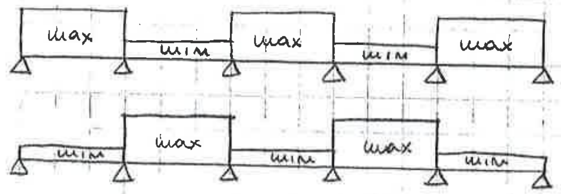
4/10/17



Ed
q_{max}
q_{min}

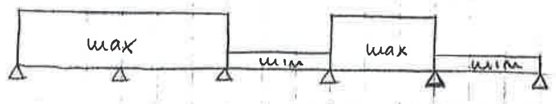
If we want to have the maximum positive bending moment, how can we do this/ obtain it?



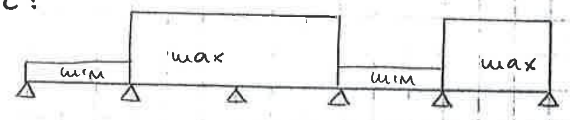
In 1, 3, 5.

In 2 and 4.

Where are the maximum negative bending moments? There are on the support:
on B:



on C:



In the support there aren't positive moment; only negative in all the situations.

on D and on E it's the same!

The continuous beam is a form of a floor for example.
We have to consider all the arrangements
The same is also true for shear (= taglio)

SEISMIC ACTION ~ HORIZONTAL → EC0 : eurocode 0

SPAN = campata
SPAN = campata

Action that arises due to earthquake ground motions.
-Generally, dead lo.

stiffness, mass and viscous must be taken into account

c = property of materials

Vertical actions of seismic is not take into account
because they're lower than horizontal ones
and because
current constructions are generally ideated for vertical loads (are uniform but venerable)
VULNERABILI
brick blocks = PIGNATTE
↳ are light and reduce the weight of the floor.

In some cases they represent also for isolation if they're in polystyrol
↳ ~~not~~ NOT always

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Content of the course

- First part
  - Theory of elastic beams and frames (structural mechanics)
  - Theory of elastic plates and walls (structural mechanics)
  - Effects produced by explicit and implicit loads
  - Effects produced by seismic loads (in low intensity seismic zones)
- Second part
  - Materials characterization and durability problems
  - Design of beams and one-way slabs
  - Design of two-way slabs and shear walls
  - Design of short and slender columns
  - Serviceability requirements
  - Strut and Tie Modeling
  - Beam-Column Joints
  - Design for Torsion Resistance
  - Prestressed concrete structures

POLITECNICO DI TORINO 5

Content of the course

- Third part
  - Safety formats and conceptual design of reinforced concrete buildings
  - Effects of horizontal loads
  - Slabs for floors
  - $E_d \leq R_d$ in beams, columns and slabs
  - Beam-Column Joints
  - Serviceability requirements

POLITECNICO DI TORINO 6

Exam

- Before the examination, the report of the course project has to be completed (10 points) mandatory
- The exam consists of two parts:
  - Written test, consisting of the solution of two simple problems – with multiple choice answer (10 points) mandatory
  - Individual oral colloquium of about 20 minutes on the content of the course (10 points) not mandatory

POLITECNICO DI TORINO 7

Combination at ULS

- Fundamental
  - $\gamma_{G1}G_1 + \gamma_{G2}G_2 + \gamma_p P + \gamma_{Q1} Q_{K1} + \gamma_{Q2}Q_{K2}\Psi_{02} + \gamma_{Q3}Q_{K3}\Psi_{03} + ...$
- Accidental design situations
  - $G_1 + G_2 + P + A_d + Q_{k1}\Psi_{21} + Q_{k2}\Psi_{22} + ...$
- Seismic situations
  - $E + G_1 + G_2 + P + Q_{k1}\Psi_{21} + Q_{k2}\Psi_{22} +$

POLITECNICO DI TORINO 8

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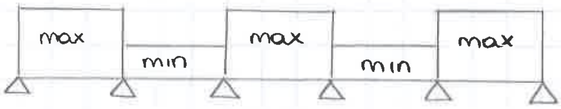
CONTINOUS BEAMS

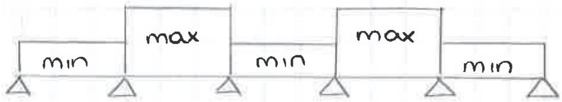
4/10/17

Ed (q_{max}, q_{min}) arrange load to obtain the max or the min M or T

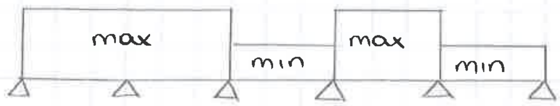
The maximum positive bending moment on 1, 3, 5 is due to:



The maximum positive bending moment on 2, 4 is due to:

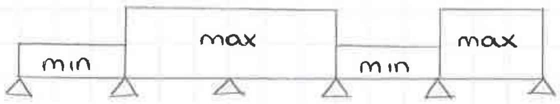


The maximum negative bending moment on B is due to:



in this case we also have the max shear in B (max cons ction reaction)

The maximum negative bending moment on C is due to:



And in the same way we can obtain the max negative bending moment on D and E

HORIZONTAL LOADS

sismic action

in ECO the definition of isismic action is action that arises due to earthquake ground motions

Generally, dead loads, live loads and those of snow depend on their mass (and gravity)

seismic loads depends on the mass, the stiffness and on the viscous damper of the structure and ground acceleration

the property to disipate sismic energy

All structures have stiffness and costant of viscous damper

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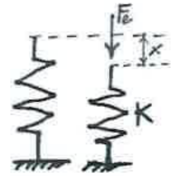
Stiffness

- The stiffness, k, of a body is a measure of the resistance offered by an elastic body to deformation
- A spring is generally used to illustrate k, which can be defined as the constant of proportionality between the applied elastic load $\mathbf{F}_e$ and the produced displacement **x**:

$$k = \frac{F_e}{x}$$

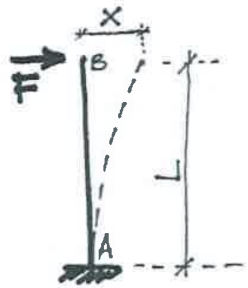


- In the standard International System of Units (SI), the vectors $\mathbf{F}_e$ and **x** are measured in Newton (N) and m, respectively, whereas the scalar k is measured in N/m.

POLITECNICO DI TORINO 5

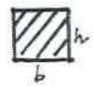
Observation

- Similarly to a dumper, a structure has the intrinsic capacity to dissipate oscillation (i.e., has a viscous damper constant c)
- Similarly to spring, a structure has a stiffness k. E.g. a cantilever beam, in which the end A has a fixed support, and a load P is applied in the free end B:



$$x = \frac{F\,L^3}{3E\,\mathrm{I}} \qquad I = \frac{b\,h^3}{12}$$

where:
E= elastic modulus of material
I = moment of inertia

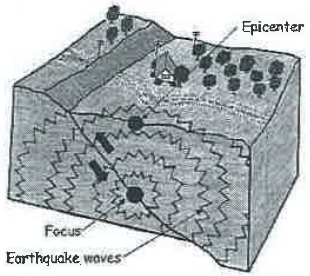


$$k = \frac{F}{x} = \frac{3E\,I}{L^3}$$

POLITECNICO DI TORINO 6

The earthquake

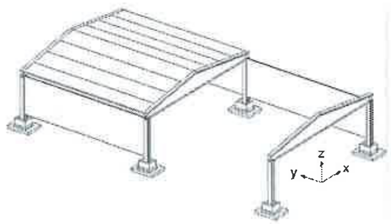
- Earthquakes results from the sudden movement of tectonic pates on the earth's crust.
- The movement takes place at fault lines, and the energy released is transmitted through the earth in the form of waves that cause ground motion many miles from the focus.
- Epicenter is the projection od the earth's crust of the focus.

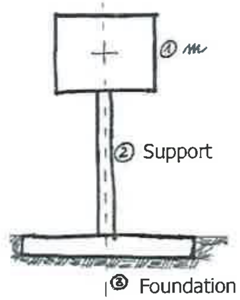


POLITECNICO DI TORINO 7

A single building

- From a structural point of view, a one-storey building (e.g., an industrial plant) can be outlined with:
- (1) A mass m (of the beams and the slabs of the roof)
- (2) The vertical supports (i.e., the columns)
- (3) The foundations





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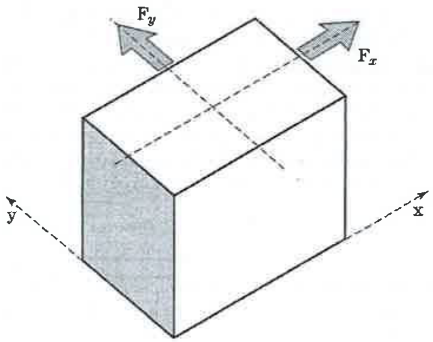
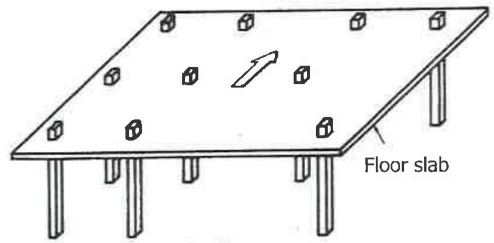
The horizontal loads

- They are generally applied, not simultaneously, in the main horizontal directions x and y of each floor (§ 4.3.3.1(7) EC8)



The horizontal loads

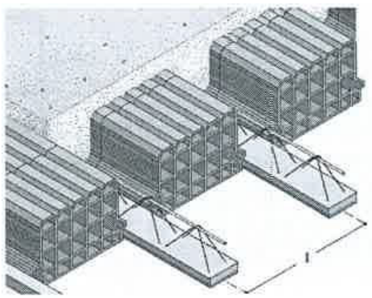
- The loads Fx and Fy on the floor slab, must be distributed to vertical elements (i.e., columns, shear walls).



- The in-plane stiffness of the floor slab shall be large enough with respect to the lateral stiffness of the vertical structural elements (so that a rigid diaphragm behavior may be assumed).

Rigid diaphragms

- A RC floor slab can be considered as rigid diaphragms (§ 7.2.6 NTC 2008):
  - When the thickness of the reinforced concrete topping layer should be not less than 40 mm if the span between supports is less than 8 m, or not less than 50 mm for longer spans



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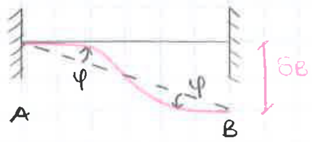
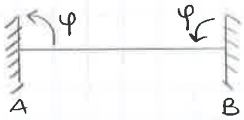
PROBLEM: calculate $K = R_B / \delta_B$ (very rigid floor, diaphragm) 4/10/17



we impose δ so R_B is unknown and the problem is to find it

Connecting the two ends, the real deformation can be obtained by two rotations of this line. In this way it's possible to find R_B
↳ considering δ_B very small

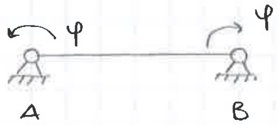
The statically indeterminated can be solved by MoF (METHOD OF FORCES)



$$\varphi = \frac{\delta_B}{L}$$

- Principal structural system without extra supports. It has to be statically determinated

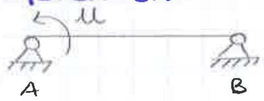
SYSTEM 0



without vertical load moment, shear and axial force are zero

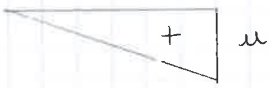
- Virtual system with the vertical reactions due to the extra supports, deleted in the system 0

SYSTEM ONE



we don't consider shear, but only bending moment

- **SYSTEM TWO**



$$M_B = \underbrace{M_0}_{=0} + X_1 M_1 + X_2 M_2$$

$$M_B = X_1 M_1 + X_2 M_2$$

↳ the equation of virtual work leads to compute X_1 and X_2

Internal work = external work

$$L_{INT}^{i,b} = L_{EXT}^{i,b}$$

$i = 1,2$

↳ we have to apply this equation twice

$$\int_L M_i \cdot \frac{M_B}{EI} \, dz = u\varphi \qquad \begin{cases} M_i = M_1 \\ M_i = M_2 \end{cases}$$

virtual forces · real displacement

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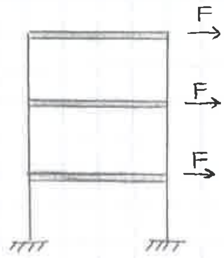
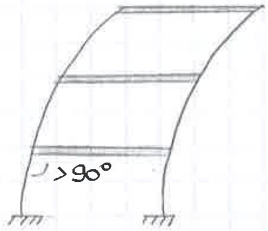
Frame : vertical and horizontal elements + horizontal forces



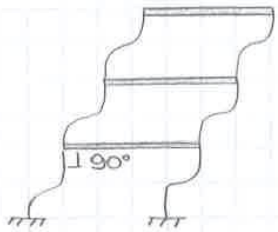
There are two kinds of deformations

1) **BENDING TYPE**

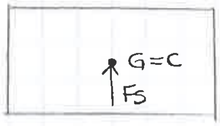


rigid diaphragm, different deformation → typical for reinforced concrete

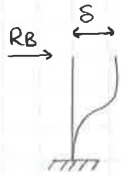
2) **SHEAR TYPE**



If the "floor" is rigid and rectangle, the force is applied in the center of the mass (in this case it is the same as centroid)



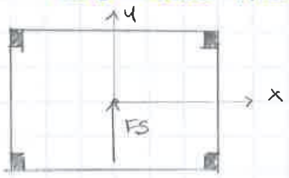
$F_S = m \cdot a$



This force has to be distributed to the vertical elements behind (columns) so through F_S we have to calculate R_B.

__1° CASE__

columns with the same geometry, are located simmetrically respect to the load F_S and have the same material



L, E, A, I = COSTANT PROPERTIES
↳ even k = cost

Because of the symmetry the only movement which is possible is along y (δ) and it's equal for all the columns because there aren't relative movements between the columns themselves.

$$R_B = \frac{F_S}{n}$$

n : number of colomns

It has to be the equilibrium too, so $F_S = \sum_{i=1}^{n} R_{B,i} = n R_B$

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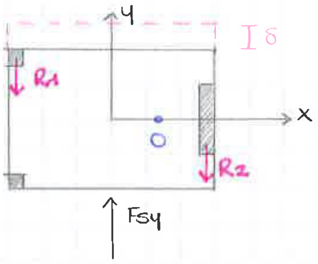
DISTRIBUTION OF HORIZONTAL LOADS

9/10/17

3rd CASE



Two type of columns
① a x a
② a x 2a

In we consider a rigid movement in y direction, δ is the displacement.
The reactions are

$R_1 = k\, I_{x1} = k \frac{a^4}{12}$

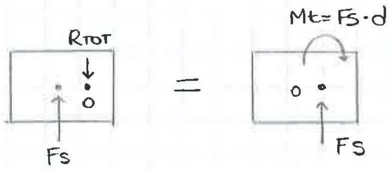
↳ directly proportional with moment of inertia

$R_2 = k\, I_{x2} = k \frac{a(2a)^3}{12}$

$R_2 = 8 R_1$

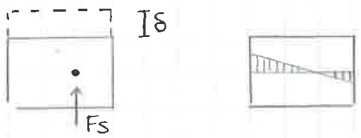
$R_{TOT} = 2R_1 + R_2$

↳ it should be equal to the applied load, due to equilibrium
↳ is located quite close to the bigger column (in the point O)
↳ the two forces are applied in different points (center of mass and O)



torsional moment if we observe it in the same plane of the diaphragm

If there is only Fs there is only a translation, with Mt there is only rotation around O. So the non simmetrical situation leads to a rotation and traslation

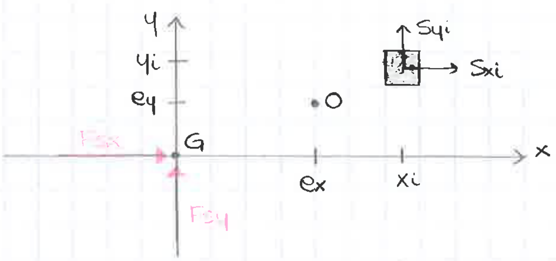


they have to be summed

So the distribution of load has to be changed because of rotation, the greatest movment is about column 1, for column 2 the displacements are in opposite sense.

⇒ In order to consider the translation and rotation we have to apply the <u>Theory of the center of tortion</u> (= point O)

Distribution of horizontal loads



S_{yi} : effect of the applied load in the y direction } for the i-th
S_{xi} : effect of the applied load in the x direction } column
G : center of mass
O : center of torsion

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$x_{oi} = x_i - \cancel{e_x} \quad x_i - a_x$
$y_{oi} = \cancel{y_i} - e_y \quad y_i - a_y$

We can define two movments due to rotation. a_y is the translation of the column in the y direction due to rotation
$a_y = x_{oi} \operatorname{sen}\alpha \simeq x_{oi} \cdot \alpha$
↳ little displacements and rotations so $\operatorname{sen}\alpha \simeq \alpha$
$a_x = - y_{oi} \operatorname{sen}\alpha \simeq - y_{oi} \cdot \alpha$
↳ translation of the column in the x direction due to rotation

⟹ The rotation becames composed on two translations

$$S_{xi}^{(M)} = K_{xi} \cdot a_{xi} = -\frac{12\, E I_{yi}}{L^3}\, \alpha \cdot y_{oi}$$

L: lenght or high of the column

$$S_{yi}^{(M)} = \frac{12 E I_{xi}}{L^3} \cdot \alpha\, x_{oi}$$

Problem: α is onknown. It can be defined through equilibrium to rotation around point O.

$$M_t = \sum_{i=1}^{n} S_{yi}^{(M)} \cdot x_{oi} - \sum_{i=1}^{n} S_{xi}^{(M)} \cdot y_{oi}$$

$$M_t = \frac{12 E\alpha}{L^3} \underbrace{\left[\sum_{i=1}^{n} I_{xi} \cdot x_{oi}^2 + \sum_{i=1}^{n} I_{yi} \cdot y_{oi}^2 \right]}_{\text{geometrical property } \psi}$$

$$M_t = \frac{12 E\alpha}{L^3} \cdot \psi$$

$$\alpha = \frac{M_t L^3}{12 E \psi}$$

When α is known we can substitute in the before expressions and so compute $S_{xi}^{(M)}$ and $S_{yi}^{(M)}$

$$S_{xi}^{(M)} = - M_t \cdot \frac{I_{yi} \cdot y_{oi}}{\psi}$$

$$S_{yi}^{(M)} = M_t \cdot \frac{I_{xi} \cdot x_{oi}}{\psi}$$

$$\begin{cases} S_{xi}^{TOT} = F_{sx} \cdot \dfrac{I_{yi}}{\sum_{J=1}^{n} I_{yJ}} - M_t \dfrac{I_{yi} \cdot y_{oi}}{\psi} \\ S_{yi}^{TOT} = F_{sy} \dfrac{I_{xi}}{\sum_{J=1}^{n} I_{xJ}} + M_t \dfrac{I_{xi} \cdot x_{oi}}{\psi} \end{cases} \qquad i\text{-th} = 1, \dots, n$$

⟹ We can distribute the loads on the columns (valid all the possible distribution, it's the GENERAL CASE)
↳ it's a simplified appraach, but the real situation is closer to these solutions if some hypotesis are verified:
HP:
- Linear-elastic regime ($\sigma = E\,\varepsilon$)
- we neglet all the stiffness of non-structural elements (the resistent element are only the columns, not the others)
  ↳ in some cases these non-structural elements should be considered

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$$I_{xi} = \frac{B_{xi}(B_{yi})^3}{12} \qquad I_{yi} = \frac{B_{yi}(B_{xi})^3}{12}$$

- compute e_x and e_y

$e_x = x_O - x_G$

$e_y = y_O - y_G$

- compute the effect of translation

$$S_{xi}^{(F)} = \frac{F_{sx} \dfrac{I_{yi}}{L_i^3}}{\sum_{J=1}^{n} \dfrac{I_{yJ}}{L_J^3}} \qquad S_{yi}^{(F)} = \frac{F_{sy} \dfrac{I_{xi}}{L_i^3}}{\sum_{J=1}^{n} \dfrac{I_{xJ}}{L_J^3}}$$

- effects of rotation

$M_t = F_{sy}\, e_x - F_{sx}\, e_y$

$x_{oi} = x_i - x_o$

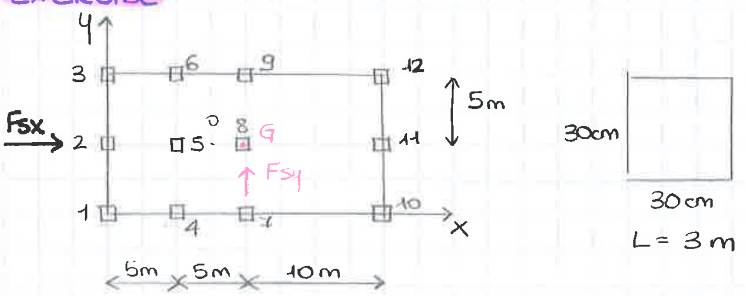
$y_{oi} = y_i - y_o$

- compute ψ

$$\psi = \sum_{i=1}^{n} \frac{I_{xi}}{L_i^3} x_{oi}^2 + \frac{I_{yi}}{L_i^3} \cdot y_{oi}^2$$

$$S_{xi}^{(M)} = -M_t \cdot \frac{I_{yi}}{L_i^3} \cdot \frac{y_{oi}}{\psi}$$

EXERCISE



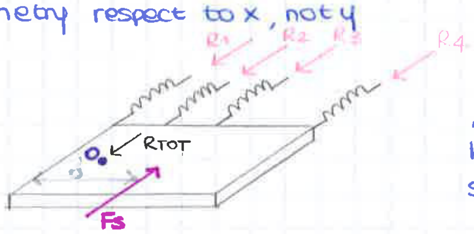
$F_{sy} = 50$ kN
$F_{sx} = 0$ kN

N.B.
The datas haven't unit measures, but they has to be cohirent

number of columns = 12
$B_{xi} = 0{,}3$ } for every
$B_{yi} = 0{,}3$ } columns

(1)	(0;0)	(5;0)	(10;0)	(20;0)
	(0;5)	(5;5)	(10;5)	(20;5)
	(0;10)	(5;10)	(10;10)	(20;10)

G (10;5)
O (8,75;5) close to the most rigid part
$e_x = 1{,}25$
$e_y \cong 0$
Symmetry respect to x, not y



All the columns have the same stiffness (k)

In O the resultant (R1 + R2 + R3 + R4) acts.

$R_{TOT} \cdot d = R_1 \cdot 0 + R_2 \cdot 5 + R_3 \cdot 10 + R_4 \cdot 20$

$R_i \propto k$

$$d = \frac{k \cdot 0 + k \cdot 5 + k \cdot 10 + k \cdot 20}{k + k + k + k} = \frac{35}{4} = 8{,}75$$

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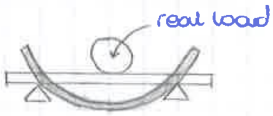
Effects of loads (Ed) can be producted not only by directy real load, but 11/10/17
through the **THEORY OF ELASTIC COACTIONS** we can study other effects.
→ state of sress in a body in which there is a sort of constraint between some parts and the rest of the body
→ constraint is a limitation of movement which is allowed and so produced a state of stress



→ this state can be removed, removing the load or the wape
→ state of stress without the application of any external load

⇒ typical for some situations of renforced concrete
- Temperature
- Shrinkage (= ritiro del cls) due to evaporation of water
- Prestressed concrete

⇒ This theory can be also used for **NON-LINEAR BEHAVIOUR**

It is developed by **GUSTAVO COLONNETTI (1896-1968)**
→ if we impose a strain on a structure/body, generally this strain is incompatible

$\{\bar{\varepsilon}\}$ = imposed strain $\qquad \{\varepsilon\}_{el}$ = elastic strain

→ elastic strain is generated within the body so we can compute the total one

$\{\varepsilon\}_{TOT} = \{\bar{\varepsilon}\} + \{\varepsilon\}_{el}$ = compatible

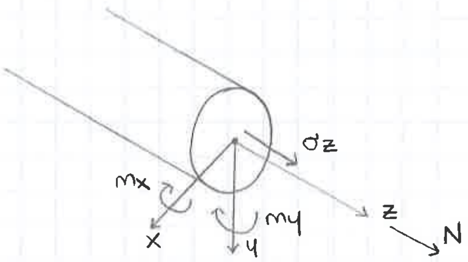
in general it is composed on the 6 components of strain

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \end{Bmatrix} = \{\varepsilon\}$$

In an elastic body a state of stress is generated too.
$\{\sigma\} = [D]\{\varepsilon\}_{el} = [D](\{\varepsilon\}_{el} - \{\bar{\varepsilon}\})$

If we remove the imposed strain we don't have elastic strain nor state of stress generated because of it.

This theory has to be applied to beams and Saint-Venant's body.



Shear deformations can be neglected, but the most important effects are produced by orthogonal tensions (σz).
The distribution of σz should be balanced by the actions of the cross sections (normal load, bending moment around y and bending moment around x)
→ equilibrium is composed on 3 equations.

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$$\int_{A} m_1 \, x \, dA + \int_{A2} m_2 \, x \, dA = 0 = S_y$$

↳ first order moment of area, with respect to the centroid axis so it is zero

$$S_x = \int_{A1} m_1 y \, dA + \int_{A2} m_2 y \, dA = 0$$

This relations can also be writen by matrixes :

$$\begin{Bmatrix} N \\ M_x \\ M_y \end{Bmatrix} = E_0 \begin{bmatrix} A_0 & 0 & 0 \\ 0 & I_{x0} & I_{xy0} \\ 0 & I_{xy0} & I_{y0} \end{bmatrix} \begin{Bmatrix} \lambda \\ \mu_x \\ \mu_y \end{Bmatrix} - \begin{Bmatrix} E_0 \int_{A1} m_1 \bar{\varepsilon}_z \, dA \\ E_0 \int_{A1} m_1 \bar{\varepsilon}_z \cdot y \, dA \\ E_0 \int_{A1} m_1 \bar{\varepsilon}_z \cdot x \, dA \end{Bmatrix}$$

↳ we can obtain the state of strain inverting the matrixes

$$\begin{Bmatrix} \lambda \\ \mu_x \\ \mu_y \end{Bmatrix} = \frac{1}{E_0} \begin{bmatrix} 1/A_0 & 0 & 0 \\ 0 & I_{y0}/D & -I_{xy0}/D \\ 0 & -I_{xy0}/D & I_{x0}/D \end{bmatrix} \begin{Bmatrix} N \\ M_x \\ M_y \end{Bmatrix} + \begin{bmatrix} 1/A_0 & 0 & 0 \\ 0 & I_{y0}/D & -I_{xy0}/D \\ 0 & -I_{xy0}/D & I_{x0}/D \end{bmatrix} \begin{Bmatrix} \int_{A1} m_1 \bar{\varepsilon}_z \, dA \\ \int_{A1} m_1 \bar{\varepsilon}_z \, y \, dA \\ \int_{A1} m_1 \bar{\varepsilon}_z \, x \, dA \end{Bmatrix}$$

$$D = I_{x0} I_{y0} - I_{xy0}^2$$

If we applied N, M_x and M_y the state of strain can be produced by this loads or by this imposed strain

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$$\bar{\lambda} = \frac{1}{A_0}\int_{A_1} m_1 \, \bar{\varepsilon}_z \, dA = \frac{B}{A_0}\int_{-H/2}^{0} \bar{\varepsilon}_z \, dy$$

$dA = B\,dy$

$$\bar{\lambda} = -\frac{B}{A_0}\frac{2{,}5 \cdot 10^{-6}}{mm}\int_{-H/2}^{0} y\,dy = +\frac{B}{BH}\cdot\frac{2{,}5\cdot 10^{-6}}{mm}\cdot\frac{H^2}{8} = \frac{400}{8}\cdot 2{,}5\cdot 10^{-6} =$$
$$= 12{,}5 \cdot 10^{-5}$$

$$\bar{\mu}_x = \frac{1}{I_{x0}}\int_{A_1} \bar{\varepsilon}_z \cdot y \, dA = \frac{B}{I_{x0}}\int_{-H/2}^{0} \bar{\varepsilon}_z \cdot y^2 dy = -\frac{B \cdot 2{,}5\cdot 10^{-6}}{I_{x0}}\int_{-H/2}^{0} y^2 dy$$

$dA \to B\,dy$

$$\frac{I_{y0}}{I_{y0} I_{x0} - I_{xy}^2 = 0}$$

$$\bar{\mu}_x = -\frac{B \cdot H^3}{24\, I_{x0}}\cdot\frac{2{,}5\cdot 10^{-6}}{mm} = -1{,}25\cdot 10^{-6} \,/mm$$

$$\bar{\mu}_y = \frac{1}{I_{y0}}\int_{A_1} \bar{\varepsilon}_z \cdot x \cdot dA = \frac{1}{I_{y0}}\underbrace{\int_{-B/2}^{+B/2} x\,dx}_{0}\int_{-H/2}^{0} \bar{\varepsilon}_z \, dy$$

$\bar{\mu}_y = 0$

↳ the rotation is only in one plane

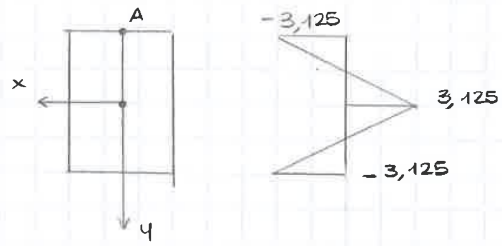
$\sigma_z = E\,(\lambda + \mu_x \cdot y + \mu_y \cdot x - \bar{\varepsilon}_z)$

↳ these are total deformations in this case they are only imposed

$\sigma_z = f(y)$ — sum of different linear functions

↳ it changes along y as the strains

$$\sigma_z = 25'000\left(12{,}5\cdot 10^{-5} - \frac{1{,}25\cdot 10^{-6}}{mm}\cdot y - \bar{\varepsilon}_z\right)$$



A) $y = -\frac{H}{2}$

$$\sigma_z = 25'000\left(12{,}5\cdot 10^{-5} + \frac{1{,}25\cdot 10^{-6}}{mm}\cdot 200\,mm - \frac{2{,}5\cdot 10^{-6}}{mm}\cdot 200\,mm\right)$$

$= -3{,}125$ MPa

G) $y = 0$

$\sigma_z = 25000 \cdot 12{,}5\cdot 10^{-5} = 3{,}125$ MPa

B) $y = \frac{H}{2}$

$$\sigma_z = 25000\left(12{,}5\cdot 10^{-5} - \frac{1{,}25\cdot 10^{-6}}{mm}\cdot 200\right) = -3{,}125 \text{ MPa}$$

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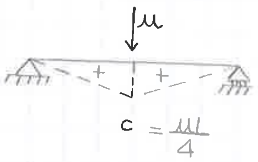
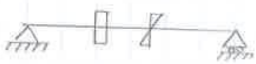
20/10/17

From the previous exercise:
$\bar{\lambda} = 12,5 \cdot 10^{-5}$
$\bar{\mu}_x = -1,25 \cdot 10^{-6}$ 1/mm
$M = 0$
$N = 0$
$T = 0$

We want to compute f_c
$L_{ext}^{a,b} = L_{int}^{a,b}$
a: virtual system



b: real system

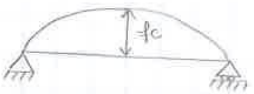


$M_a = -\frac{\mu L}{4}$
$N_a = 0$

$M_b = 0$
$T_b = 0$
$N_b = 0$

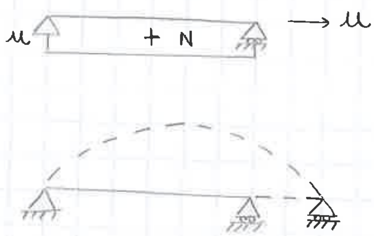
$L_{ext}^{a,b} = \mu \cdot f_c$
$L_{int}^{a,b} = \int_L M_a \cdot \mu_b \, dz = \int_L M_a \cdot \bar{\mu} \, dz = \bar{\mu} \int_L M_a \, dz$
$\mu_b = \underbrace{\mu_{el}}_{=0} + \bar{\mu}$
$\mu f_c = \bar{\mu} \int_L M_a \, dz$ area of triangle of M_a
$\mu f_c = \bar{\mu} \frac{\mu \cdot L^2}{8}$
$f_c = \bar{\mu} \frac{L^2}{8} = -1,25 \cdot 10^{-6} \cdot \frac{4000^2}{8} = -2,5$ mm

DEFLECTION



We don't have the contribution of λ, only curvation deflect in this case

$L_{ext}^{a,b} = \mu \cdot \eta_b$
$L_{int}^{a,b} = \int N_a \cdot \lambda_b \, dz = \mu \bar{\lambda} \cdot L$
$\lambda_b = \underbrace{\lambda_{el}}_{0=} + \bar{\lambda}$
$\eta_b = \bar{\lambda} L = 12,5 \cdot 10^{-5} \cdot 4000 = 0,5$ mm



$M_a = 0$
$N_a = \mu$

		$\exists$ modification of:	
		kinematical conditions	statical conditions
type of structure	statically determinate	YES	NO
	statically indeterminate	YES 4	YES 3

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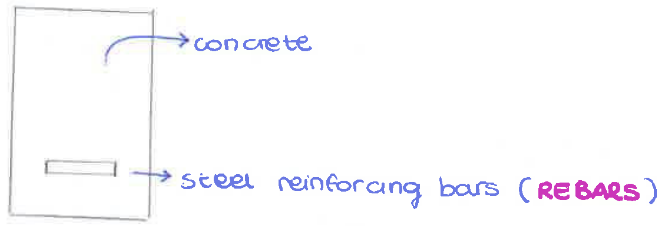
$Ed \leq Rd$ (reinforced concrete RC) 25/10/17



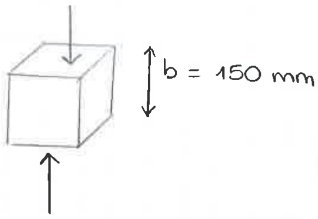
Mechanical properties of concrete and rebars

CONCRETE

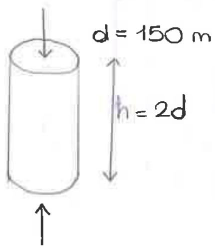
1. It's an artificial rock made by:
   - water (18%)
   - cement (10%)
   - aggregates (70%) ~ coarse and fine (sand 25%)
   - air (6%)
   - additives
2. Density = 24 kN/m³

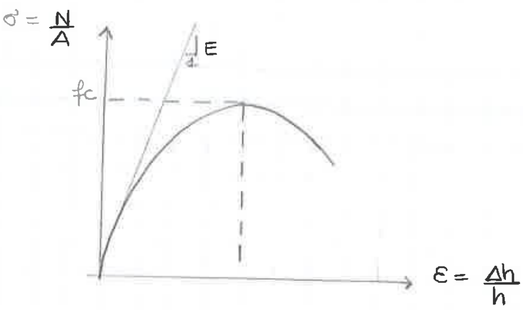
   MC 2010
   - light weight → 8 ÷ 20 kN/m³ (sismic zone)
   - normal weight → 20 ÷ 26 kN/m³
   - heavy weight → ≥ 26 kN/m³ (nuclear plats?)
     ↳ + BARITE
3. Behavior in compression
   ↳ compression tests

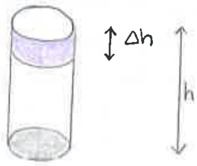
EN



ASTM







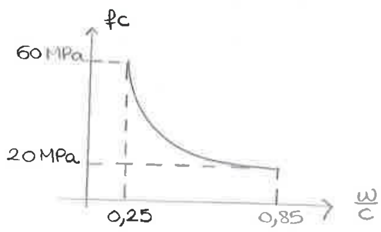
E: elastic modulus
fc: strength (max strength in compression)

EC2 → $E_c = 22'000 \left(\frac{f_c}{10}\right)^{0,3}$ [MPa]

ACI → $E_c = 57'000 \sqrt{f_c}$

$\nu = 0,15 \div 0,2$ (**poisson coefficient**)

fc depends on:
1) water / cement RATIO w/c
2) Type of cement
   - normal : type I
   - modified : type II



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The effects of horizontal actions

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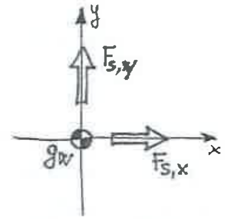
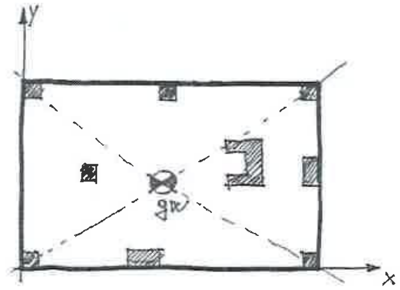
The center of the mass

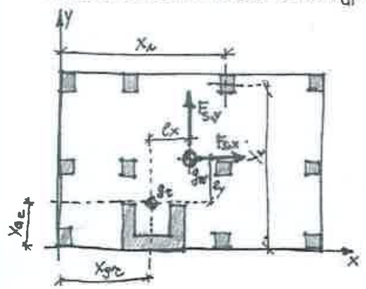
- In each storey, the horizontal forces are applied in the center of the mass g_W.
- In the case of reinforced concrete structures, mass is mainly due to the dead load on the storey (the contribution of columns can be neglected).
- If the loads are uniformly distributed, g_W is the centroid of the storey





The center of stiffness

- Due to the rigid movement of the storey, columns show a reaction through the stiffness k.
- The inertia to the movement of the storey is therefore produced by the columns, and the the global reaction to the seismic loads is applied on the center of stiffness g_r, whose coordinates are x_{gr} and y_{gr}



$$x_{gr} = \frac{\sum_{i=1}^{n} k_{x,i}\, x_i}{\sum_{i=1}^{n} k_{x,i}} = \frac{\sum I_{x,i}\, x_i}{\sum I_{x,i}}$$

$$y_{gr} = \frac{\sum_{i=1}^{n} k_{y,i}\, y_i}{\sum_{i=1}^{n} k_{y,i}} = \frac{\sum I_{y,i}\, y_i}{\sum I_{y,i}}$$

n=number of columns

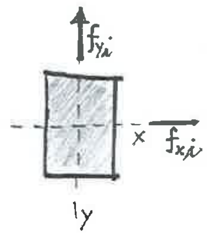
$$I_x = \frac{b\,h^3}{12} \qquad I_y = \frac{h\,b^3}{12}$$

Loads in the columns

- In the i-th columns, two horizontal forces are applied in the x and y directions, respectively:

$$f_{x,i} = \frac{F_{s,x}\, I_{y,i}}{\sum_{j=1}^{n} I_{y,j}} - \frac{\left(F_{s,x}\, e_y + F_{s,y}\, e_x\right) x_i\, I_{y,i}}{\sum_{j=1}^{n}\left(I_{y,j}\, y_j^2 + I_{x,j}\, x_j^2\right)}$$

$$f_{y,i} = \frac{F_{s,y}\, I_{x,i}}{\sum_{j=1}^{n} I_{x,j}} + \frac{\left(F_{s,x}\, e_y + F_{s,y}\, e_x\right) y_i\, I_{x,i}}{\sum_{j=1}^{n}\left(I_{y,j}\, y_j^2 + I_{x,j}\, x_j^2\right)}$$



- where:
  - e_x = distance between the center of mass and the center of stiffness in the x direction
  - e_y = distance between the center of mass and the center of stiffness in the y direction
- The forces $f_{x,i}$ and $f_{y,i}$ are applied on the column in addition to those produced by vertical loads

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Regularity in elevation

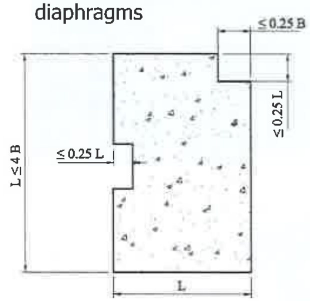
A building can be categorized as regular in elevation (4.2.3.3 EC8) when:

- All lateral load resisting systems, such as columns, shall run without interruption from their foundations to the top of the building.
- Both the lateral stiffness and the mass of the individual stories shall constant or reduce gradually, without abrupt changes, from the base to the top of a building.
- If the setbacks do not preserve symmetry, in each face the sum of the setbacks at all stories shall be not greater than 30 % of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10 % of the previous plan dimension.

POLITECNICO DI TORINO 9

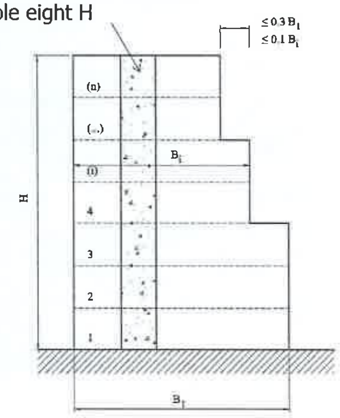
Regularities

Rigid horizontal diaphragms



In plan

Columns shall run on the whole eight H

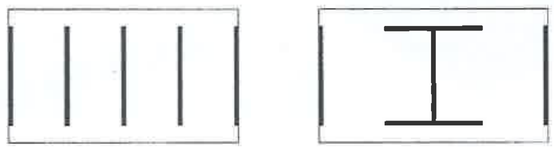


In elevation

POLITECNICO DI TORINO 10

Some basic rules

- All lateral load resisting systems should be in both the main dimensions



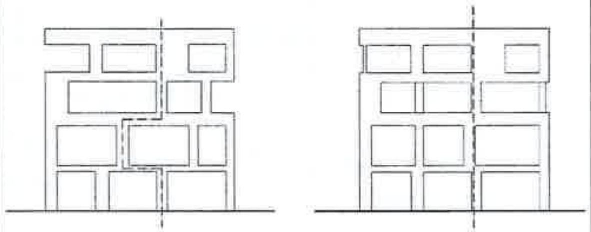
Bad configuration Good configuration

- Vertical elements must be adequately connected to horizontal diaphragms
- lateral load resisting systems must have a foundation
- Structural regularity

POLITECNICO DI TORINO 11

Some basic rules

- The lighter the building and the structure, the better. Horizontal forces are in direct proportion to the mass.)
- A simple structure is desirable: loads should arrive directly to the foundations (e.g., avoiding beams supporting columns)



Bad configuration Good configuration

POLITECNICO DI TORINO 12

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Structural defects

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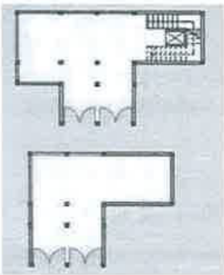
POLITECNICO DI TORINO
Dipartimento di Ingegneria Strutturale e Geotecnica

Outline

- Structural defects
  - Irregularity in plan
  - Irregularity in elevation
  - Cantilever beams
  - One-way frames
  - Slender columns and short columns
  - Irregularity in the presence of non-structural walls
  - Irregularity in the frames

Irregularity in plan

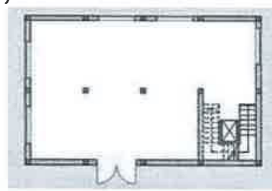
An irregular (asymmetric) shape of the plan results in a long distance between the center of mass and that of stiffness. In presence of horizontal forces, a torsional moment can be generated and some columns can be heavily damaged.



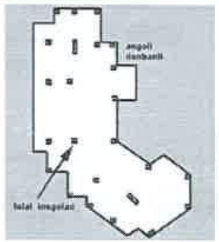
Some asymmetric plans (L,T,C, . .)

Irregularity in plan

Presence of asymmetrical high stiffness vertical elements (e.g., stairs and elevator shaft)



Irregular frames



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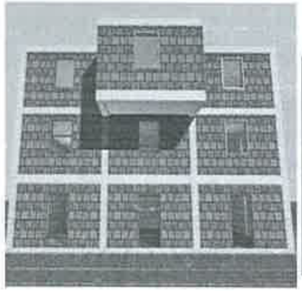
Irregularity in plan and elevation

L'Aquila earthquake

A building having and irregularity in plan and an irregularity in elevation, the latter due to an irregular position of the non-structural walls in the ground floor

Cantilever beams

- The presence of cantilever beams with large spans produces an irregularity in elevation, because a large mass can insist on an insufficient load resisting system.
- Moreover, due to the plasticity of the beam in the fixed end produced by the earthquake, the statically determinate beam can degenerate into a mechanism.



Cantilever beams

One-way frames

Several structures, built before the more recent seismic rules, show the frame only in a single direction.

Accordingly the other direction is a "weak direction", and the seismic loads can produce a collapse mechanism

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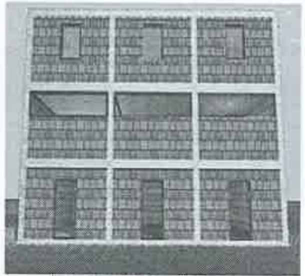
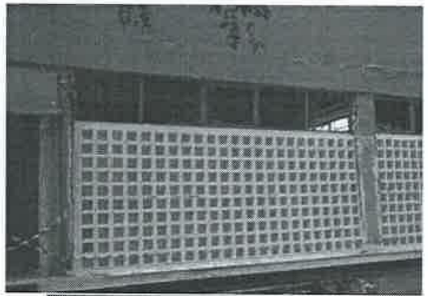
Short columns

Around the stairs

Short columns

In presence of irregular walls.

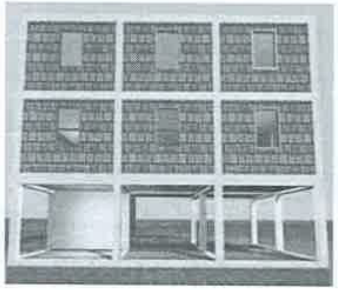




Non-structural walls

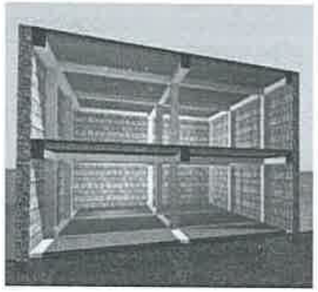
When non structural walls (infilled walls, exterior walls) are irregularly positioned in plan, they can contribute to separate the center of mass from that of the stiffness

This is the case of the so-called "pilotis" storey, which can determine a soft storey in the ground floor.



Irregular external walls

In some cases the external non-structural walls are outside the frame. Thus, they are not connected to the structure and the seismic actions can separate them from the building.



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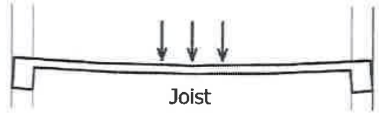
Irregular frames

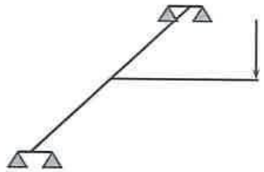
Too many beams in a single node

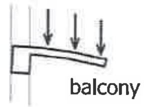
The width of the beam is remarkably larger than that of the column

In presence of torsion



Torsion in the beams that support the joist of a floor slab
(the static equilibrium does not depend on the torsional resistance of the beams)





Torsion in the beam that supports a balcony
(the static equilibrium depends on the torsional resistance of the supporting beam) – do not use in seismic zones

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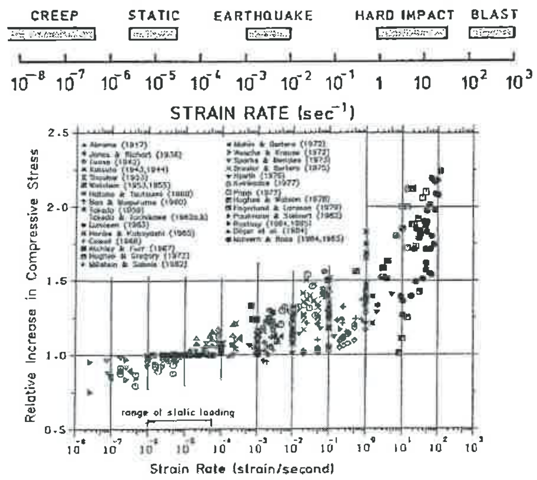
Type of cement/strength

ASTM C 150 Portland Cement Mortar Compressive Strength Specifications in MPa

Curing Time	Portland Cement Type							
	I	IA	II	IIA	III	IIIA	IV	V
1 day	-	-	-	-	12.4	10.0	-	-
3 days	12.4	10.0	10.3	8.3	24.1	19.3	-	8.2
7 days	19.3	15.5	17.2	13.8	-	-	6.9	15.2
28 days	-	-	-	-	-	-	17.2	20.7

Strain rate



Types of concrete

Type of concrete		Strength
NS	Normal strength	20-50 MPa
HS	High strength	50-100 MPa
VHS	Very high strength	100-150 MPa
UHS	Ultra high strength	>150 MPa
UHP	Ultra high performance	

Indirect tensile tests

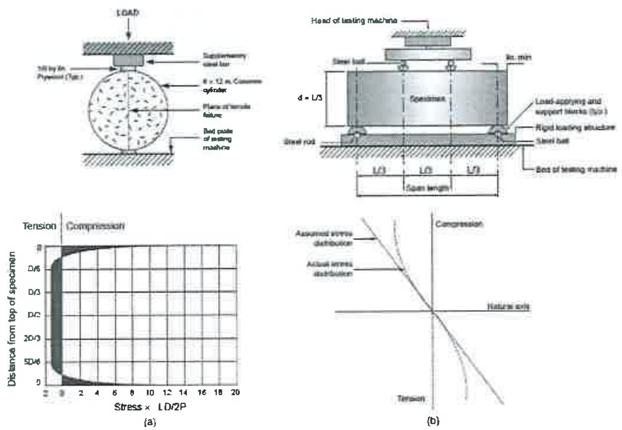


Figure 3-16 (a) Splitting tension test (ASTM C 496): top, diagrammatic arrangement of the test; bottom, stress distribution across the loaded diameter of a cylinder compressed between two plates. (b) Flexural test by third-point loading (ASTC C 78): top, diagrammatic arrangement of the test, bottom, stress distribution across the depth of a concrete beam under flexure.

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Steel reinforcing bars

U.S. rebar size chart

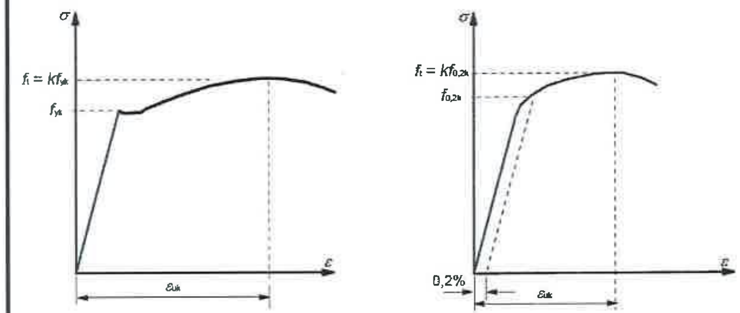
Imperial bar size	Metric size	Linear Mass Density lb/ft	Linear Mass Density (kg/m)	Nominal diameter (in)	Nominal diameter (mm)	Nominal area (in²)	Nominal area (mm²)
#2	No.6	0.167	0.249	0.250 = ¼	6.35	0.05	32
#3	No.10	0.376	0.561	0.375 = ⅜	9.525	0.11	71
#4	No.13	0.668	0.996	0.500 = ½	12.7	0.20	129
#5	No.16	1.043	1.556	0.625 = ⅝	15.875	0.31	200
#6	No.19	1.502	2.24	0.750 = ¾	19.05	0.44	284
#7	No.22	2.044	3.049	0.875 = ⅞	22.225	0.60	387
#8	No.25	2.670	3.982	1.000	25.4	0.79	509
#9	No.29	3.400	5.071	1.128	28.65	1.00	645
#10	No.32	4.303	6.418	1.270	32.26	1.27	819
#11	No.36	5.313	7.924	1.410	35.81	1.56	1006
#14	No.43	7.650	11.41	1.693	43	2.25	1452
#18	No.57	13.60	20.284	2.257	57.3	4.00	2581
#18J		14.60	21.775	2.337	59.4	4.29	2678

European rebar size chart

Metric bar size	Linear mass density (kg/m)	Nominal diameter (mm)	Cross-sectional area (mm²)
6,0	0.222	6	28.3
8,0	0.395	8	50.3
10,0	0.617	10	78.5
12,0	0.888	12	113
14,0	1.21	14	154
16,0	1.58	16	201
20,0	2.47	20	314
25,0	3.85	25	491
28,0	4.83	28	616
32,0	6.31	32	804
40,0	9.86	40	1257
50,0	15.4	50	1963

Length 9-12 m

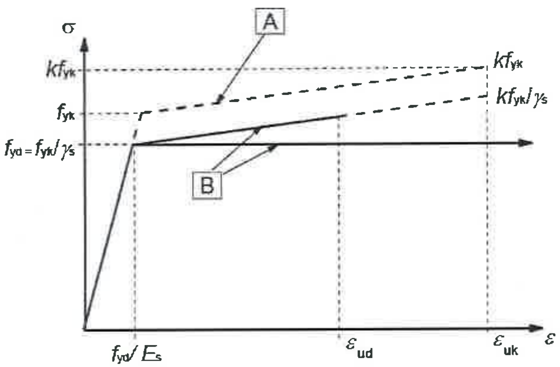
Behavior in tension



a) Hot rolled steel

b) Cold worked steel

EC2 stress-strain relationships



$k = (f_t/f_y)_k$

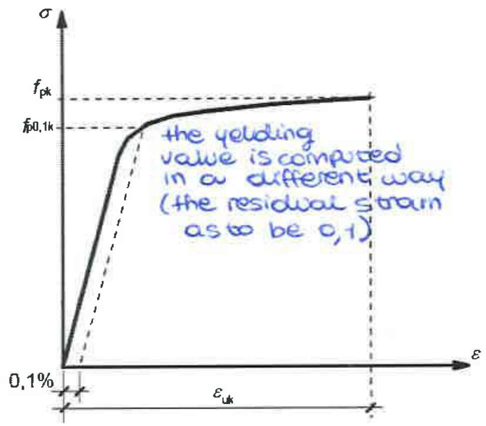
A Idealised

B Design

Pre-stressing steel

Result of direct stress to this kind of steel

made of wires from 2 to 6



the yielding value is computed in a different way (the residual strain as to be 0,1)

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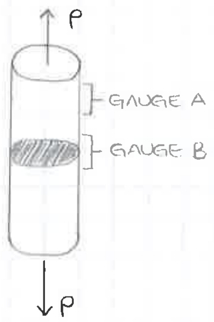
In this two cases we consider only the first part of the relation, we have to include the post peak behaviour. We don't do because the behaviour can't be represented by a stress-strain relationship, both in compression and tension

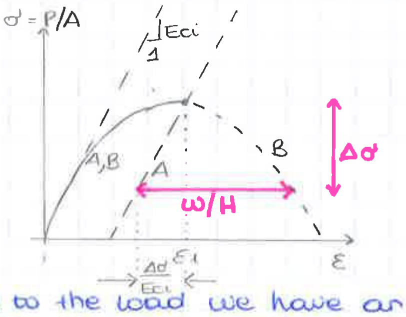
Tensile test

we do the same test performed by **HILLER BORG** (1976)



- cilindre
- two gauge which measure the stress
  A
  B : located exactly in the crack
- cross section A

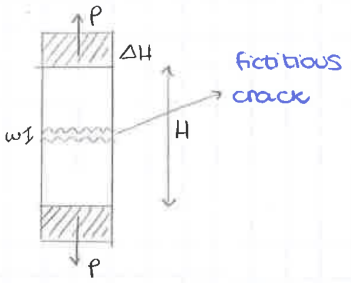
In the pre-peak the stress in A and B are the same, after the peak they are different



A unloading // to the loading
B increase of the strain (continouing the parabola)

The increment of the strain is only on the crack zone, outside we observe an elastic unload

Due to the load we have an increasing of the specimen



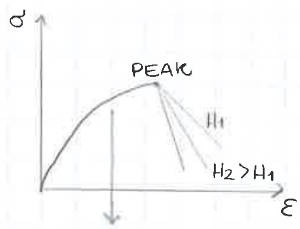
If we have a crack we have an increment so H grows (+ΔH)
w : crack width (of the fictitious crack)
in the other points unloading is linear
↳ so we have an unlinear part due to the difference between A and B and a decrement of σ (Δσ) (w)

ε in the post peak $= \varepsilon_1 + \frac{w}{H} - \frac{\Delta\sigma}{E_{ci}}$

If we consider $\varepsilon - \varepsilon_1$ we have the post-peack strain

$\varepsilon - \varepsilon_1 = \frac{w}{H} - \frac{\Delta\sigma}{E_{ci}}$

it's function of the hight of the sample which isn't a material property so we can't use a universal curve it depends on H



all the parametres are related to the material

So the post-peak can be defined in term of stress-strain and width

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w : relative sliding of the concrete sample divided by the fictitious crack

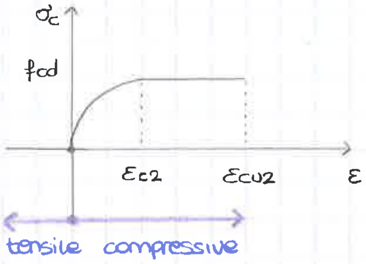
$\varepsilon = \varepsilon_1 + \frac{w}{H} - \frac{\Delta\sigma}{E_{ci}}$

$f_c = 10\, f_{ct}$

To study the behaviour we have to:
1) simplify
2) f_c, f_{cm}, etc are average value so we need SAFETY APPROACH (it has to be included in the relationships)
3) reduce tests

⟹ have a relationship which is as easy as possible

When we compute the Rd in structural elements we use a $\sigma-\varepsilon$ relationship



we simplify the problem and reduce test with a conventional relationship which neglects the tensile part, it's called **PARABOLA-RECTANGLE**

the contribution in tensile is provided by steel

$$\begin{cases} \sigma_c = f_{cd}\left[1-\left(1-\frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] & 0 \le \varepsilon_c \le \varepsilon_{c2} \\ \sigma_c = f_{cd} & \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2} \end{cases}$$

Three main parametres which are function of the class of concrete (compressive strength)

$$\begin{cases} n \\ \varepsilon_{c2} \\ \varepsilon_{cu2} \end{cases}$$

If the class of concrete is lower than C50/60

$$\begin{cases} n = 2 \\ \varepsilon_{c2} = 0{,}2\,\% \\ \varepsilon_{cu} = 0{,}35\,\% \end{cases}$$

The safety aprach is included in the definition of fcd (= design compressive strength)

$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c}$

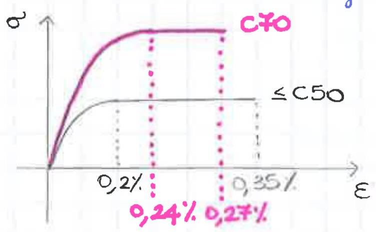
γ_c : reduction factor for material (= 1,5)

f_{ck} : characteristic value (5% percentile)

$\alpha_{cc} = 0{,}85$ (effect of long term actions)

In EC2, MC 2010 from the class of concrete we have all the necessary parametres

For classes ≥ C50 the graphic is shifted



the strength increases but the constant part is reduced

So ≥C50 is stronger, but more brittle

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$f_{td} = \frac{f_{tk}}{\gamma_s}$

$\gamma_s = 1,15$ (lower than concrete)

$f_{yd} = \frac{f_{yk}}{\gamma_s}$

$\varepsilon_{ud} \cong 0,9\ \varepsilon_{uk}$

$\varepsilon_{yd} = \frac{f_{yd}}{E_s}$

In EC2 there is only a possibility but in reality we can have the second branch as complete plastic with no limitation of strain (very large strain) (u)

In Europe the steel B450 C, used for rebar, has:

- $f_{yk} = 450$ MPa
- $k = \frac{f_{tk}}{f_{yk}} = 1,15 \div 1,35$
- $\varepsilon_{uk} = 7,5\ \%$
- $f_{yd} = \frac{450}{1,15} \cong 390$ MPa
- $f_{td} = 1,25 \cdot \frac{f_{yk}}{1,15} = 489$ MPa
- $\varepsilon_{ud} = 0,9\ \varepsilon_{uk} = 6,\ 5\ \%$
- mass $= 7850\ kg/m^3$

In compression the steel has exactly the same behaviour

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Any kind of shrinkage occurs without any external actions.

In EC2 we have 2 different kind of shrinkage

The total one is given by the sum of drying and autogenous shrinkage
$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$

A) $\varepsilon_{cd} = \beta_{ds}(t, t_s)\, k_h \cdot \varepsilon_{cd0}$
ε_{cd0} is just after casting, shrinkage at time $t=0$, found in Table 3.2

PAG 20 A

$\beta_{ds}(t, t_s)$
↳ coefficient which depends on two times :
- t: time at which we want to calculate ε_{cd}
- t_s: time of set or cast (at which shrinkage starts)

if $t \to \infty$ $\beta_{ds} = 1$, so if we consider the effect of shrinkage after a long time from setting, $\beta_{ds} = 1$

$k_h = f(h_0)$

$$h_0 = \frac{\text{two times the area of the section}}{\text{perimeter exposed to environment}} = \frac{2A_c}{u}$$

↳ exposed to dry

PAG 20 B

$\varepsilon_{cd\infty} = k_h \cdot \varepsilon_{cd0}$

The maxium effect of the shrinkage because β is the maximum value

B) $\varepsilon_{ca} = \beta_{as}(t) \cdot \varepsilon_{ca}(\infty)$

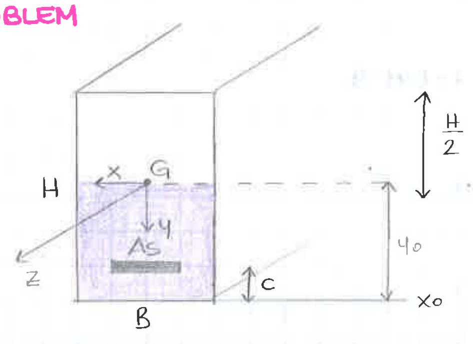
$\varepsilon_{ca}(\infty) = 2{,}5\,(f_{ck} - 10) \cdot 10^{-6}$ ⟶ maximum autogenous shrinkage
f_{ck} in MPa

$\beta_{as}(t) = 1 - e^{-0{,}2t^{0{,}5}}$
↳ is a coefficient in which t is in days
↳ the maximum value is for $t \to \infty$ so $\beta_{as} = 1$ as before

⟹ maximum total shrinkage
$\boxed{\varepsilon_{cs\infty} = k_h\, \varepsilon_{cd,0} + 25\,(f_{ck} - 10) \cdot 10^{-6}}$

This isn't an exact formula, because it can be 40÷50% higher or lower than this value because it depends on many factors

We can see what happens in a reinforced concrete section when we have shrinkage
PROBLEM



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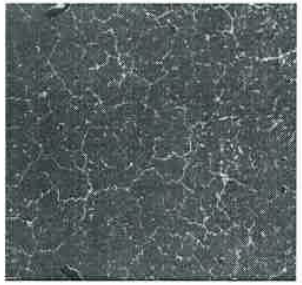
Cracking due to shrinkage

=> problems for durability (corrosion of steel leads to an increase of volume)

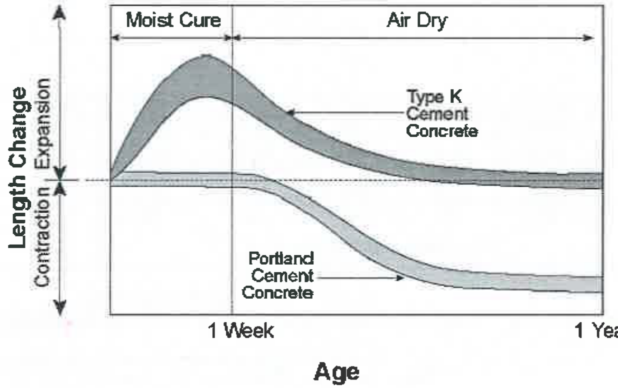


in concrete without reinforcement
↳ maybe casting in layers

in concrete with reinforcement
↳ squared cracking
↳ shrinkage of concrete
↳ deterioration of structure

Expansive agents

reacts with water and so we have an increment of volume
↳ which compensate the shrinkage



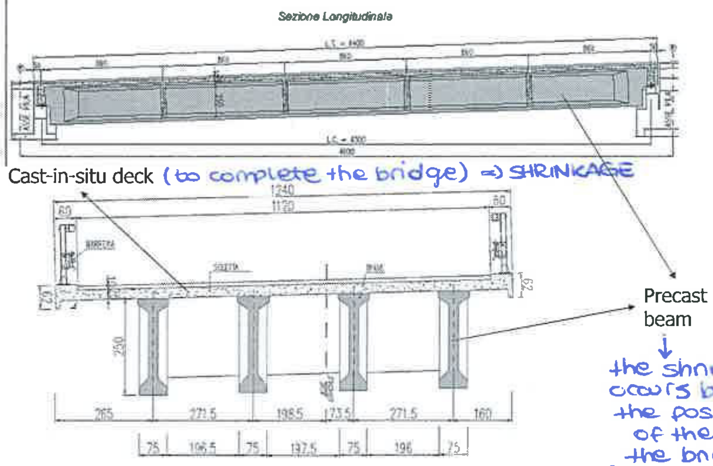
=> Difficult to control f(temperature, material, ecc.) because even the shrinkage is variable

Taking to account, not completely eliminating (it's impossible)

↱ we should consider shrinkage because it leads to a high tension state without any loads

Partially prefabricated bridge

↳ pre-cast concrete beam



Cast-in-situ deck (to complete the bridge) => SHRINKAGE

Precast beam
↓
the shrinkage occurs before the positioning of them in the bridge (not to be considered!)

In general it occurs when the structure is partially pre-fabricated, so in the cast-in-situ parts shrinkage happens.

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$$I_{x0} = n1\left[\frac{BH^3}{12} + BH\left(\frac{H}{2} - y_0\right)^2\right] - n1\,A_s(y_0 - c)^2 + n2\,A_s(y_0 - c)^2 =$$

$$= 1\left[\frac{200 \cdot 400^3}{12} + 200 \cdot 400\left(\frac{400}{2} - 194{,}2\right)^2\right] - 1 \cdot 500(194{,}2 - 40)^2 +$$

$$+ 7 \cdot 500\,(194{,}2 - 40)^2 = 1{,}14 \cdot 10^9 \text{ mm}^4$$

- we can compute the state of stress of the section, using the parametres of the general plane section.

$$\begin{Bmatrix} \lambda \\ \mu_x \\ \mu_y \end{Bmatrix} = \begin{Bmatrix} \bar{\lambda} \\ \bar{\mu}_x \\ 0 \end{Bmatrix}$$

(=0) because we have only a variation along z axis

In general, strain is elastic (due to load so 0) and imposed strain

$$\bar{\lambda} = \frac{1}{A_0}\int_{A_c} n1\,\bar{\varepsilon}_z\,dA = \frac{\bar{\varepsilon}_z}{A_0} \cdot A_c = \frac{-6 \cdot 10^{-4} \cdot (200 \cdot 400 - 500)}{83'000} = -5{,}8 \cdot 10^{-4}$$

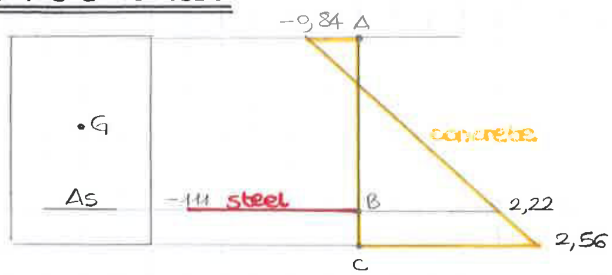
shrinkage is constant, not a function of the position

$$\bar{\mu}_x = \frac{1}{I_{x0}}\int_{A_c} n1\,\bar{\varepsilon}_z \cdot y \cdot dA = \frac{\bar{\varepsilon}_z}{I_{x0}}\underbrace{\int_{A_c} y\,dA}_{\text{static moment of concrete section } S_x}$$

$$S_x = \left[By_0 \cdot \frac{y_0}{2} - A_s(y_0 - c)\right]n1 - n1\left[B(H - y_0)\left(\frac{H - y_0}{2}\right)\right] = -5{,}4 \cdot 10^5 \text{ mm}^3$$

$$\bar{\mu}_x = \frac{-6 \cdot 10^{-4} \cdot (-5{,}4 \cdot 10^5)}{1{,}14 \cdot 10^9} = 2{,}84 \cdot 10^{-7} \text{ 1/mm}$$

STATE OF STRESS



$\sigma_z = E(\bar{\lambda} + \bar{\mu}_x \cdot y - \bar{\varepsilon}_z) \sim$ elastic = total − imposed

↳ linear function with respect to the y axis

A) $\sigma_{zA} = E_c(\bar{\lambda} + \mu_x \cdot (y_0 - H) - \bar{\varepsilon}_z) = [-5{,}8 \cdot 10^{-4} + 2{,}84 \cdot 10^{-7}(194{,}2 - 400) + 6 \cdot 10^{-4}]$
$= -0{,}84$ MPa

B) $\sigma_{zB,\,concrete} = E_c[\bar{\lambda} + \mu_x(y_0 - c) - \bar{\varepsilon}_z] = 2{,}22$ MPa

B) $\sigma_{zB,\,steel} = E_s\,[\bar{\lambda} + \mu_x(y_0 - c)] = -111$ MPa

↳ no imposed strain

c) $\sigma_{zC} = E_c[\bar{\lambda} + \mu_x y_0 - \bar{\varepsilon}_z] = 2{,}56$ MPa

Tipical situation: when we have shrinkage, reinforcing steel is under compression, but concrete around steel is in tension

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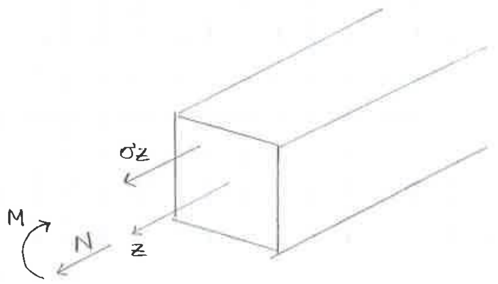
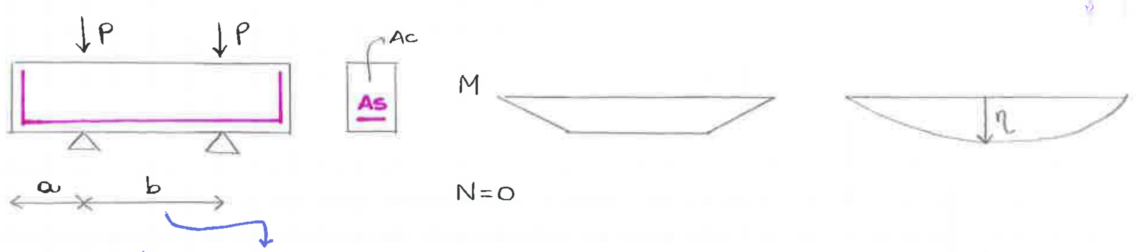
CONCRETE SECTION

8/11/17

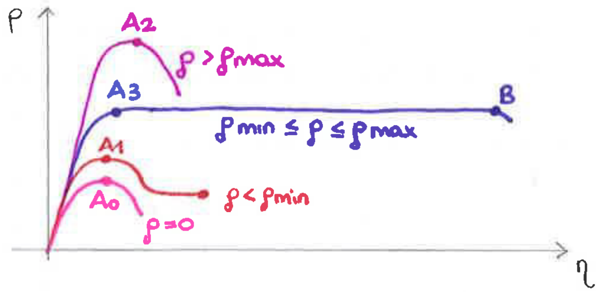


Only σ_z on the cross section M and/or N (only M: pure flexion ; M+N: complexion flexion). We neglected shear stresses.
~ column
~ beam



constant bending moment, so in terms of stresses we have only normal stress and the reinforcement are in the tensile zone (ω)

The behavior can be represented in the $P\eta$ plane (applied load, span deflection)



we can compute the relationship between Ac and As **REINFORCEMENT RATIO**

$\rho = \frac{A_s}{A_c}$

↳ it leads to a lot of possibilities for the $P\eta$ diagram

1) $A_s = 0$
$\rho = 0$
↳ no reinforcement in the tensile zone
↳ brittle behaviour

2) Just a little reinforcement
$A_s \neq 0$
$\rho < \rho_{min}$
UNDER-REINFORCED CONCRETE BEAM
↳ increment of strength reaching the failure
↳ A_1: cracking B: yelding of steel which is lower than load so load at cracking is bigger than load at yielding ($A_1 > B$)

3) Large quantity of reinforcement
$A_s \neq 0$
$\rho > \rho_{max}$
OVER-REINFORCED CONCRETE BEAM
↳ the failure can't be on the tensile zone, the failure is reached in the maximum value for compression
↳ very brittle behaviour, we have crushing of concrete in compression at [illegible]

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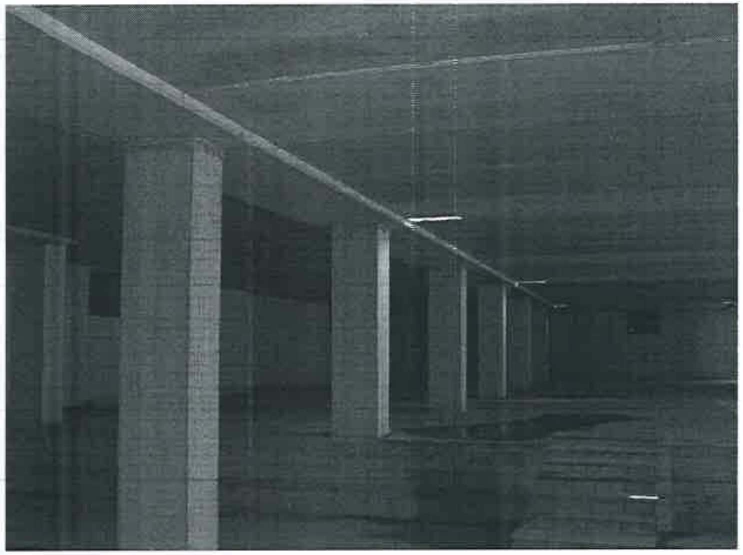


Applications

Prédalles slabs can be used to build :

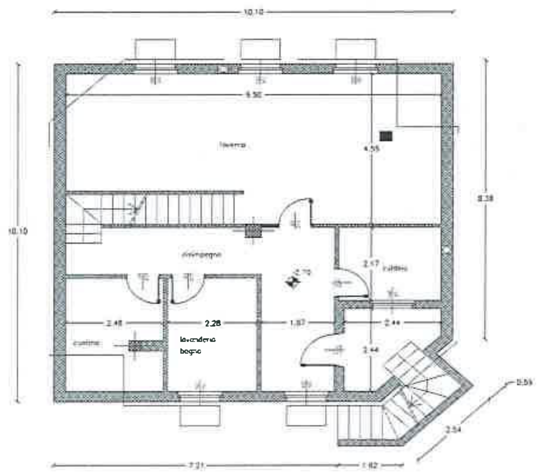
- Floors of civil housing
- Floors of garage
- Bridge decks
- Fire resistant floors (e.g., REI 120') → Fire resistance is one of the positive aspects, especially in garages, because it satisfied parameters given by the code rules.
- ....

This slab has the sufficient thickness to respect fire resistance

They are particularly indicated in all the cases in which the surface of the intrados does not include the presence of a plaster.

All designs start from draws of the building

Input data: the plan of the floor



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concrete protects steel but external agents attac concrete too (+ fire resistance). Corrosion increases actions and volume of steel

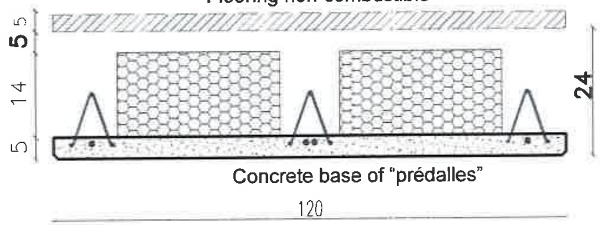
Definition of concrete cover

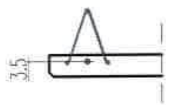
- Concrete cover is defined by the durability requirements (Table 4.1 of EC2).
- In the case of floors for civil housing

XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain

- In some cases, concrete cover depends on the fire resistance of the slab (EC2 1-2). For the standard fire exposure, members shall comply with criteria R, E and I as follows:
  - integrity (criterion E)
  - insulation (criterion I)
  - mechanical resistance (criterion R)

Cross-section of the slab





Concrete cover

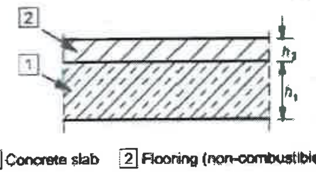
it resists 90 minutes to fire

Definition of concrete cover

If a REI 90' concrete floor is selected, from EC2 1-2 and Italian concrete rules, we have

part 1-
(in general we use part 1-1)

we use tables to design rebars.



1 Concrete slab 2 Flooring (non-combustible)

criterion R

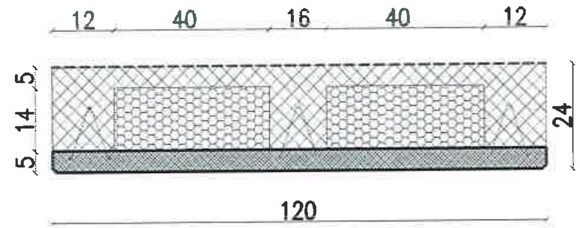
H = 24 cm ; concrete cover = 3,5 cm

criterion EI

h_2 = 10 cm (part of this height – 5 cm – should be made of concrete)

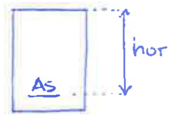
Geometry of the floor

Hypothesis: H = 600 / 25 = 24 cm



Effective depth of a cross-section:

$h_{ut} = 24 - 3{,}5 = 20{,}5$ cm



4

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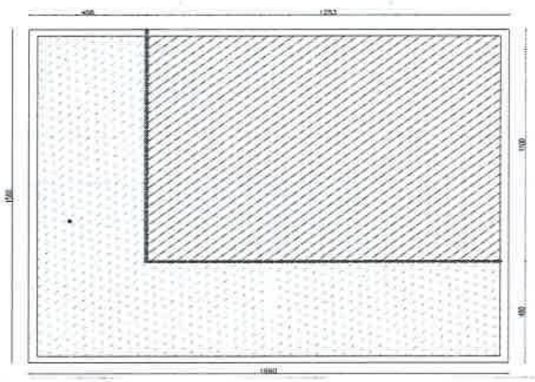
Load analysis

Indoor
G_1 = 370 daN/m^2
G_2 = 350 daN/m^2
Q_1 = 200 daN/m^2

Outdoor
G_1 = 370 daN/m^2
G_2 = 250 daN/m^2
Q_1 = 400 daN/m^2



Combination of the actions

INDOOR

- U.L.S.

$Ed = \gamma_{G1}*G_1+\gamma_{G2}*G_2+\gamma_{Q1}*Q_1$= 1,3*370+1,5*(350+200)= 1306 daN/m^2

- S.L.S. – Rare

$Ed = G_1+G_2+Q_1$= 370+350+200= 920 daN/m^2

- S.L.E. – Frequent

$Ed = G_1+G_2+ \psi_{11}*Q_1$= 370+350+0,5*200= 820 daN/m^2

- S.L.E. – Quasi-Permanent

$Ed = G_1+G_2+ \psi_{21}*Q_1$= 370+350+0,3*200= 780 daN/m^2

Combination of the actions

OUTDOOR

- U.L.S. (used for bending moment, shear, failure and collapse)

$Ed = \gamma_{G1}*G_1+\gamma_{G2}*G_2+\gamma_{Q1}*Q_1$= 1,3*370+1,5*(250+400)= 1456 daN/m^2

↳ safety coefficient

- S.L.S. – Rare

$Ed = G_1+G_2+Q_1$= 370+250+400= 1020 daN/m^2

- S.L.S. – Frequent

$Ed = G_1+G_2+\psi_{11}*Q_1$= 370+250+0,5*400= 820 daN/m^2

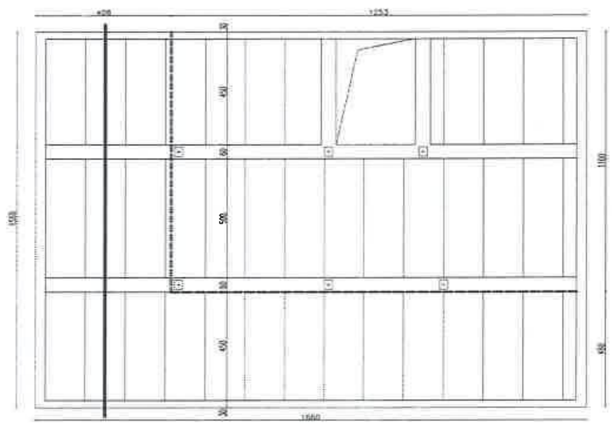
↳ combination coeff.

- S.L.S. – Quasi-Permanent

$Ed = G_1+G_2+ \psi_{21}*Q_1$= 370+250+0,3*400= 740 daN/m^2

Analysis of serviceability some times is respected only respecting geometric rules.

Outdoor floor



4 supports

6

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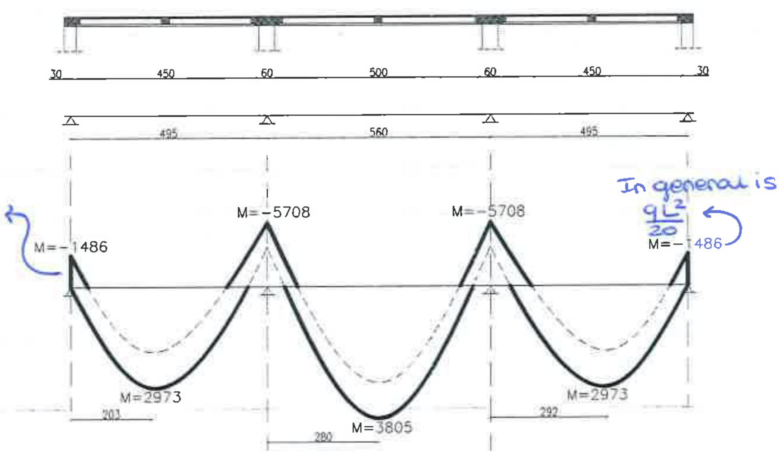
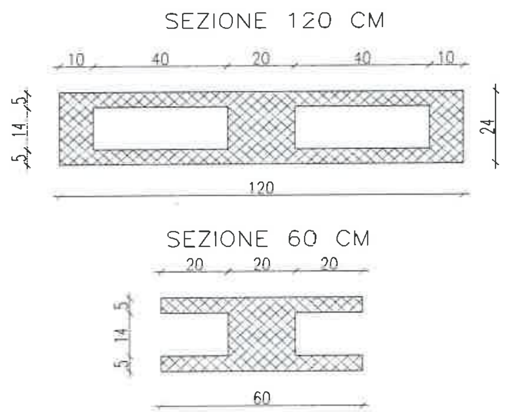
In general if fixed points we don't have any moment, but for safety factor, we don't put it = 0.
In fact there is a beam beneath the slab which has thickness so it's not a perfect support

The bending moments on the ends



Design of reinforcement (prédalles)



Design of reinforcement (prédalles)

Prédalles A

M_{sd} = 1,2*2973 = 3568 daN*m

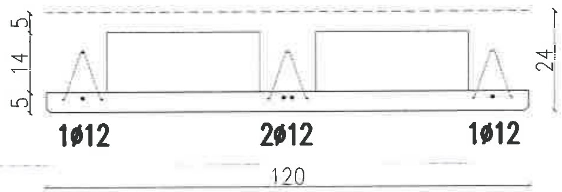
μ_d = 3568 10^4 / (1200*205^2*14,1) = 0,050 (Montoya)

ρ = 0,18%

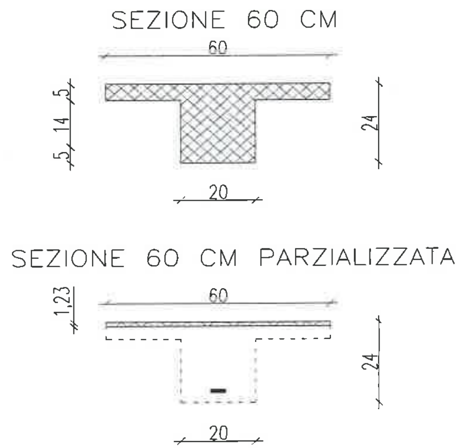
$A_{s,inf}$ = 0,18*1200*205 / 100 = 443 mm^2 = 4,43 cm^2 (from design theory)

$A_{s,inf}$ = 1ϕ12+ 2ϕ12 +1ϕ12 = 4,52 cm^2 (real)

ξ_u = 0,06 x_u = 0,06*205 = 12,3 mm = 1,23 cm



Design of reinforcement (prédalles)



8

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Design calculations at S.L.S.

Deflection and cracking

Moment of inertia in the 1° stage (uncracked cross-section)
$J' = 119946\ cm^4$
Moment of inertia in the 2° stage (cracked cross-section)
$x = 4{,}90\ cm$; $J'' = 27192\ cm^4$; $\sigma_c = 48\ daN/cm^2$; $\sigma_s = 2293\ daN/cm^2$
Bending moment at first cracking $M_{cr} = f_{ctm} * J' / (H-x)$
$M_{cr} = 2559\ daN*m$ con $f_{ctm} = 25{,}6\ daN/cm^2$
$M_{sd} = 1934\ daN*m < M_{cr}$ (SLS-QP)
$M_{sd} = 2665\ daN*m > Mcr$ (SLS-R) ; $L_{cr} = 560-(2559/2665)*560 = 22\ cm$

$f' = (5/384)*(q*L^4/E_{cm}*J') = 0{,}35\ cm$
$f'' = f'*(J'/J'') = 1{,}54\ cm$
$\xi = 1-c*\beta^2 = 1-0{,}5(M_{cr}/M_{sd}) = 0{,}54$ (C4.1.12)

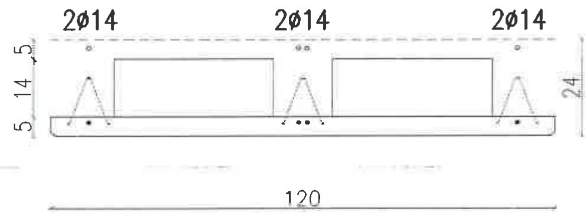
$f = 0{,}54*1{,}54+(1-0{,}54)*0{,}35 = 0{,}99\ cm < L/500 = 560/500 = 1{,}12\ cm$

Reinforcement has to cover negative and positive moment and + has to be put outside the prédalles

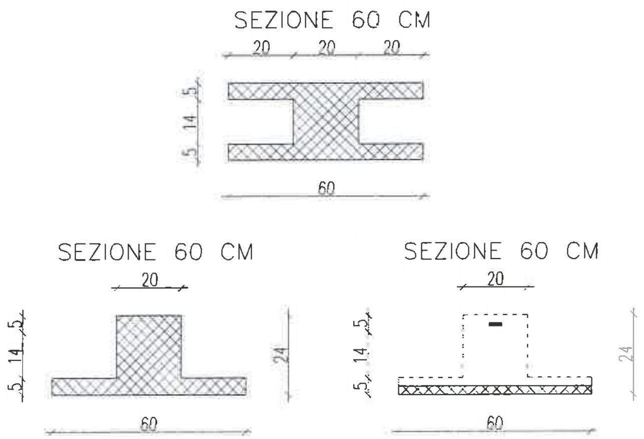
Design of reinforcement (out of prédalles)

before the final casting

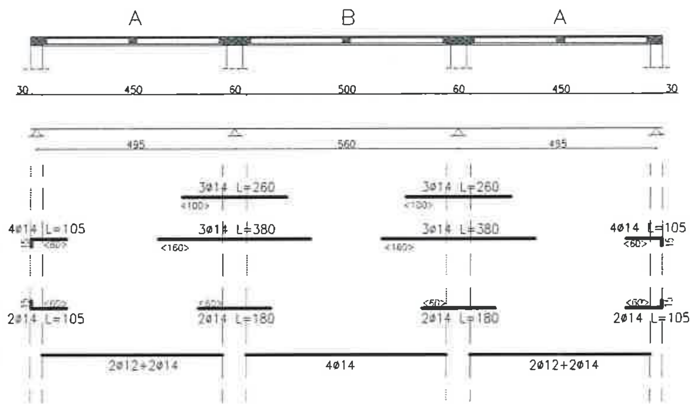
Prédalles A and B (upper concrete cover= 4 cm)
$M_{sd} = -1{,}2*5428 = -6514\ daN*m$
$\mu_d = 6514\ 10^4 / (1200*200^2*14{,}1) = 0{,}096$
$\rho = 0{,}35\%$
$As_{,inf} = 0{,}35*1200*200 / 100 = 840\ mm^2 = 8{,}40\ cm^2$
$As_{,inf} = 6\phi14 = 9{,}24\ cm^2$
$\xi_u = 0{,}126$ $x_u = 0{,}126*200 = 25{,}2\ mm = 2{,}52\ cm$



Design of reinforcement (out of prédalles)



Design of all the reinforcement



4) upper part to balance the negative moment
3)
2)
1) lower part to complete the positive moment
in precast elements

10

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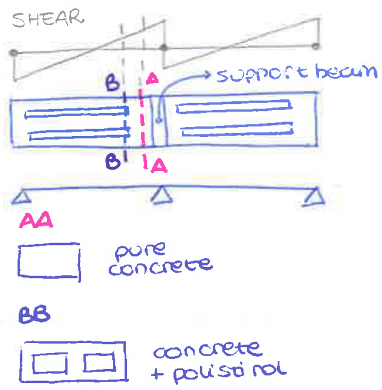
Shear actions



From the envelope

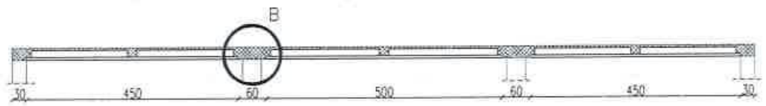
U.L.S. :
$T_1 = Q^*x_1$ = 1456*2,03 = 2952 daN
$T_2 = Q^*x_2$ = 1456*2,92 = - 4252 daN
$T_3 = Q^*x_3$ = 1456*2,80 = 4077 daN
$T_4 = Q^*x_4$ = 1456*2,80 = - 4077 daN
$T_5 = Q^*x_5$ = 1456*2,80 = 4252 daN
$T_6 = Q^*x_6$ = 1456*2,80 = - 2952 daN

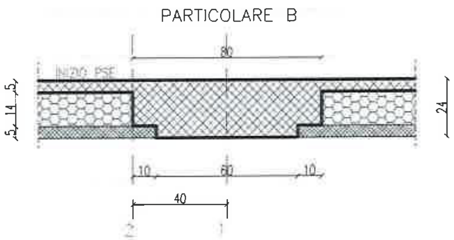
S.L.S. Rare :
S.L.S. Freq :
S.L.S. Q.P. :

Between AA and BB shear doesn't have a big variation, but sections are completely different because when shear isn't verified we don't put reinforcement, but we change only shape of polistirol bars.

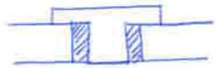
Then we interrupt the reduction of polistirol when the section is shear verified.

Details of the supports

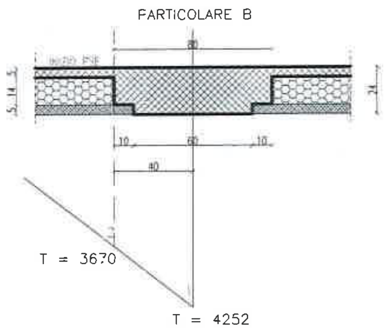




Shear strength

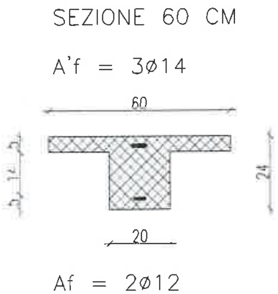


U.L.S. :
$T_2 = Q^*x_2$ = 4252 daN
$T_{2\text{-}1} : x_{2,1} = T_{2\text{-}2} : (X_{2,1} - x_{2,2})$
$x_{2,1}$ = 292 cm
$x_{2,2}$ = 40 cm
4252 : 292 = $T_{2\text{-}2}$: (292-40)
$T_{2,2}$ = 3670 daN



Shear design

U.L.S. :
V_{Ed} = 0,60*3670 = 2202 daN
Non-dimensional shear strength
$\eta\ = V_{Ed} / (b_w\, d\, f'_{cd})$
dove per le N.T.C. $f'_{cd} = \nu_1 f_{cd}$ = 7,05 N/mm²
η = 22020 / (200*205*7,05) = 0,076
Se $\eta < 0{,}424 * k\,(100\rho_l / f^2_{ck})^{1/3}$ (Verified)
$k = 1+(200/d)^{1/2} = 1+(200/205)^{1/2} = 1{,}99 < 2$
$100\rho_l$ =100*226/(200*205) = 0,55%
$100\rho_l = 0{,}025\,(k^3 f_{ck})^{1/2} = 0{,}025\,(1{,}99^3 \cdot 24{,}9)^{1/2}$
$100\rho_l$ = 0,35%
$0{,}076 < 0{,}424 \cdot 1{,}99\,(0{,}55/24{,}9^2)^{1/3} = 0{,}081$ (Verified)



12

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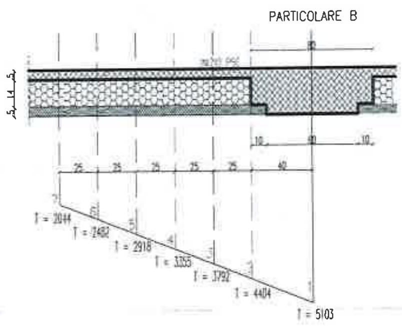
Shear design

Reduction of the polystyrene block (25 cm)
$T_{3\text{-}3}$ = 3792 daN   x=40+10+25=75
η = 37920 / (200*205*7,05) = 0,131
NOT Verified

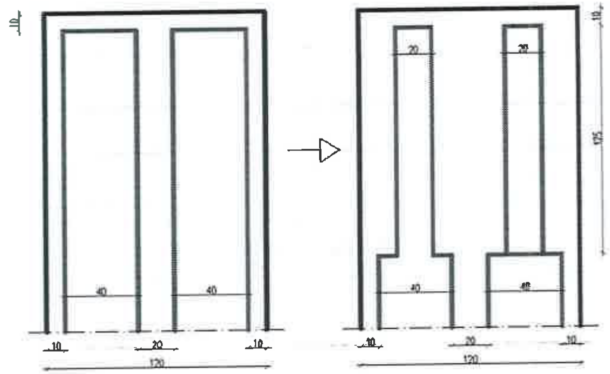
$T_{4\text{-}4}$ = 3355 daN   x=40+10+50=100
η = 0,116
NOT Verified

$T_{5\text{-}5}$ = 2918 daN   x=40+10+75=125
η = 0,101
NOT Verified

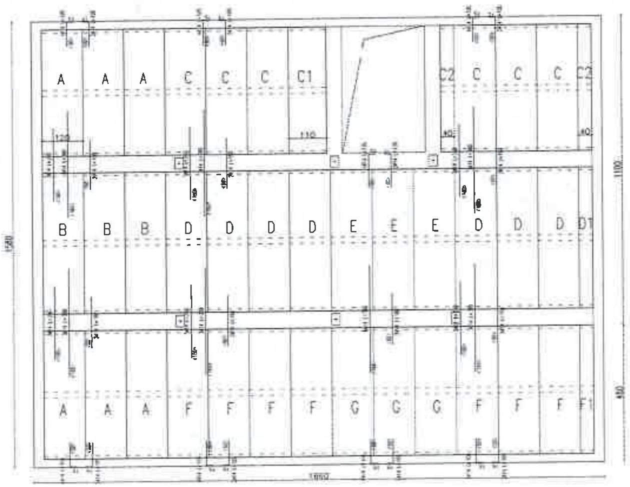
$T_{6\text{-}6}$ = 2045 daN   x=40+10+125=175
η = 0,071 < 0,076
Verified



Shear design



Final project of the floor



Final project of the floor

14

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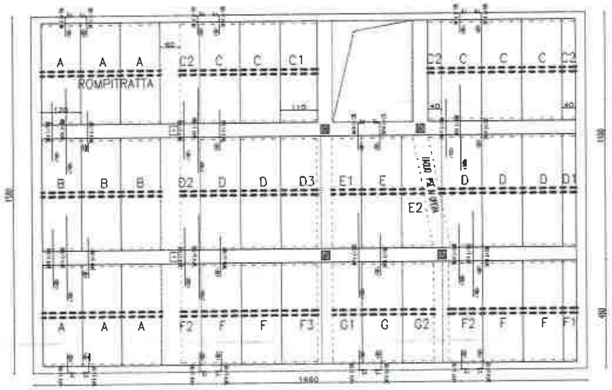
Some details

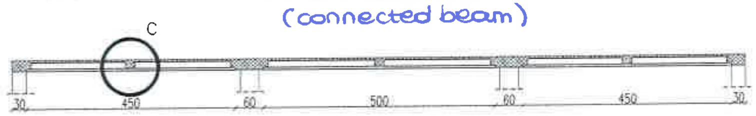
- DISTRIBUTION RIBS



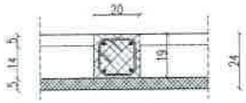
All the slabs has to be connected because we can't allow that one slab is flexed and the other is unloaded

Some details

- DISTRIBUTION RIBS (connected beam)



PARTICOLARE C



it has fixed dimensions, it isn't calculated, it's only used to distribute the loads in a way as uniforme as possible

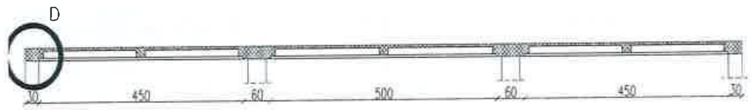
- Stirrups ϕ 8 @ 20 cm + 2+2 ϕ 12 longitudinal rebar

Some details

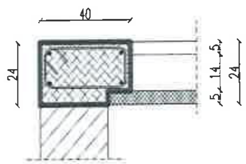
- RIBS ON EXTERNAL WALLS

Some details

- RIBS ON EXTERNAL WALLS



PARTICOLARE D



- Stirrups ϕ 8 @ 20 cm + 2+2 ϕ 12 longitudinal rebar

16

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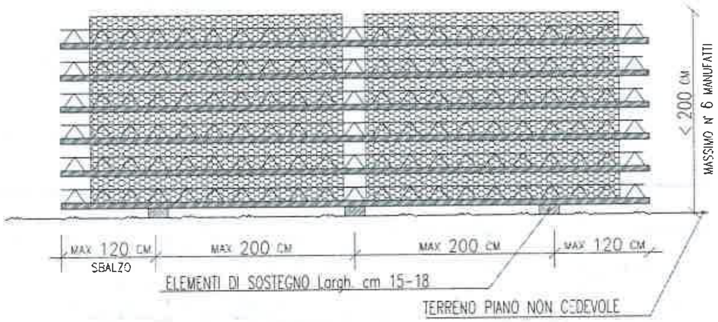
Storing the prédalles on the ground



ISTRUZIONI PER LO STOCCAGGIO

LO STOCCAGGIO IN CANTIERE DOVRA' AVVENIRE SU FONDO PIANO E COMPATTO, E NON IN UNA ZONA DI TRANSITO DI AUTOMEZZI. LE PREDALLE ACCATASTATE (SEMPRE IN ORRIZZONTALE) DOVRANNO ESSERE POSTE SU LISTELLI DI LEGNO AVENTI INTERASSE NON SUPERIORE A 200 CM E CON SBALZO MASSIMO INFERIORE A 100 CM. IL NUMERO DI PREDALLE SOVRAPPOSTE NON DEVE ESSERE MAI SUPERIORE A 6 E/O A 2 METRI DI ALTEZZA. LA BOTTA SRL DECLINA OGNI RESPONSABILITA' PER I DANNI CAUSATI AI MANUFATTI PER LA MANCATA OSSERVANZA DELLE ISTRUZIONI ILLUSTRATE

Construction manual


BOTTA PREFABBRICATI

NOTA BENE: POSARE RETE Ø5/20/20 NELLA CALDANA SUPERIORE SU TUTTO IL SOLAIO

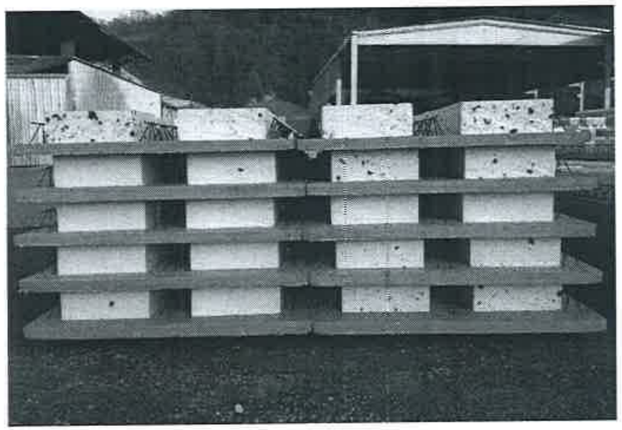
Lastre	Spessore solaio cm	Portata daN/mq			Traliccio							
TIPO A-B-C	5+14+5	370	200	400	5/7/5	12.5	30	B450C	NO	20	120	38
TIPO D-E-F	5+20+5	435	250	750	6/8/6	12.5	30	B450C	NO	20	120	38

OGGETTO: SCHEMA DI MONTAGGIO DELLE LASTRE TERMODALLE E POSIZIONAMENTO TRONCONI

CALCOLO LASTRE	
SOLAIO	PIANO PRIMO
SCALA	1:50
IMPRESA	ROSSO
CANTIERE	CHIVASSO (TO)
LAVORO	PF W2
DATA	10.11.13

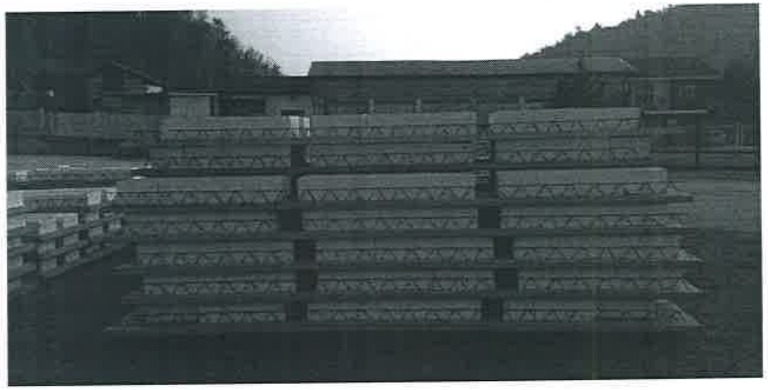
Production of prédalles

- Storing the precast elements



Production of prédalles

- Storing the precast elements



18

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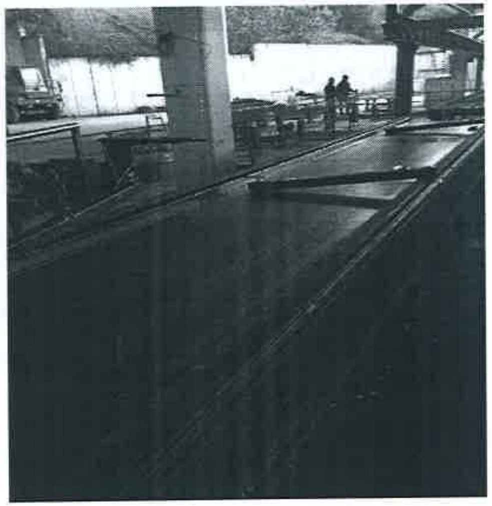
Production of prédalles

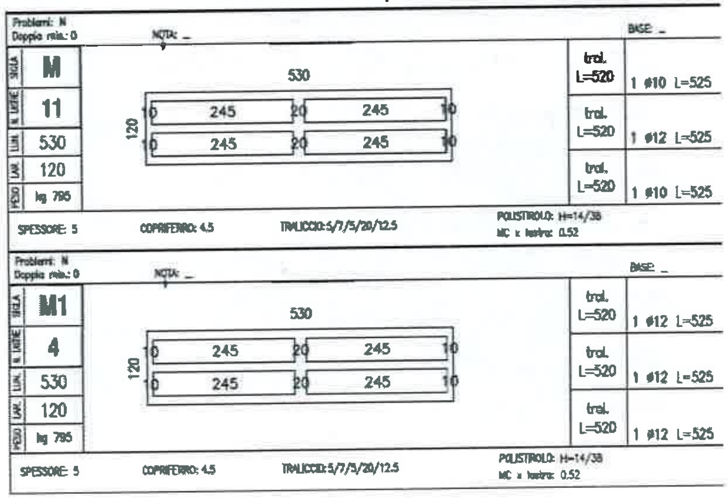
- Cleaning of the molds



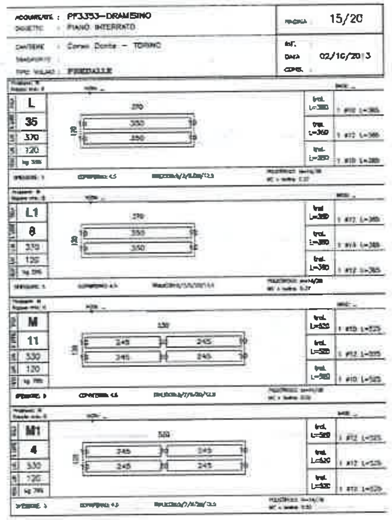
the truck has to be complementar to the dimensions of slabs

dimensions = f(transportation)

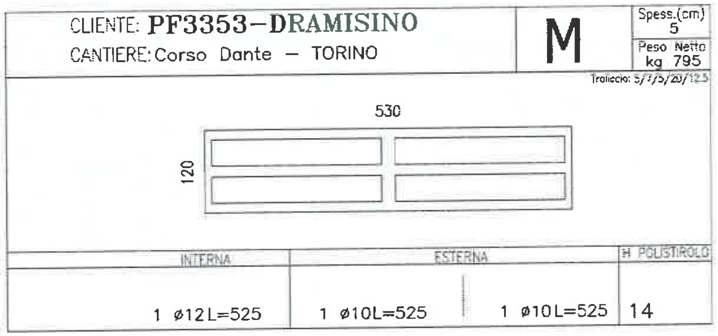
Production of prédalles



Production of prédalles



Production of prédalles



20

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Production of prédalles

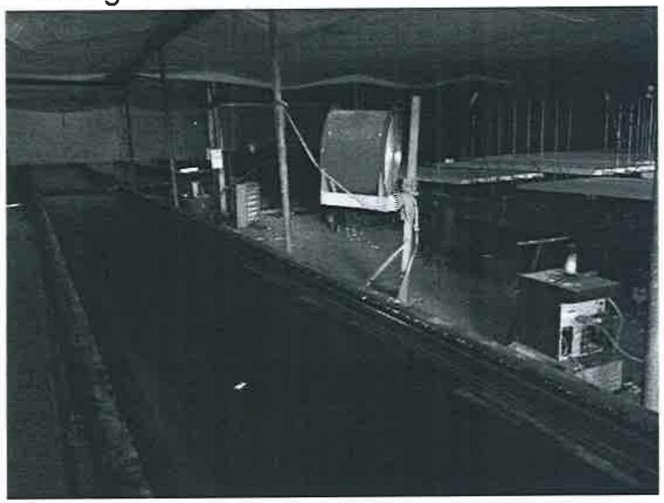
- rebar positioning

Production of prédalles

- Polystyrene blocks on the slabs

Production of prédalles

- Curing room



there are steam machines or windows to control internal temperature both in winter and summer

Humidity is also controlled but it's less important than temperature

Thank you very much for your attention

Info at:
www.bottaprefabbricati.com
paolo@bottaprefabbricati.com

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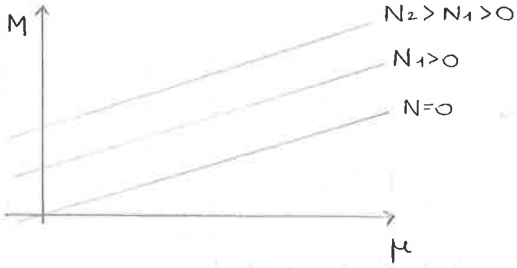
EX.

$C \neq O \equiv G$

(6) $M + Ne = E_c \mu_x I_o$

↳ If $N = 0$ we have the same situation as before

↳ If $N \neq 0$ the before relationship is traslated



Both concrete and steel haven't a linear relationship between σ and ε

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to the stage two curve. We know that when steel is yielded we are at the end of the second stage so we are in the third stage.

In the last stage (**STAGE THREE**): steel has a tension equal to the yielding one. Moment is equal to M_y (= moment of yielding in the rebar in tension)

⟹ the theoretical linear approach isn't real so we need to change something in the three equations we used in the bending beams

1) Equilibrium equation:

$N = \int_A \sigma \, dA$ (1)

$M + Ne = \int_A \sigma_z \cdot y \, dA$ (2)

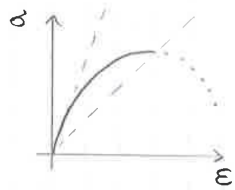
2) compatibility equation (plane sections remain plane after deformation)

$\varepsilon = \lambda + \mu_x \cdot y$ (3)

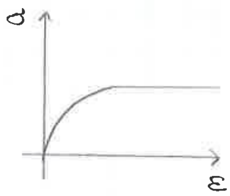
3) constitutive relationship of material

$\sigma \neq E\varepsilon$ (⟹ should be NON LINEAR)

POSSIBLE σ-ε RELATIONSHIP FOR CONCRETE

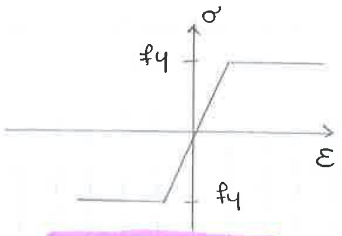


- No contribution of concrete in tension
- Except for the first part, the behaviour isn't linear
- SARAGIN



- PARABOLA – RECTANGLE
- Non-linear relationship between σ and ε

POSSIBLE σ-ε RELATIONSHIP FOR STEEL



- linear relationship up to the yielding stage, then it becomes constant

⟹ $\boxed{\text{NON LINEAR}}$

$\sigma = f(\varepsilon)$ σ is a general function of ε

Eq (3) into eq. (4)

$\sigma = \sigma(\lambda + \mu_x \cdot y)$ into eq. (1) and (2) we have a system:

$$\begin{cases} N = \int \sigma(\lambda + \mu_x \cdot y) \, dA & (5) \\ M + Ne = \int \sigma(\lambda + \mu_x \cdot y) \cdot y \, dA & (6) \end{cases}$$

⟹ equations and system are non-linear because σ-ε is non linear. In general if we know N and M we can obtain λ and μ_x and viceversa

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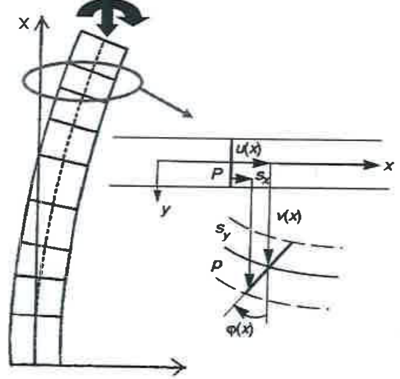
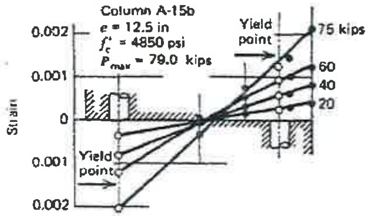
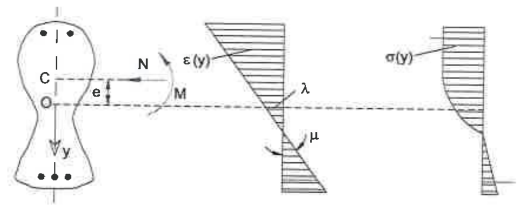
Theory of beams in bending

✓ In S. Venant body under bending and axial loads, each cross-section remains plane. This is true also for RC beams





In a cross-section



Equilibrium equations

$$\int_A \sigma \, dA = N \qquad (1) \qquad \int_A \sigma \, y \, dA = M + N e \qquad (2)$$

Compatibility equation

$$\varepsilon = \lambda + \mu \, y \qquad (3) \qquad \mu = \text{curvature}$$

The linear elastic case

✓Constitutive relationships

$$\sigma = E \, \varepsilon \qquad (4)$$ (E_s for steel and E_c for concrete)

✓Equations of a beams subjected to uniaxial bending and axial loads (Eqs. 3-4 into Eqs. 1-2)

$$N = E_c(\lambda A_0 + \mu S_0) \quad (5) \qquad M + N e = E_c(\lambda S_0 + \mu I_0) \quad (6)$$

where

$$A_0 = \int_A \frac{E}{E_c} dA \qquad S_0 = \int_A \frac{E}{E_c} y \, dA \qquad I_0 = \int_A \frac{E}{E_c} y^2 \, dA$$

depend on the geometrical properties of the cross-section and on the ratio $n=E_s/E_c$ (n method)

Moment-curvature relationships

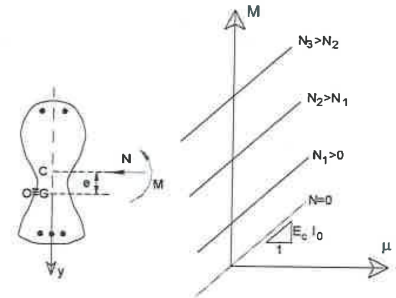
The behavior of a beam in bending can be summarized by the moment-curvature (M-μ or M-μ-N) relationships of its cross sections, which can be obtained from equations 5-6

✓For a given N

✓For a given μ

✓λ is obtained from eq. 5

✓M is obtained from eq. 6



$E_C I_0$ = flexural rigidity of the cross-section

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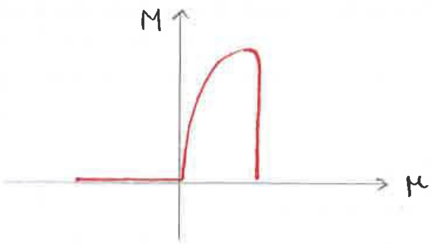
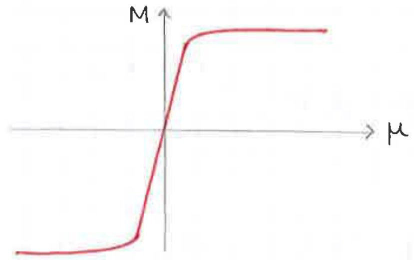
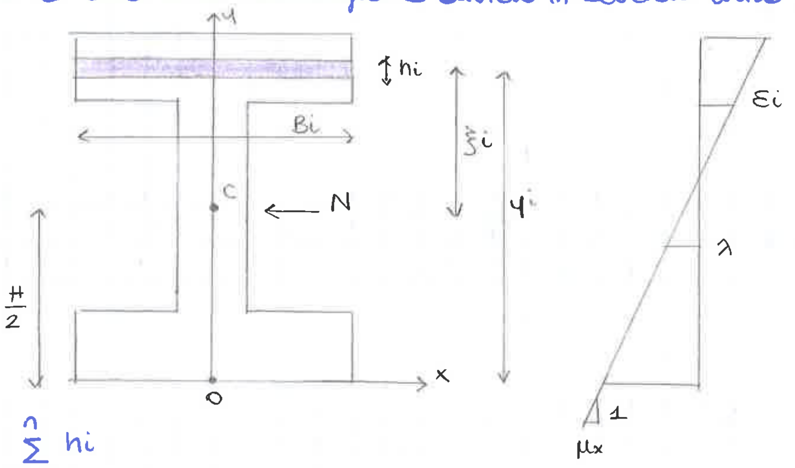
M-μ diagram (non symmetrical)



If we consider a symmetrical cross-section so change dimensions and As we have:



To compute M-μ relationship we consider N applied at H/2, then we move the origin O to C and each rectangle is divide in several little rectangles ($B_i \times h_i$)



$$H = \sum_{J=1}^{n} h_i$$

n : number of rectangles

$$\varepsilon_i = \lambda - \mu_x \cdot \xi_i$$
$$\sigma_i = \sigma(\varepsilon_i)$$

In this way (different rectangles) in state of the integral we have the sum

$$N = \int \sigma dA = \underbrace{\sum_{i=1}^{m} \sigma_{ci} \cdot h_i \cdot B_i}_{\text{CONCRETE}} + \underbrace{\sum_{J=1}^{p} A_{sJ}\, \sigma_{sJ}}_{\text{STEEL}} - \underbrace{\sum_{J=1}^{p} A_{sJ}\, \sigma_{cJ}}$$

m : number of stripes
p : number of steel levels

Part of the concrete stripes can be occupied by steel

$$M = \int \sigma \xi \, dA = \sum_{i=1}^{m} \sigma_{ci} \cdot h_i \cdot B_i \cdot \xi_i + \sum_{J=1}^{p} A_{sJ} (\sigma_{sJ} - \sigma_{cJ}) \xi_J$$

=> aproximation, close to the real value when h_i is close to zero
=> lead to a solution of the integral

EX 1

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