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# EMMEDUE STRUCTURAL ENGINEERING HANDBOOK (in accordance with EC)





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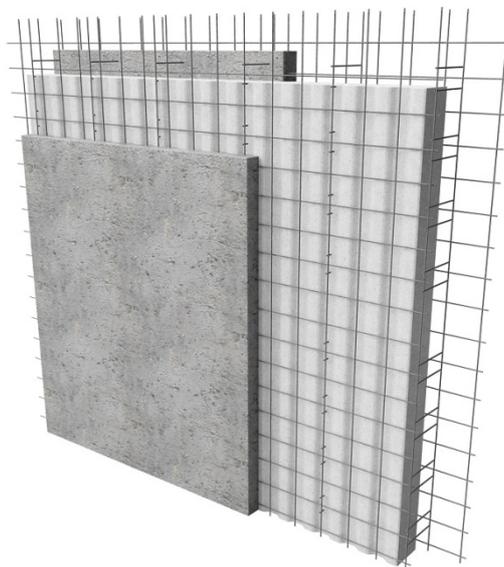


## INTRODUCTION

Constructions made with EMMEDUE building system are structures in which each panel is a load bearing component. The wall is composed of two external layers, realized with sprayed concrete (shotcrete), galvanized steel meshes and one inner polystyrene core. The EPS shape has been especially designed for the introduction of traditional plaster or shotcrete during on-site panel installation.

EMMEDUE provides a system of industrialized modular panels allowing for faster assembly than conventional systems. The EMMEDUE system fulfils the required structural and load-bearing functions, offering high thermal and sound resistance and a wide range of shapes and finishes to provide versatility in the design process.

Being a system of load-bearing walls of reinforced concrete, this structural handbook adopts EC2 (Design of concrete structures), using appropriate correction factors that best describe the mechanical behavior of the panel and which are calibrated on the basis of tests carried out on panels.



*Fig. 1 – EMMEDUE panel with two layer of shotcrete*

For the design of EMMEDUE panels it is necessary to know the behavior once subjected to different load stresses. From the laboratory test results are deducted the resistances of elements such as walls, slabs, beams and typical connections.

In professional practice the codes referring to reinforced concrete are adapted to EMMEDUE panel technology, this due to the similarity of behavior observed in the results of laboratory tests and the simplicity of applying the theories of calculus because the EMMEDUE technology essentially represents a typical lightened reinforced panel (through the use of concrete and steel mesh reinforcement).

All the components of a building can be constructed with EMMEDUE panels.

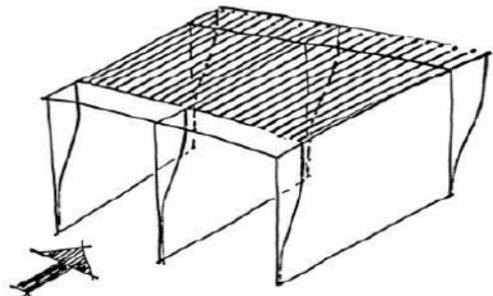
These buildings are conceived as structures formed by vertical and horizontal elements which are made of panels linked on site. The succession of panels linked, materializes all the elements of a construction: exterior walls, interior walls, floor slabs or roof deck slabs.

The projection of shotcrete layers on the panels converts all walls, slabs and joints on monolithic elements. The structure has thus achieved a high degree of internal hyperstaticity and also high ductility. This capability is generally not considered in the evaluation of resistant capabilities.

Bearing walls offer a resistance to horizontal forces in their plane much higher to that offered to horizontal forces perpendicular to the plane.

EMMEDUE walls are connected with each other and this give to the building the so-called "box-like behavior" which is very important against earthquakes.

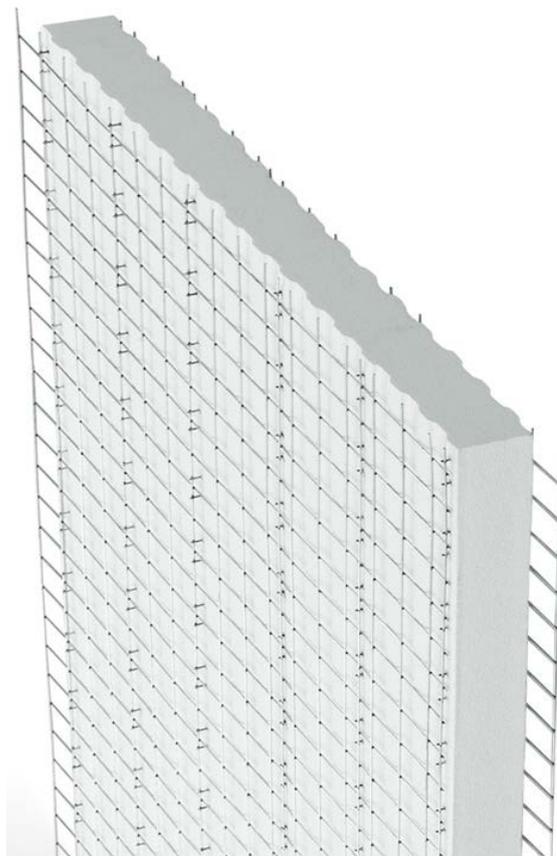
The seismic response of structures built with EMMEDUE building system is characterized, differently from concrete frame structures, by the above-mentioned box like behavior thanks to the favorable proportion between horizontal stiffness and mass.



## 1. EMMEDUE COMPONENTS

EMMEDUE single panels are made up of two electro-welded galvanized steel meshes positioned adjacent to the faces of a central block in wave-shape expanded polystyrene. The automatic industrial production assures the constant quality of the product. The mesh is also realized automatically and continuously by machines. The parameters that influence welding are set in these machines. The panels produced are characterized by a standard width of 120 cm and a variable length depending on the engineering requirements.

This type of panel can be used to made walls and slabs.



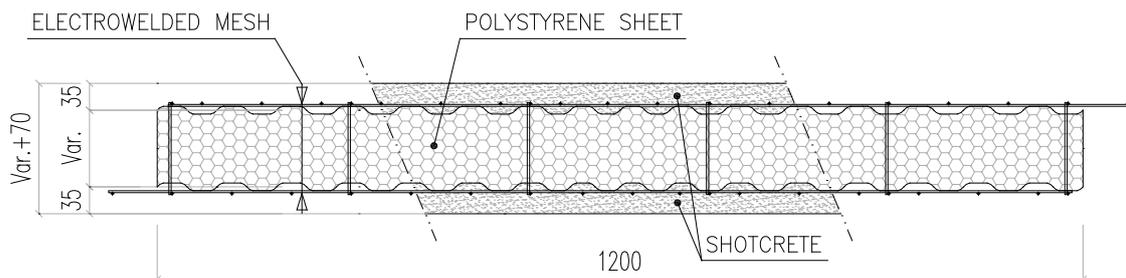
*Fig. 2 – PSM panel*

To see all the characteristics of the panels please refer to document “EMMEDUE Panel specifications”. For more specific guidelines regarding the installation of the panels on site please refer to the document “EMMEDUE Operator’s handbook”.

### 1.1. PANEL USED FOR BEARING WALLS (PSM)

The structures performed using the EMMEDUE PSM panels are completed by two external layers of shotcrete. This double layer of shotcrete, with a thickness of at least 35 mm and with a resistance of at least 250 daN/cm<sup>2</sup> must be applied to both sides of the panel in order to use it in structural work. The panel obtained in this way will form a double block of reinforced concrete with an isolating core of expanded polystyrene.

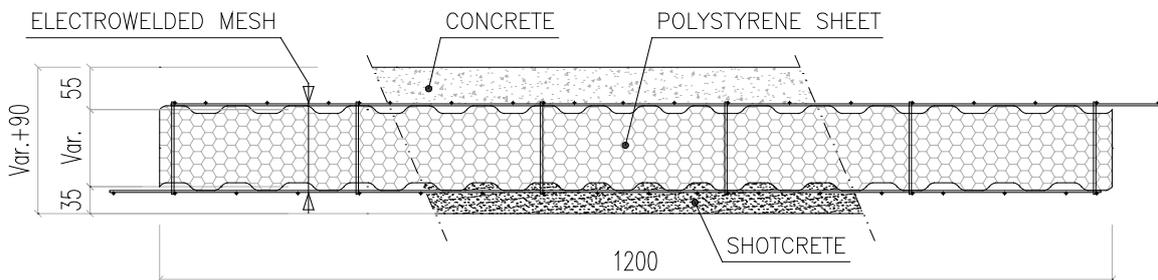
#### PSM PANEL SECTION



## 1.2. PANEL USED FOR SLABS (PSS1)

EMMEDUE single panel (PSS1) has the same characteristic of PSM panel; the difference is that PSS1 panel is used for slabs (floors or roofs), combining bearing capacity, isolating functions and disposable insulating formwork in a unique element. EMMEDUE panel is made up of spatial steel framework, which covers the intermediate polystyrene. The floor is completed on site with casting of concrete at the extrados and cement based spritz-beton (lime < 5% on cement weight) at the intrados. The panel obtained in this way will form a double block of reinforced concrete with an isolating core placed in the expanded polystyrene.

### PSS1 PANEL SECTION



The structural behavior of the slabs made with PSS1 panel is a plate behavior. This type of slabs are mainly designed as two-way slab and the loads are distributed over all the perimeter walls (see fig.3).

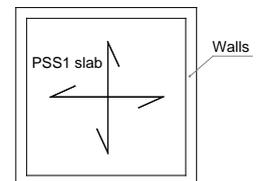
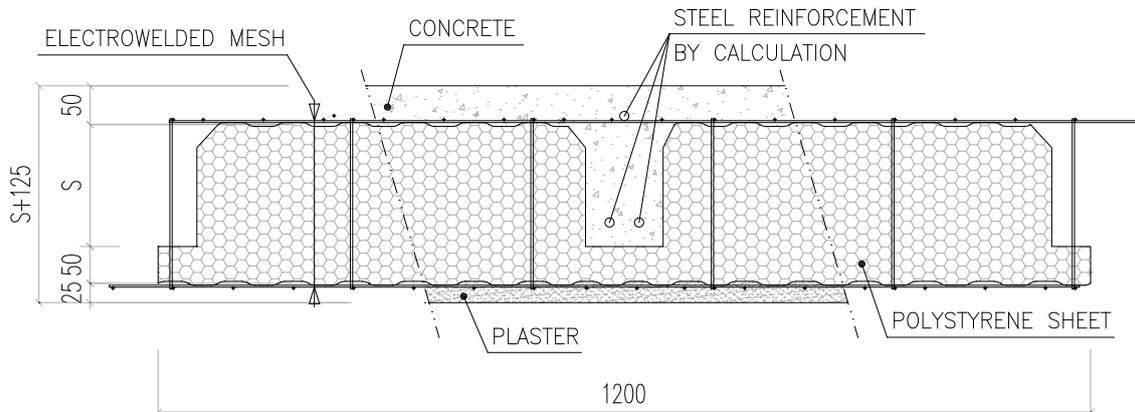


Fig. 3 – Two-way slab

### 1.3. PANEL USED FOR SLABS (PSG)

Floors made with EMMEDUE PSSG2 panels, realize floors and coverings with excellent isolation, resistance and cost performance. The panel produced by EMMEDUE is made up of shaped expanded polystyrene confined by two inter-connected electro-welded steel meshes. This is completed during laying with the addition of supplementary steel in the relevant beams and successive casting of the concrete. The job is completed at the intrados with traditional or pre mixed cement based plaster (lime <5% on cement weight). The floor obtained in this way will form a reinforced concrete ribbed plate with an isolating core of expanded polystyrene.

#### PSG PANEL SECTION



The structural behavior of the slabs made with PSG panel is a one-way behavior. The loads are distributed on two bearing walls (see fig.4).

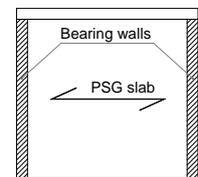


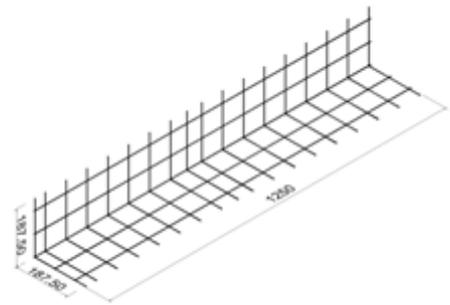
Fig. 4 – One-way slab

## 1.4. REINFORCEMENT MESHES

Made with galvanized steel wire ( $\varnothing$  2.5 mm), the meshes are used to reinforce all the openings and corner-joints between panels so to obtain a monolithic behavior of the structure.

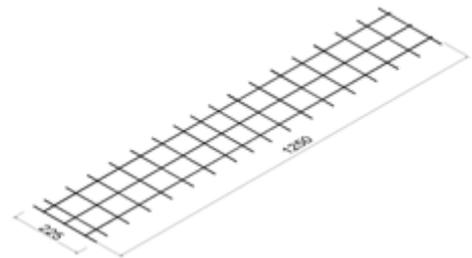
### ANGULAR MESH RG1

- used on every wall intersection;
- used on every wall and ceiling intersection;



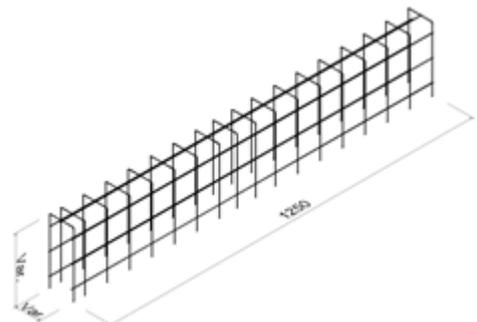
### FLAT MESH RG2:

- used on every opening corner;
- used on every discontinuity of the mesh;



### “U” SHAPED MESH RU:

- used around any opening;
- used to complete the top side of wall panels such as parapet and garden walls;



## 2. MATERIALS AND COMBINATIONS OF ACTIONS

### 2.1. CONCRETE

In Eurocode 2 the design of reinforced concrete is based on the characteristic cylinder strength rather than cube strength and should be specified according to BS 8500: Concrete – complementary British Standard to BS EN 206–17 (e.g. for class C20/25 concrete the cylinder strength is 20 MPa, whereas the cube strength is 25 MPa). Typical concrete properties are given in Table 2. The minimum characteristic cylinder strength required for the sprayed concrete on the Emedue panels is 20 MPa.

Strength classes for concrete															Analytical relation
$f_{ck}$ (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
$f_{cm}$ (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck} + 8$ (MPa)
$f_{ctm}$ (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm} = 0,30 \times f_{ck}^{(0.9)} \leq C50/60$ $f_{ctm} = 2,12 \times \ln [1 + (f_{cm}/10)] > C50/60$
$f_{ctk,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk,0,05} = 0,7 \times f_{ctm}$ frattile 5%
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk,0,95} = 1,3 \times f_{ctm}$ frattile 95%
$E_{cm}$ (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22 [(f_{cm})/10]^{0,3}$ ( $f_{cm}$ in MPa)
$\epsilon_{c1}$ (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	vedere figura 3.2 $\epsilon_{c1} (\text{‰}) = 0,7 f_{cm}^{0,31} < 2,8$
$\epsilon_{cu1}$ (‰)	3,5								3,2	3,0	2,8	2,8	2,8	vedere figura 3.2 per $f_{ck} \geq 50$ MPa $\epsilon_{cu1} (\text{‰}) = 2,8 + 27 [(98 - f_{cm})/100]^4$	
$\epsilon_{c2}$ (‰)	2,0								2,2	2,3	2,4	2,5	2,6	vedere figura 3.3 per $f_{ck} \geq 50$ MPa $\epsilon_{c2} (\text{‰}) = 2,0 + 0,085 (f_{ck} - 50)^{0,53}$	
$\epsilon_{cu2}$ (‰)	3,5								3,1	2,9	2,7	2,6	2,6	vedere figura 3.3 per $f_{ck} \geq 50$ MPa $\epsilon_{cu2} (\text{‰}) = 2,6 + 35 [(90 - f_{ck})/100]^4$	
$n$	2,0								1,75	1,6	1,45	1,4	1,4	per $f_{ck} \geq 50$ MPa $n = 1,4 + 23,4 [(90 - f_{ck})/100]^4$	
$\epsilon_{c3}$ (‰)	1,75								1,8	1,9	2,0	2,2	2,3	vedere figura 3.4 per $f_{ck} \geq 50$ MPa $\epsilon_{c3} (\text{‰}) = 1,75 + 0,55 [(f_{ck} - 50)/40]$	
$\epsilon_{cu3}$ (‰)	3,5								3,1	2,9	2,7	2,6	2,6	vedere figura 3.4 per $f_{ck} \geq 50$ MPa $\epsilon_{cu3} (\text{‰}) = 2,6 + 35 [(90 - f_{ck})/100]^4$	

Tab. 2 – Typical concrete properties

The stress-strain relation for the design of cross sections is represented in fig.2.1

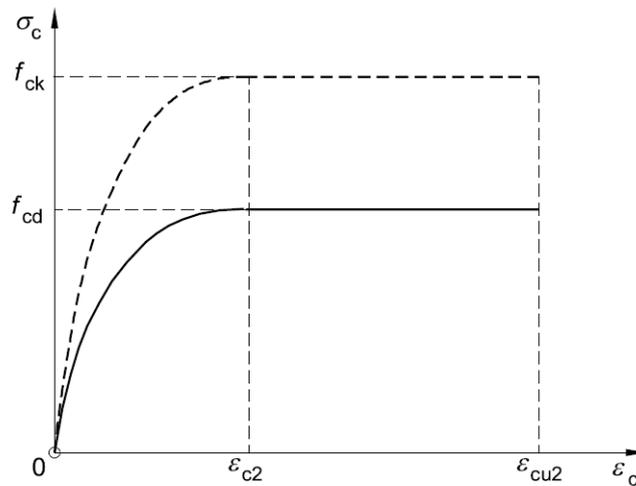


Fig. 2.1 – Concrete stress strain relation

where:

$\sigma_c = f_{cd} \left[ 1 - \left( 1 - \frac{\varepsilon_c}{\varepsilon_{c2}} \right)^n \right]$	for	$0 \leq \varepsilon_c \leq \varepsilon_{c2}$
$\sigma_c = f_{cd}$	for	$\varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2}$
$n = 1,4 + 23,4 \left[ \frac{90-f_{ck}}{100} \right]^4$	for	$f_{ck} \geq 50 \text{ MPa}$ otherwise 2,0
$\varepsilon_{c2} (\text{‰}) = 2,0 + 0,085 (f_{ck}-50)^{0,53}$	for	$f_{ck} \geq 50 \text{ MPa}$ otherwise 2,0
$\varepsilon_{cu2} (\text{‰}) = 2,6 + 35 \left[ \frac{90-f_{ck}}{100} \right]^4$	for	$f_{ck} \geq 50 \text{ MPa}$ otherwise 3,5

This relation can be simplified as a bi-linear relation:

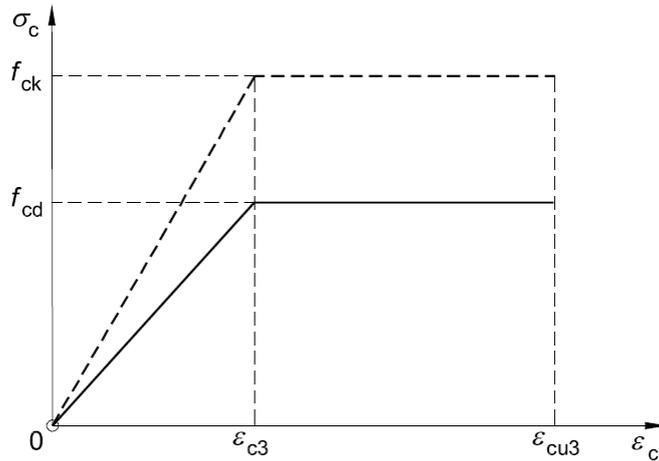


Fig. 2.2 – Concrete stress strain bi-linear relation

where:

$$\begin{aligned} \epsilon_{c3} (\%) &= 1,75 + 0,55 [(f_{ck}-50)/40] && \text{for } f_{ck} \geq 50 \text{ MPa} && \text{otherwise } 1,75 \\ \epsilon_{cu3} (\%) &= 2,6 + 35 [(90-f_{ck})/100]^4 && \text{for } f_{ck} \geq 50 \text{ MPa} && \text{otherwise } 3,5 \end{aligned}$$

The relationship between concrete compressive distribution and concrete strain shall be assumed to be rectangular. For design, the code allows to use of an equivalent rectangular compressive stress distribution (stress block) to replace the more exact concrete stress distribution.

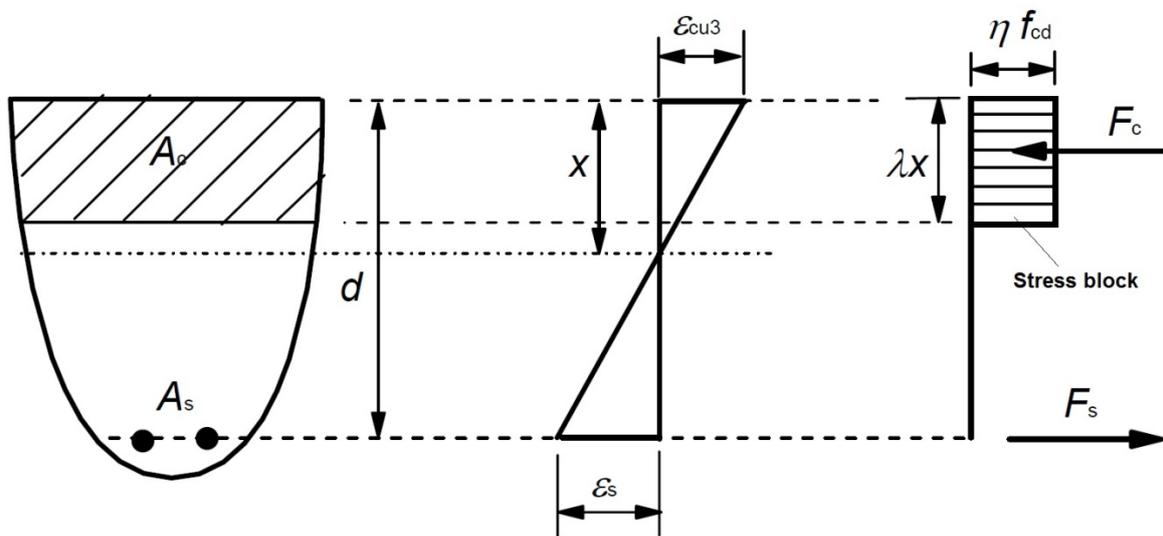


Fig. 2.3 – Stress block

where:

$$\lambda = 0,8 \quad \text{for } f_{ck} \leq 50 \text{ MPa}$$

$$\lambda = 0,8 - (f_{ck} - 50)/400 \quad \text{for } 50 \text{ MPa} \leq f_{ck} \leq 90 \text{ MPa}$$

and:

$$\eta = 1,0 \quad \text{for } f_{ck} \leq 50 \text{ MPa}$$

$$\eta = 1,0 - (f_{ck} - 50)/200 \quad \text{for } 50 \text{ MPa} \leq f_{ck} \leq 90 \text{ MPa}$$



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The design compressive and tensile strength (as indicated in the EC2 chapter 3.1.6) are:

Design compressive strength, ( $f_{cd}$ )

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

Design tensile strength, ( $f_{ctd}$ )

$$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_c$$

$\alpha_{cc}$  and  $\alpha_{ct}$  are coefficients which take account of long term effects on the compressive and tensile strengths and of unfavorable effects resulting from the way the load is applied.

$\gamma_c$  is the partial safety factor for the concrete. It depends from the design situations, particularly:

$\gamma_c = 1,5$  for persistent and transient

$\gamma_c = 1,2$  for exceptional



## 2.2. STEEL

The wire used is realized with galvanized steel, so it meets the durability requirements and it is usually made of  $\Phi$  2,5 (net) and  $\Phi$  3 (connectors).

The idealized and design stress strain relations for reinforcing steel is shown in the graphic below:

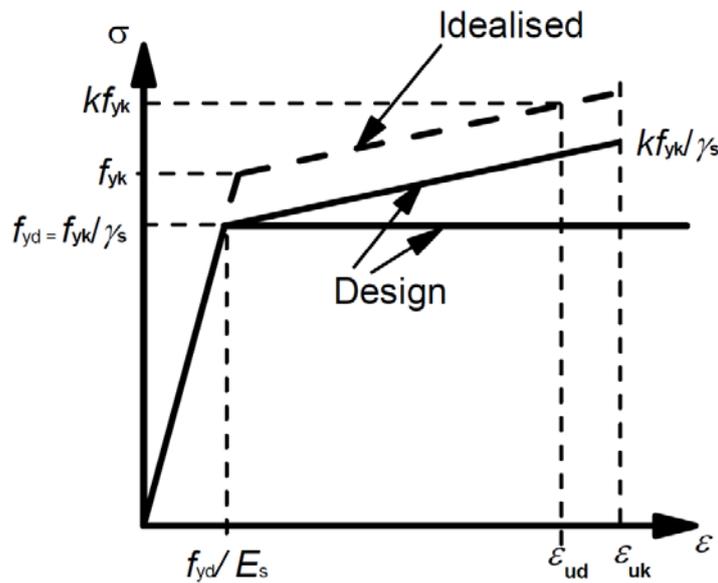


Fig. 2.4 –Steel stress strain diagram

According to EC2, alternative design stress/strain relationships are permitted.

In our case we use horizontal top branch with no strain limit.

The design value of steel yield strength ( $f_{yd}$ ) is derived from the value of characteristic yield stress ( $f_{yk}$ ):

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

$\gamma_s$  is the partial safety factor for steel. It depends from the design situations, particularly:

$\gamma_s = 1,15$  for persistent and transient

$\gamma_s = 1,0$  for exceptional

From tests carried out on steel wires with a diameter of 2.5 mm was possible to evaluate the elastic modulus of the steel, in particular the following values were obtained:

$$\varepsilon_{yk} = 0,305889 \%$$

$$f_{yk} = 600 \text{ MPa}$$

$$E_s = \frac{\sigma_1}{\varepsilon_1} = 1999486 \text{ daN/cm}^2 \cong 2000000 \text{ daN/cm}^2$$

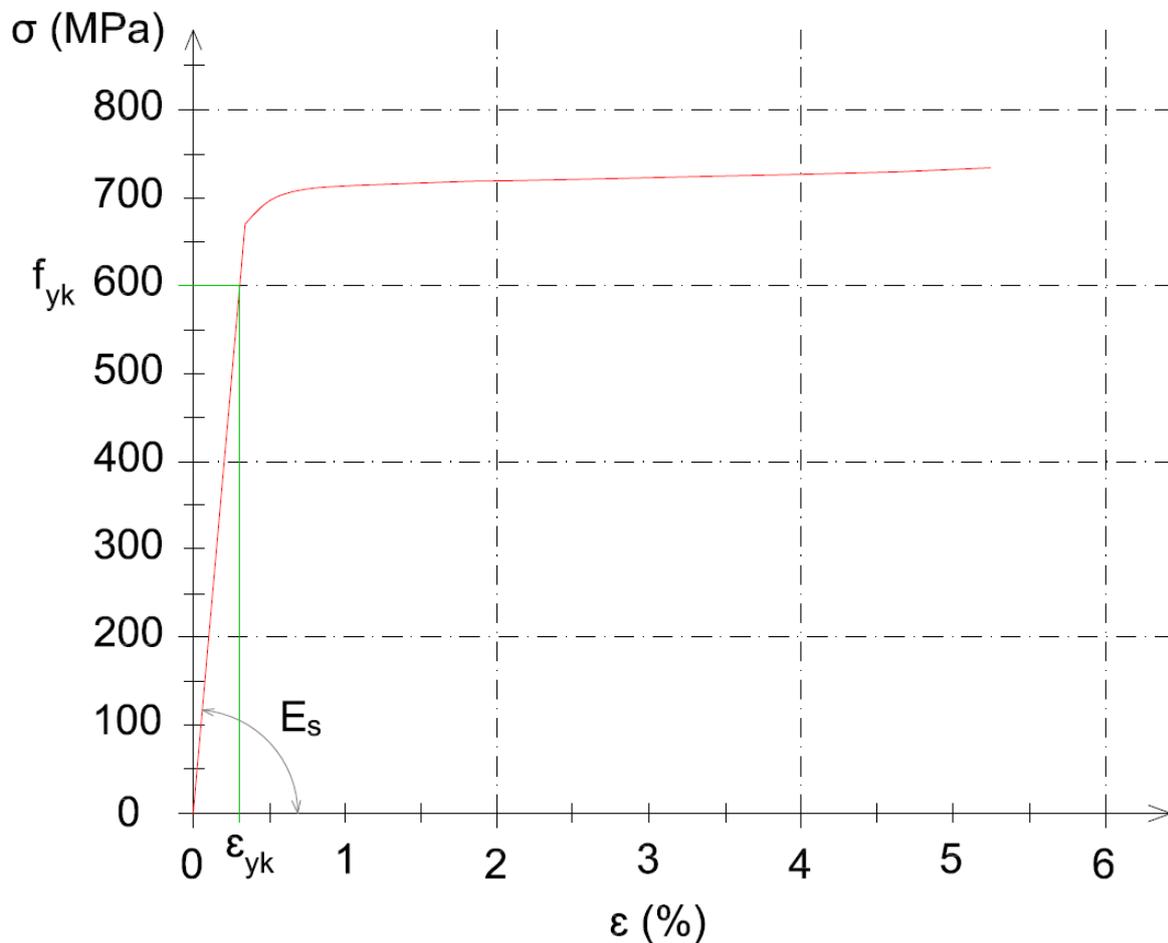


Fig. 2.5 – Test carried out on galvanized wire ( $\Phi 2,5$  mm) used for EMMEDUE panels

For the calculation we use a value of  $f_{yk} = 600 \text{ N/mm}^2$ , also if the tests shown a bigger value.

### 2.3. COMBINATION OF ACTIONS

In the Eurocode the term “combination of action” is specifically used for the definition of the magnitude of actions to be used when a limit state is under the influence of different actions. It should not be confused with “load cases”, which are concerned with the arrangement of the variable actions to give the most unfavorable conditions and are given in the material Eurocode. The following process can be used to determine the value of actions used for analysis:

1. Identify the design situation (e.g. persistent, transient, accidental).
2. Identify all realistic actions.
3. Determine the partial factors (see below) for each applicable combination of actions.
4. Arrange the actions to produce the most critical conditions.

Where there is only one variable action (e.g. imposed load) in a combination, the magnitude of the actions can be obtained by multiplying them by the appropriate partial factors.

Where there is more than one variable action in a combination, it is necessary to identify the leading action ( $Q_{k,1}$ ) and other accompanying actions ( $Q_{k,i}$ ). The accompanying action is always taken as the combination value.

Symbol	Definition
$G_k$	Characteristic value of permanent action
$Q_k$	Characteristic value of single variable action
$\gamma_G$	Partial factor for permanent action
$\gamma_Q$	Partial factor for variable action
$\psi_0$	Factor for combination value of a variable action
$\psi_1$	Factor for frequent value of a variable action
$\psi_2$	Factor for quasi-permanent value of a variable action
$\xi$	Combination factor for permanent actions

Fig. 2.6 –Selected symbols for Eurocode

The fundamental combination of actions (at the ULS) may be expressed as :

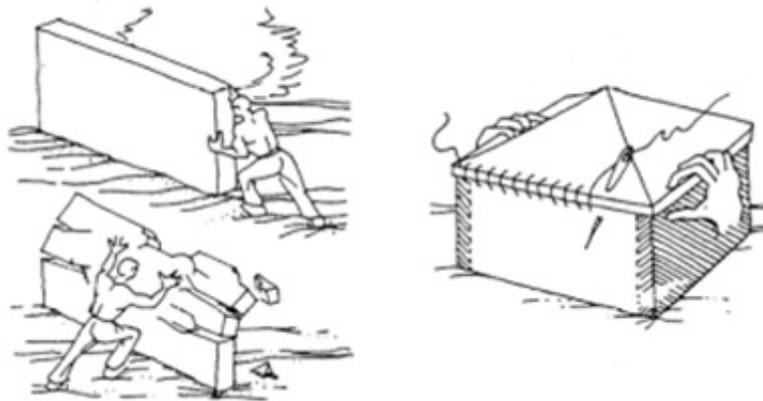
$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

For more information, see the table A1.2 (B) Eurocode.

### 3. INDICATIONS FOR THE STRUCTURAL ANALYSIS

#### 3.1. GENERAL RULES FOR THE DESIGN

- The assessment of the structural safety has to be done with the limit state method.
- Must be respected all the principles and method relatives to concrete construction (EC2).
- In addition must be followed all the indication for the seismic design of concrete structures (EC8).
- The structure must be designed and carried out to guarantee a box behavior.



- The walls perform the double function of bearing walls and braced walls.
- The slabs must be rigid and resistant.
- There aren't default height limits; the only height limits are those defined from the achievement of the ultimate characteristics.
- The resistant elements must be continuous for all the height of the building.
- Prefer compact shapes in plan, rectangular to ensure a requirement of uniform deformation.
- Avoid shapes very elongated because the structure could be placed on various land (with different seismic motions); alternatively subdivide the structure with joints.
- Is recommended the construction of bond-beam.
- Ensure the continuity of the walls in height.

### 3.2. METHODS OF ANALYSIS

Can be used all the methods indicated in the Eurocode:

- linear static analysis
- dynamic linear analysis
- nonlinear static analysis
- nonlinear dynamic analysis

The use of nonlinear methods is permitted only if there is an adequate and exhaustive preliminary study of the constitutive models.

The calculation of the design stresses should be performed with a suitable method to correctly represent the distribution of stiffness and masses, in accordance with the result of the experimental tests.

### 3.3. COMBINATION OF THE MODAL RESPONSES

According to Eurocode, the effects relatives to the single modes must be combined to assess the effects of the earthquake along the principal directions.

It is recommended the use of the complete quadratic combination (CQC).

$$E = \sum_i \sum_j (\rho_{ij} \cdot E_i \cdot E_j)^{0.5}$$

where:

- $E_x$  is the effect of the application of the seismic actions in X direction
- $E_i, E_j$  are the effects of the application of the seismic actions in X direction, due to the i and j vibration modes
- $\rho_{i,j}$  is the correlation factor between i and j modes (depend from periods and damping)



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### 3.4. COMBINATION OF THE SEISMIC ACTIONS

According to Eurocode, the responses to the seismic actions must be calculated separately for each component and then combined according to the formula:

$$1.0 E_x + 0.3 E_y$$

with the rotation of the multiplication factors and consequently identification of the more burdensome effects.

Like prescribed from Eurocode, is necessary to take into account the uncertainty in the location of the masses by applying accidental eccentricity of at least 5% of the size of the building measured perpendicular to the direction of the seismic actions.

### 3.5. NUMERICAL ANALYSIS

Modeling of the structures in question, in addition to the issue of traditional structures, it is necessary to evaluate the following effects:

- not monolithic section

*Solution:* These effects will be considered adequately reducing the stiffness of the section

- type of clamping (constraints) between the elements in the T and L wall-wall connections, wall-slab and wall-foundation

*Solution:* These effects will be considered inserting junctions in the nodes of the mathematic model



For the analysis of this type of structures can be used two types of modeling:

- model based on the use of bi-dimensional finite elements (shell):



*Advantages:*

- because the structure consists mainly of walls, the creation of this type of finite element model is more intuitive (but not less laborious);
- regardless of the limitations of linear models, the local distributions of the stresses between the different elements are picked with greater accuracy;

*Disadvantages:*

- the most common calculation software, when using two-dimensional finite elements, provide stresses on output, therefore it is necessary to integrate the stresses (to get the actions) before making any modal combinations

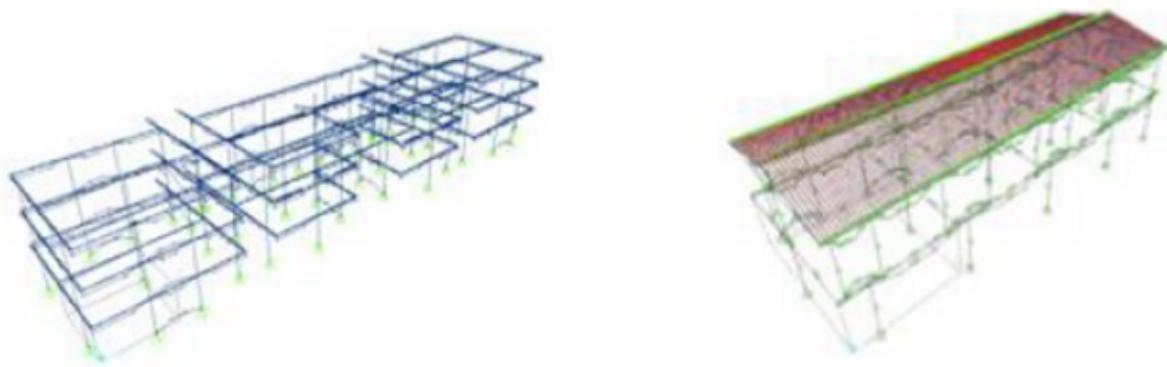
It should be noted that when for effects of the earthquake we have internal forces acting on structural elements, these actions must be combined (pay attention to interstorey displacement).

Working with models that use two-dimensional elements the most common calculation software provide the results in terms of internal stresses and not of internal forces.

Then comes the need to integrate these stresses BEFORE performing the modal combination (CQC).

Integration must be done at all critical sections where will be conducted the verifications

- models based on the use of monodimensional macro-elements (frame):



*Advantages:*

- the model is less onerous from a computational point of view;
- the data processing is easier (consequently it is easier to carry out checks on the accuracy of the data and results obtained);

*Disadvantages:*

- the creation of the model is more complicated;
- the distributions of stresses at the local level are less accurate (globally the result changes only marginally)

## TRICKS for the structural model

The stiffness of the structural elements for the ULS verifications must be reduced to 20% of the initial value, in accordance with the operation shown by the panels tested at the laboratories of the Structural Mechanics Department of the University of Pavia and Eucentre.

This decrease is due to cracking phenomena due to bending and shear.

For the input in the FEM model, three equivalent parameters were studied (modulus of elasticity, thickness and consequently a modified specific weight). In this way it is possible to simplify the different behavior of the panel in both directions. An example for a PSM80 panel is reported below:

DEFINITION OF THE PARAMETERS FOR THE STRUCTURAL MODEL				
DEFINITION	SYMBOL	VALUE	DIM.	NOTE
Characteristic compressive cubic strength of concrete	$R_{ck}$	25	Mpa	at 28 days
Characteristic compressive cylinder strength of concrete	$f_{ck}$	20,75	Mpa	at 28 days
Mean value of concrete cylinder compressive strength	$f_{cm}$	28,75	Mpa	
Secant modulus of elasticity of concrete	$E_{cm}$	30200	Mpa	not cracked
Modulus of elasticity of the panel (along the plane)	$E_{II}$	8000	MPa	according to ETA
Specific weight of the concrete	$\gamma$	25	kN/m <sup>3</sup>	
Thickness of 1 layer of shotcrete	$b/2$	3,5	cm	
Thickness of polystyrene	$s$	8	cm	
Total thickness of the panel	$h$	15	cm	(polystyrene + 2 layers of shotcrete)
Reduction factor for the out-of-plane stiffness	$k$	0,2		
	$E'B$	560000	N/mm	
	$E'B^3$	1,73E+10	Nmm	
Equivalent thickness	$B$	17,57	cm	to input in the model
Equivalent modulus of elasticity	$E'$	3187	Mpa	to input in the model
Equivalent specific weight	$\gamma'$	10,0	kN/m <sup>3</sup>	to input in the model

N.B.: Refer to the “Equivalent parameters” spreadsheet.

For seismic analysis will be necessary to include the structural masses and the 30% of the variable masses (for residential buildings).

In order to avoid the presence of numerous local modes related to the vibration out of the plane of the walls, the mass can be concentrated at the level of the horizontal elements.

If the slabs are considered infinitely rigid in their plane, the mass of each plan will be concentrated in the center of mass of the plane.



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### 3.6. BEHAVIOR FACTOR

Specific numerical studies, carried out on the results of the experimental tests, have led to the definition of a behavior factor  $q_0$  which is variable from 1.5 to 2. According to EC8 this coefficient may possibly be changed through the adoption of a regularity factor and may be used for the definition of the design spectrum.

The final behavior factor will be the following:

$$q = q_0 \cdot K_s \cdot K_r$$

where:

- $K_s$  is the over strength factor equal to 1.2 for regular structures on plan and 1.1 for the irregular
- $K_r$  is the reduction factor for regular structures in height equal to 1, 0.8 for irregular
- $q_0$  is the basic behavior factor



## 4. ULS VERIFICATIONS

### 4.1. HYPOTESIS

The calculation of the element resistance with the "strength design method" requires that two basic conditions are satisfied: static equilibrium and compatibility of deformations. The first condition requires that compressive and tensile forces acting in cross section are balanced, while the second condition requires the compatibility between the deformations of the shotcrete and reinforcement under ultimate conditions with design hypothesis. From a rational and practical point of view, the determination of the nominal resistances of structural elements with EMMEDUE panels can be based on the general hypothesis set for reinforced concrete sections.

#### HYPOTHESIS N.1

"The specific deformations in the reinforcement and shotcrete should be assumed directly proportional to the distance from the neutral axis".

It is assumed that the normal plane sections to the flexion axis remain plane also after the flexion.

#### HYPOTHESIS N.2

"The maximum deformation in the extreme compressive fiber is assumed equal to  $\epsilon_{cu} = 0.003$ ". In the figure n.1 is shown the adaptation of design hypothesis 1 and 2 to the EMMEDUE panels.

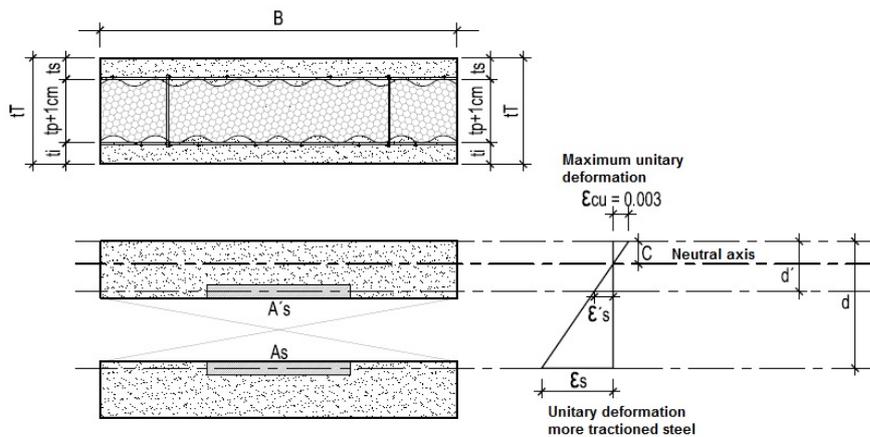


Fig. 1

### *HYPOTHESIS N.3*

"The stress in the steel  $f_s$  under the specified yield strength of the steel, will be taken as  $E_s$  (modulus of elasticity of the steel) multiplied for the steel strain specific  $\varepsilon_s$ .

For strain greater than  $f_y/E_s$ , the stress in the reinforcement is considered independent from the strain and equal to  $f_y$ ". The force developed in the tension or compression reinforcement is function of the specific strain  $\varepsilon_s$  in the reinforcement, and is calculated as follows:

$$\text{When } \varepsilon_s < \varepsilon_y \text{ (yield strain):} \\ f_s = E_s \varepsilon_s \rightarrow A_s f_s = A_s E_s \varepsilon_s$$

$$\text{When } \varepsilon_s \geq \varepsilon_y \text{ (yield strain):} \\ f_s = E_s \varepsilon_s = f_y \rightarrow A_s f_y$$

### *HYPOTHESIS N.4*

"In the design of EMMEDUE panels under bending stresses will not be considered the tensile strength of micro-concrete".

However flexural strength of micro-concrete also known as modulus of rupture, is a variable property more than compression strength, and its value is about 8% to 12% of the compressive strength.

### *HYPOTHESIS N.5*

"It will be assumed a stress in the micro-concrete equal to  $0.85 f_{cd}$  uniformly distributed in an equivalent compressed area limited by the edges of the cross section and a line parallel to the neutral axis located at a distance  $0.8X$  be assumed from the fiber with the high specific compressive strain. The distance "X" between the fiber with the high specific compressive strain and the neutral axis will be measured in a perpendicular direction to said axis.

All the structural elements (load bearing walls and connecting beams), should be checked against the different failure mechanisms that can be subjected:

- Bending
- Compression + bending moment
- Buckling
- Shear
  - Shear-Compression
  - Shear-Traction
  - Shear-Sliding

## 4.2. BENDING

The EMMEDUE elements, differently from the classic sections in reinforced concrete, have a steel area, distributed along the height of the element. Therefore, calculation of the resisting moment is conditioned in this case from the knowledge of the stresses at several points along the height of the element, points that represent the position of the steel wires of the panel. This makes it less simple analytical approach than the classical section of reinforced concrete with the reinforcements only arranged in two positions, although you can use a simplified method that is proposed below.

The simple bending is characterized by the absence of axial force, this results in having to set the equation (4.4) equal to zero:

$$N_c + N'_s + T_s = 0$$

where:

$N_c$  = force due to the compressed concrete - negative sign

$N'_s$  = force due to the resultant force of the compressed bars – negative sign

$T_s$  = force due to the resultant force of the tensioned bars – positive sign

The forces of the equation (4.4) may be expressed by the following relationships:

$N_c = 0,8 B f_{cd} X$  because the section is a partialized section

$N'_s = \rho (X-c) f_{yd}$

$T_s = \rho (d_n - X) f_{yd}$

where:

$X$  is the neutral axis;

$c$  is the concrete cover: the distance between the first bar and the compressed side;

$d_n$  is the distance between the last bar and the compressed side;

$\rho$  is the reinforcement “density” =  $2nA_{sl}/(d_n - c)$

$n$  is the number of the longitudinal wires

$A_{sl}$  is the area of one steel wire

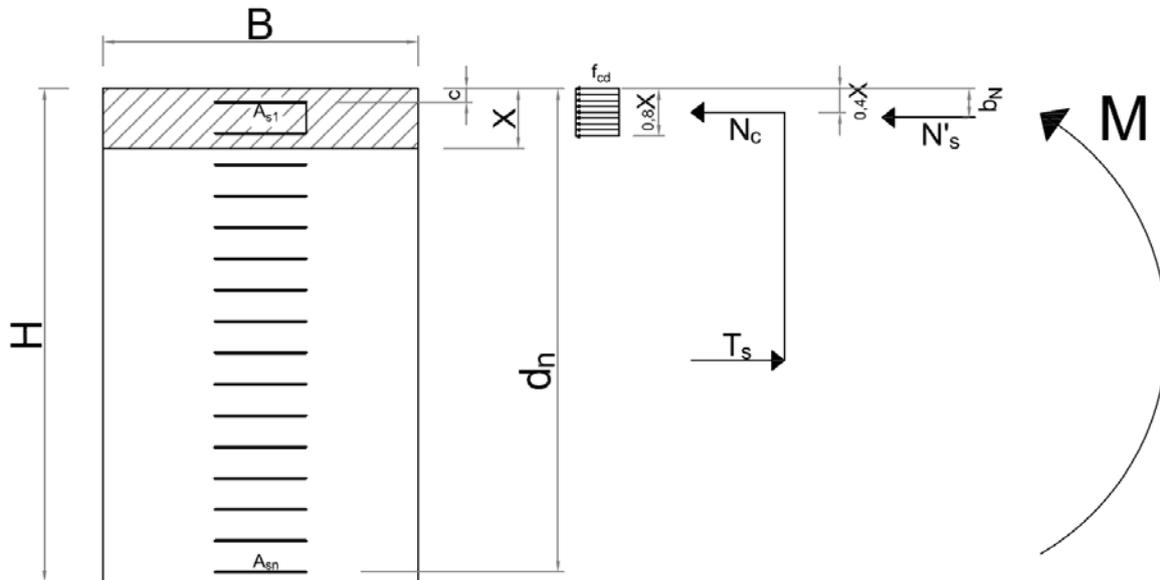


Fig. 4.1 – Rectangular section – Simple bending

It is assumed that the stress of the steel bars is equal to the yield strength. If the analyzes aren't made on elements which have considerable heights the error committed is almost negligible.

Developing relations in the above (4.4), we obtain the neutral axis in closed form:

$$X = \frac{\rho(d_n+c)f_{yd}}{0,8Bf_{cd}+2\rho f_{yd}}$$

known the neutral axis  $X$ , is possible to calculate the resistance moment  $M_{Rd}$  using the relationship:

$$M_{Rd} = N_c 0,4X + N'_s b_N + T_s b_T$$

## EXAMPLE – Bending

In this example we will calculate the resistance moment  $M_{Rd}$  for a PSM80 beam panel with an height equal to 60 cm. The characteristics of the panel are indicated below:

### GEOMETRICAL AND MECHANICAL CHARACTERISTICS

Height of the panel (H) = 60 cm

Thickness of the shotcrete (B) = 7 cm (is the sum of the 2 layers of 3,5 cm)

Diameter of the steel wire = 2,5 mm

Transversal pitch of the steel mesh = 112 mm

Longitudinal pitch of the steel mesh = 67 mm

Characteristic cube strength of concrete ( $f_{ck,CUBE}$ ) = 25 MPa

Characteristic cylinder strength of concrete ( $f_{ck}$ ) =  $f_{ck,CUBE} * 0,83 = 25 \text{ MPa} * 0,83 = 20,75 \text{ MPa}$

Coefficient for long term effect ( $\alpha_{cc}$ ) = 0,85 (from the Italian rules NTC08, according to EC2)

Design compressive strength ( $f_{cd}$ ) =  $\alpha_{cc} f_{ck} / \gamma_c = 0,85 * 20,75 / 1,5 = 11,76 \text{ MPa}$

Yield strength of reinforcing steel ( $f_{yk}$ ) = 600 MPa

Yield design strength of reinforcing steel ( $f_{yd}$ ) =  $600 \text{ MPa} / 1,15 = 521,74 \text{ MPa}$

Number of the longitudinal wires (n) = 9

Concrete cover (c) = 3 cm

Distance between the last bar and the compressed side ( $d_n$ ) = 57 cm

Area of one steel wire ( $A_{sl}$ ) =  $4,91 \text{ mm}^2$

Reinforcement “density” ( $\rho$ ) =  $2nA_{sl}/(d_n-c) = 2*9*4,91 / (570-34) = 0,16485 \text{ mm}$

Is possible to calculate the position of the neutral axis:

$$X = \frac{\rho(d_n+c)f_{yd}}{0,8Bf_{cd}+2\rho f_{yd}} = \frac{0,16485*(570+34)*521,74}{0,8*70*11,76+2*0,1648*521,74} = 62,55 \text{ mm}$$

CALCULATION OF THE INTERNAL FORCES:

$$N_c = 0,8 B f_{cd} X = 0,8 * 70 \text{ mm} * 11,76 \text{ MPa} * 62,55 \text{ mm} = - 41188 \text{ N}$$

$$N'_s = \rho (X - c') f_{yd} = 0,16485 * (62,55 - 34) * 521,74 = - 2456 \text{ N}$$

$$T_s = \rho (d_n - X) f_{yd} = 0,16485 * (570 - 62,55) * 521,74 = 43644 \text{ N}$$

CALCULATION OF THE RESISTANCE MOMENT  $M_{Rd}$ :

$$\begin{aligned} M_{Rd} &= N_c 0,4X + N'_s b_N + T_s b_T = -41188 * 0,4 * 62,55 - 2456 * 48,28 + 43644 * 316,28 \\ &= -1,03 - 0,12 + 13,80 = 12,65 \text{ kNm} \end{aligned}$$

where:

$$b_N = \frac{(X - c')}{2} + c' = \frac{(62,55 - 34)}{2} + 34 = 48,28 \text{ mm}$$

$$b_T = \frac{(d_n - X)}{2} + X = \frac{(570 - 62,55)}{2} + 62,55 = 316,28 \text{ mm}$$

N.B.: Refer to the "Beam panel" spreadsheet to calculate the resistance bending moment.



#### 4.4. BUCKLING

As prescribed by the NTC08 (4.1.2.1.7.2) and EC2 (5.8.3), in order to prevent the out of plane instability and in the absence of more precise formulations, for each combination of load provided, you will have to comply with the following limits:

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

This formula defines a slenderness limit as a function of the applied load and the geometry of the section, as well as of the direction of the moments at the ends. This slenderness represent the limit for which is possible to neglect the effects of the second order and then the phenomenon of instability.

In the formula is possible to define:

$A$  = is a parameter according to the viscosity that the EC2 suggested to put equal to 0,7

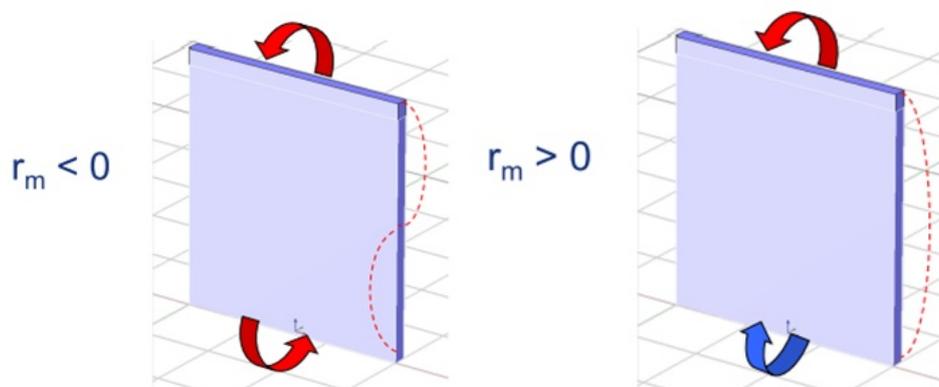
$B$  = is a parameter as a function of the mechanical relationship of the reinforcement that the EC2 suggests to put equal to 1.1

$C = 1,7 - r_m$

$$r_m = |M_{01}|/|M_{02}|$$

$M_{01}, M_{02}$  = moments of the first order at the ends. If cause traction on the same side, it is recommended that  $r_m$  is assumed positive ( $C \leq 1.7$ ), otherwise negative ( $C > 1.7$ )

$n = N_{ED}/(A_c * f_{cd})$  is the adimensional axial force



Substituting the above values to the parameters A and B is possible to obtain the following formula:

$$\lambda_{lim} = 15,4 * C / \sqrt{n}$$

The slenderness  $\lambda$  (section 5.8.3.2 EC2) is calculated as the ratio between the buckling length and radius of inertia of the section of no-cracked concrete:

$$\lambda = l_o / i_{min,eff}$$

where:

$$l_o = h \cdot \beta$$

$l_o$  is the buckling length, may be assumed equal to 70% of the wall height when the horizontal reinforcement and vertical are respectively arranged on a single layer or two layers

$\beta$  depends on the type of constraint end (hinge-hinge  $\beta=1$ )

$i_{min,eff}$  is the radius of gyration of the minimum effective area (only concrete cast in situ)

$$i = \sqrt{J/A}$$

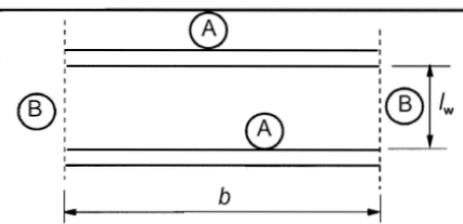
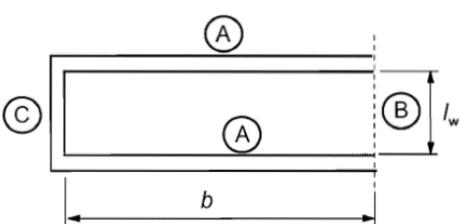
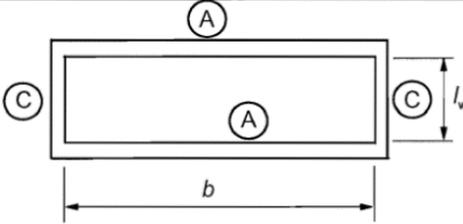
$J$  = inertia of the section

$A$  = area of the section

To neglect the second order effects (section 5.8.3.1 EC2) we must have:

$$\lambda \leq \lambda_{lim}$$

In addition to the previous calculation, is possible to evaluate the height of buckling with the procedure laid down in Eurocode 2 (12.6.5.1) considering therefore the contribution of the different boundary condition.

Lateral restraint	Sketch	Expression	Factor $\beta$																		
along two edges			$\beta = 1,0$ for any ratio of $l_w/b$																		
Along three edges		$\beta = \frac{1}{1 + \left(\frac{l_w}{3b}\right)^2}$	<table border="1"> <thead> <tr> <th><math>b/l_w</math></th> <th><math>\beta</math></th> </tr> </thead> <tbody> <tr><td>0,2</td><td>0,26</td></tr> <tr><td>0,4</td><td>0,59</td></tr> <tr><td>0,6</td><td>0,76</td></tr> <tr><td>0,8</td><td>0,85</td></tr> <tr><td>1,0</td><td>0,90</td></tr> <tr><td>1,5</td><td>0,95</td></tr> <tr><td>2,0</td><td>0,97</td></tr> <tr><td>5,0</td><td>1,00</td></tr> </tbody> </table>	$b/l_w$	$\beta$	0,2	0,26	0,4	0,59	0,6	0,76	0,8	0,85	1,0	0,90	1,5	0,95	2,0	0,97	5,0	1,00
$b/l_w$	$\beta$																				
0,2	0,26																				
0,4	0,59																				
0,6	0,76																				
0,8	0,85																				
1,0	0,90																				
1,5	0,95																				
2,0	0,97																				
5,0	1,00																				
Along four edges		<p>If <math>b \geq l_w</math></p> $\beta = \frac{1}{1 + \left(\frac{l_w}{b}\right)^2}$ <p>If <math>b &lt; l_w</math></p> $\beta = \frac{b}{2l_w}$	<table border="1"> <thead> <tr> <th><math>b/l_w</math></th> <th><math>\beta</math></th> </tr> </thead> <tbody> <tr><td>0,2</td><td>0,10</td></tr> <tr><td>0,4</td><td>0,20</td></tr> <tr><td>0,6</td><td>0,30</td></tr> <tr><td>0,8</td><td>0,40</td></tr> <tr><td>1,0</td><td>0,50</td></tr> <tr><td>1,5</td><td>0,69</td></tr> <tr><td>2,0</td><td>0,80</td></tr> <tr><td>5,0</td><td>0,96</td></tr> </tbody> </table>	$b/l_w$	$\beta$	0,2	0,10	0,4	0,20	0,6	0,30	0,8	0,40	1,0	0,50	1,5	0,69	2,0	0,80	5,0	0,96
$b/l_w$	$\beta$																				
0,2	0,10																				
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0,8	0,40																				
1,0	0,50																				
1,5	0,69																				
2,0	0,80																				
5,0	0,96																				

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

Fig. 4.3 – Prospect 12.1 of EC2

### EXAMPLE - Slenderness

In this example we will calculate the slenderness of a wall made with PSM120 panel.

#### GEOMETRICAL AND MECHANICAL CHARACTERISTICS

Length of the wall ( $b$ ) = 250 cm

Height of the wall ( $H$ ) = 290 cm

Thickness of the shotcrete ( $t_{SHOT}$ ) = 7 cm (is the sum of the 2 layers of 3,5 cm)

Thickness of the EPS ( $t_{EPS}$ ) = 12 cm

Total thickness of the wall ( $h$ ) = 19 cm

Coefficient for lateral constraints ( $k$ ) = 0,7

Lateral constraints = along 3 edges

Characteristic cube strength of concrete ( $f_{ck,CUBE}$ ) = 30 MPa

Characteristic cylinder strength of concrete ( $f_{ck}$ ) =  $f_{ck,CUBE} * 0,83 = 30 \text{ MPa} * 0,83 = 24,9 \text{ MPa}$

Coefficient for long term effect ( $\alpha_{cc}$ ) = 0,85 (from the Italian rules NTC08, according to EC2)

Design compressive strength ( $f_{cd}$ ) =  $\alpha_{cc} f_{ck} / \gamma_c = 0,85 * 24,9 / 1,5 = 14,11 \text{ MPa} = 141,1 \text{ daN/cm}^2$



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## LOAD ANALYSIS

- Floor

Structural weight = 215 daN/m<sup>2</sup>

Variable load = 200 daN/m<sup>2</sup>

Permanent load = 170 daN/m<sup>2</sup>

Influence length = 4,5 m

TOTAL ULS FLOOR LOAD = ((215+170) daN/m<sup>2</sup> \* 1,35 + 200 daN/m<sup>2</sup> \* 1,5) \* 4,5 m = 3689 daN/m

- Wall

Structural weight = 155 daN/m<sup>2</sup>

TOTAL ULS WALL LOAD = 155 daN/m<sup>2</sup> \* 1,35 \* 2,9 m = 607 daN/m

- ULS LOAD FOR THE BUILDING

Number of storeys = 2

TOTAL ULS LOAD FOR THE BUILDING = 3689 daN/m \* 2 + 607 \* 1 = 7985 daN/m



## SLENDERNESS CALCULATION

Effective length ( $l_w$ ) =  $H * k * 1 = 290 \text{ cm} * 0,7 * 1 = 203 \text{ cm}$

$$\beta = \frac{1}{1 + \left(\frac{l_w}{3b}\right)^2} = \frac{1}{1 + \left(\frac{203}{3 * 250}\right)^2} = 0,9317$$

$$l_o = \beta * l_w = 0,9317 * 203 = 189,1 \text{ cm}$$

Cross sectional area of shotcrete ( $A_c$ ) =  $250 \text{ cm} * 7 \text{ cm} = 1750 \text{ cm}^2$

Resistant compressive axial force ( $NR_d$ ) =  $f_{cd} * A_c = 14,11 \text{ MPa} * 1750 \text{ cm}^2 = 246925 \text{ daN}$

Axial design force ( $NE_d$ ) =  $7985 \text{ daN/m} * 2,5 \text{ m} = 19961 \text{ daN}$

$$r_m = 0,7$$

$$C = 1,7 - r_m = 1$$

$$n = \frac{NE_d}{NR_d} = \frac{19961}{246925} = 0,08083$$

$$\lambda_{lim} = 15,4 * \frac{C}{\sqrt{n}} = 15,4 * \frac{1}{\sqrt{0,08083}} = 54,16$$

$$J = \frac{b * (h^3 - t_{eps}^3)}{12} = \frac{250 * (19^3 - 12^3)}{12} = 106896 \text{ cm}^4$$

Coefficient of reduction of inertia ( $c$ ) = 0,2

Effective inertia of the wall  $J_{red} = J * c = 106896 \text{ cm}^4 * 0,2 = 21379 \text{ cm}^4$

$$i_{min,eff} = \sqrt{\frac{J_{red}}{A_c}} = \sqrt{\frac{21379 \text{ cm}^4}{1750 \text{ cm}^2}} = 3,495 \text{ cm}$$

$$\lambda = \frac{l_o}{i_{min,eff}} = \frac{189,1 \text{ cm}}{3,495 \text{ cm}} = 54,11$$

$$\lambda \leq \lambda_{lim} \quad \text{VERIFIED}$$

N.B.: Refer to the "Slenderness" spreadsheet to made the slenderness verification.

#### 4.5. SHEAR

To calculate the **shear** capacity in the plane of the panel, you can use the provided formulas given in paragraph 6.2.2 of EC2, which are outlined below.

For elements with a vertical shear reinforcement, the design shear resistance ( $V_{Rd}$ ) is the less of the following values:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta$$

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd} / (\cot\theta + \tan\theta)$$

where:

$A_{sw}$  cross-sectional area of one stirrup;

$s$  stirrup distance;

$f_{ywd}$  yield stress of stirrups;

$v_1$  is a strength reduction factor for concrete cracked in shear

$\alpha_{cw}$  is a coefficient taking account of the state of the stress in the compression chord;

$z$  inner lever arm  $\cong 0,9d$ ;

$b_w$  is the minimum width between tension and compression chords;

To have the recommended values of  $v_1$  and  $\alpha_{cw}$  refer to the paragraph 6.2.3 of the EC2.

The base of every storey of the wall should be verified in **sliding shear**. The design value of the resisting shear against horizontal sliding along the base section in a storey is given by Eurocode 8 as:

$$V_{Rd,s} = V_{fd} + V_{dd} + V_{id}$$

That is the sum of:

1. The friction resistance of the compression zone:

$$V_{fd} = \min \left( \mu_f \left[ \left( \sum A_{sj} f_{yd} + N_{Ed} \right) \xi + \frac{M_{Ed}}{z} \right]; 0.5 \cdot \eta \cdot \xi \cdot f_{cd} \cdot b_{wo} \cdot l_w \right)$$

with:

- $\mu_f$  friction coefficient, equal to 0.7 for rough interfaces or equal to 0.6 for smooth ones;
- $\sum A_{sj}$  is the total area of web vertical bars and of any additional vertical bars placed in boundary elements against shear sliding;
- $M_{Ed}, N_{Ed}$  values from the analysis for the seismic design situation;
- $\xi$  normalized neutral axis depth

2. A design value of the dowel resistance:

$$V_{dd} = \sum A_{sj} \cdot \min \left( 1.3 \sqrt{f_{cd} f_{yd}}; 0.25 f_{yd} \right)$$

For concrete class above C20/25, the second term governs; it reflects yielding of the dowel in pure shear without axial force, with a safety margin of about 2.3.

3. The horizontal projection of the design resistance of any inclined bars, with a total area  $\sum A_{si}$ , placed at an angle  $\pm\varphi$  to the base of the wall:

$$V_{id} = \sum A_{si} f_{yd} \cos\varphi$$

### EXAMPLE - Shear

In this example we will calculate the shear resistance for a PSM80 panel with the dimensions shown below and considering an axial force of  $N_{Ed} = 100 \text{ kN}$  and a  $M_{Ed} = 0 \text{ kNm}$ :

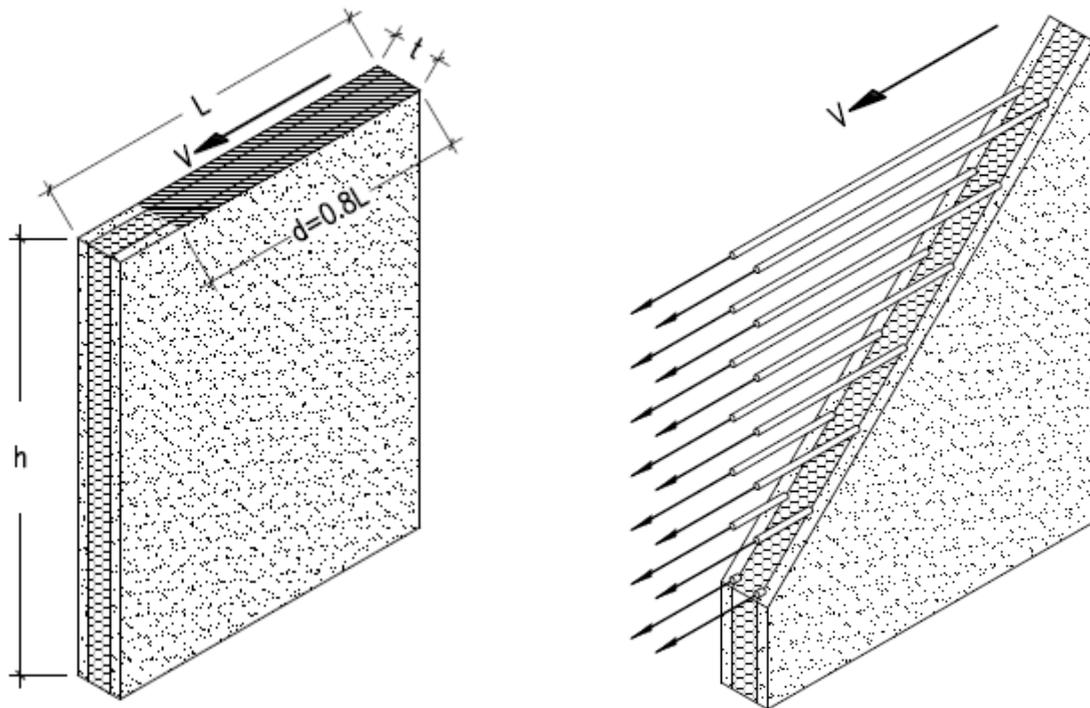


Fig. 4.4 –Shear resistance PSM panel

$$L = 1,00 \text{ m}$$

$$d = 0,8 L = 0,80 \text{ m}$$

$$t = 0,15 \text{ m} \quad \text{total thickness of PSM panel (shotcrete + EPS)}$$

$$t_c = 0,07 \text{ m} \quad \text{thickness of the shotcrete}$$



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The mechanical characteristics of the **steel wire mesh** are the following:

Vertical wires:	$\varnothing 2,5$ mm every 77,5 mm
Horizontal wires:	$\varnothing 2,5$ mm every 112,5 mm
Yield strength of reinforcement:	$f_{yk} = 600$ N/mm <sup>2</sup>
Design yield strength of reinforcement:	$f_{yd} = f_{yk} / \gamma_s = 600 / 1,15 = 521,74$ N/mm <sup>2</sup>

The mechanical characteristics of the **shotcrete** are the following:

Characteristic compressive cubic strength of shotcrete at 28 days:  $R_{ck} = 25$  N/mm<sup>2</sup>

Characteristic compressive cylinder strength of shotcrete at 28 days:  $f_{ck} = 0,83 R_{ck} = 20,75$  N/mm<sup>2</sup>

Design value of concrete compressive strength:  $f_{cd} = 0,85 f_{ck} / \gamma_c = 0,85 \cdot 20,75 / 1,5 = 11,76$  N/mm<sup>2</sup>



- Calculation of the PSM80 shear resistance

We calculate the values of  $V_{Rd,s}$  and  $V_{Rd,max}$  and we take the lower of the two values.

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \qquad V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd} / (\cot\theta + \tan\theta)$$

$$V_{Rd,s} = \frac{9,817}{112,5} \cdot 800 \cdot 521,74 \cdot 2,5 = 91 \text{ kN}$$

with:

$$A_{sw} = 2 \pi r^2 = 2 \pi (1,25)^2 = 9,817 \text{ mm}^2$$

$$s = 112,5 \text{ mm}$$

$$z = d = 0,8 L = 0,8 \cdot 1000 \text{ mm} = 800 \text{ mm}$$

$$\cot\theta = 2,5 \qquad \text{with: } 1 \leq \cot\theta \leq 2,5.$$

For the calculation of  $\cot\theta$ , we take the value for which we have equal values of  $V_{Rd,s}$  and  $V_{Rd,max}$ .

If is impossible to have equal values, look at the range of the possible values of  $\cot\theta$ .

$$V_{Rd,max} = 1,1214 \cdot 70 \cdot 800 \cdot 0,6 \cdot 11,76 / (2,5 + 0,4) = 152 \text{ kN}$$

with:

$$\sigma_{cp} = \frac{N_{Ed}}{A} = \frac{100000 \text{ N}}{70 \text{ mm} \cdot 1000 \text{ mm}} = 1,428 \text{ N/mm}^2 \qquad \text{which is } 0 \leq \sigma_{cp} \leq 0,25 f_{cd}.$$

$$\alpha_{cw} = \left(1 + \frac{\sigma_{cp}}{f_{cd}}\right) = \left(1 + \frac{1,428}{11,76}\right) = 1,1214$$

$$\tan\theta = \frac{1}{\cot\theta} = \frac{1}{2,5} = 0,4$$

Therefore, the value of  $V_{Rd}$  is **91 kN**.

Then, is possible to proceed with the calculation of the **sliding shear**.

To calculate the sliding shear we take into account the areas of the vertical bars of the web and of additional bars arranged in the boundary elements specifically for resistance against sliding.

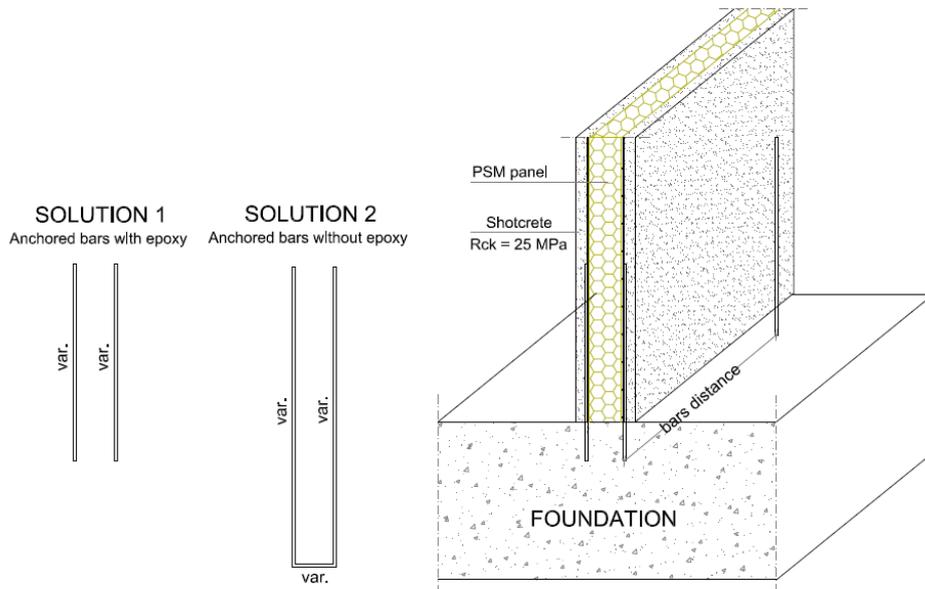


Fig. 4.5 –Anchoring bars

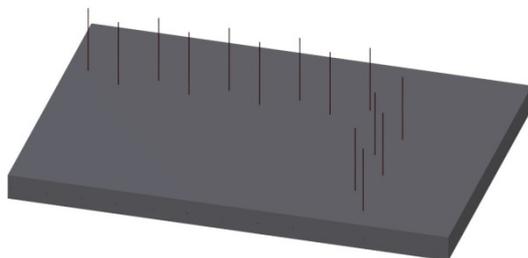


Fig. 4.6 –Positioning of anchoring bars

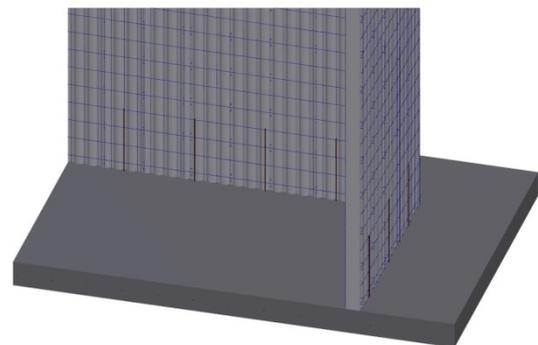


Fig. 4.7 –Positioning of EMMEDUE panels

In our example we consider anchoring bars with the following characteristics:

Bars distance (x) = 40 cm

Diameter bars = 6 mm

The design values of the shear resistance is:

$$V_{Rd,s} = V_{fd} + V_{dd} + V_{id}$$

First, we calculate the values of  $V_{fd,1}$  and  $V_{fd,2}$  and we take the lower of the two values.

$$V_{fd,1} = \mu_f \left[ \left( \sum A_{sj} f_{yd} + N_{Ed} \right) \xi + \frac{M_{Ed}}{z} \right] = 0,6 \cdot \left[ \left( 141,37 \text{ mm}^2 \cdot 391,30 \frac{\text{N}}{\text{mm}^2} + 100000 \text{ N} \right) \right]$$

$$= 93,19 \text{ kN}$$

$$V_{fd,2} = 0,5 \cdot \eta \cdot \xi \cdot f_{cd} \cdot b_{wo} \cdot l_w = 0,5 \cdot 0,5502 \cdot 1 \cdot 11,76 \frac{\text{N}}{\text{mm}^2} \cdot 70 \text{ mm} \cdot 1000 \text{ mm} = 226 \text{ kN}$$

with:

$$\sum A_{sj} = \pi r^2 \cdot \frac{L}{x} \cdot 2 = \pi 3^2 \cdot \frac{1000}{40} \cdot 2 = 141,3 \text{ mm}^2$$

$$\eta = 0,6 \cdot \left( 1 - \frac{f_{ck}(\text{MPa})}{250} \right) = 0,6 \cdot \left( 1 - \frac{20,75 \text{ MPa}}{250} \right) = 0,5502$$

$\xi = 1$  because the section is all compressed

Therefore, the value of  $V_{fd}$  is

$$V_{fd} = 93,19 \text{ kN}$$

Then, we calculate the values of  $V_{dd,1}$  and  $V_{dd,2}$  and we take the lower of the two values.

$$V_{dd,1} = \sum A_{sj} \cdot 1,3 \cdot \sqrt{f_{cd} f_{yd}} = 141,37 \text{ mm}^2 \cdot 1,3 \cdot \sqrt{11,76 \frac{\text{N}}{\text{mm}^2} \cdot 391,30 \frac{\text{N}}{\text{mm}^2}} = 12,46 \text{ kN}$$

$$V_{dd,2} = \sum A_{sj} \cdot 0,25 f_{yd} = 141,37 \text{ mm}^2 \cdot 0,25 \cdot 391,30 \frac{\text{N}}{\text{mm}^2} = 13,83 \text{ kN}$$

Therefore, the value of  $V_{fd}$  is

$$V_{dd} = 12,46 \text{ kN}$$

The value of  $V_{id}$  is equal to 0 because we don't have inclined bars.

Therefore, the value of  $V_{fd}$  is

$$V_{Rd,SLS} = V_{fd} + V_{dd} + V_{id} = 93,19 + 12,46 + 0 = 105,65 \text{ kN}$$



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