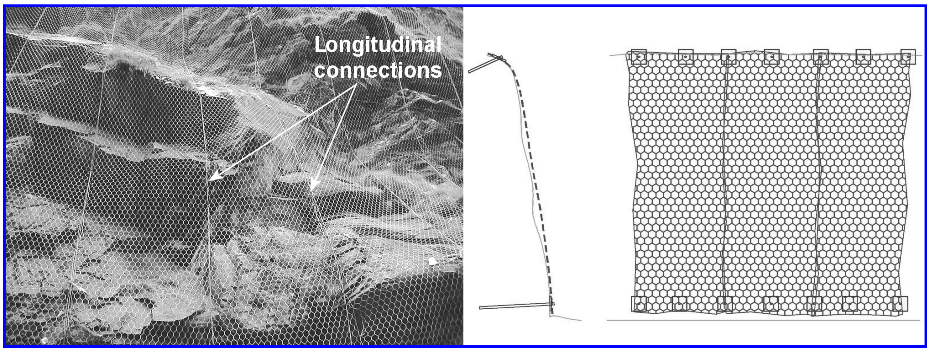
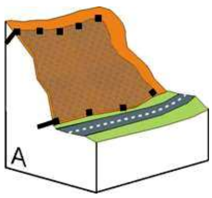
OPERE DI PROTEZIONE CONTRO LA CADUTA MASSI: ASPETTI PROGETTUALI

Reti in aderenza

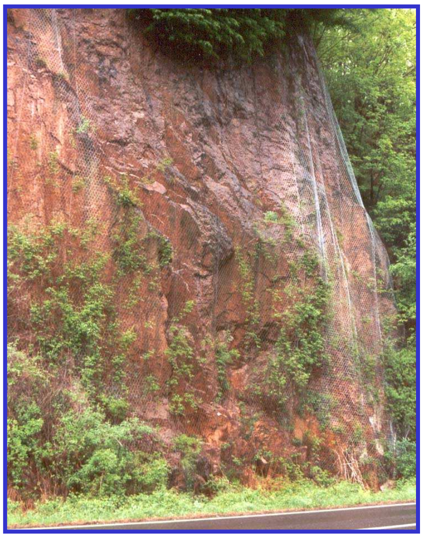
Daniele PEILA

Simple mesh drapery system

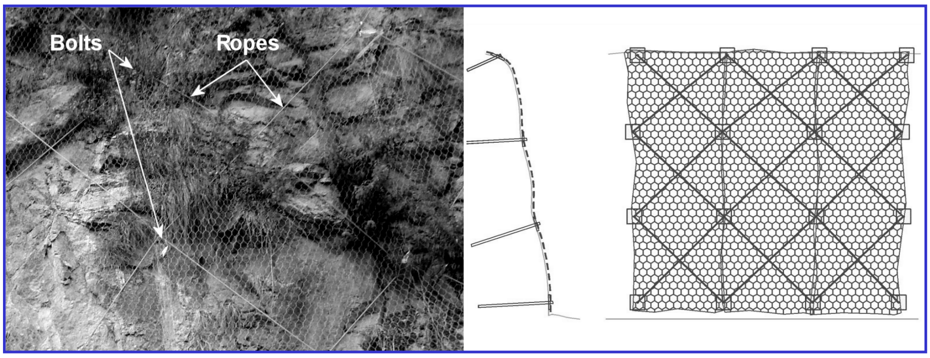


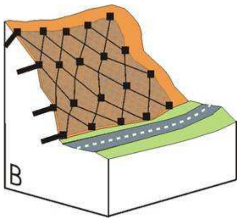


Simple mesh drapery system

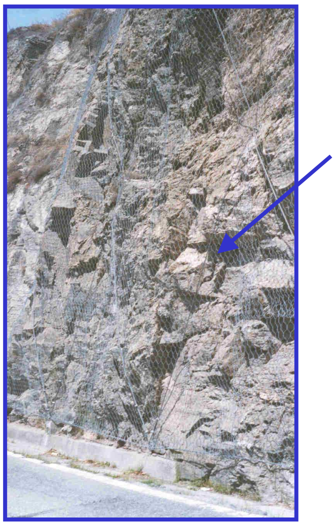


Fixed drapery sistem



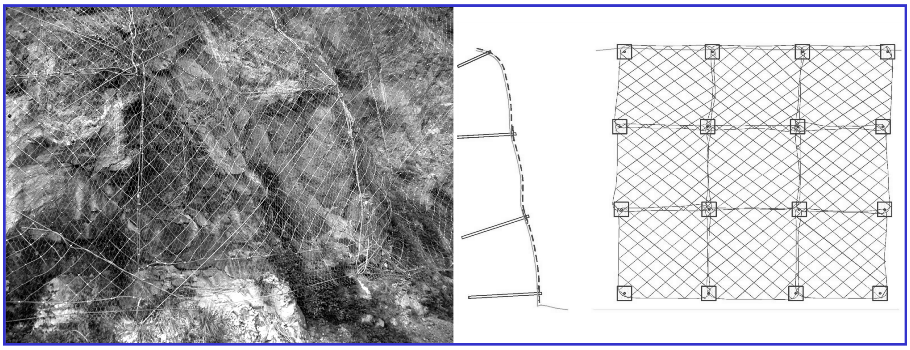


Fixed drapery sistem



Fixed drapery sistem

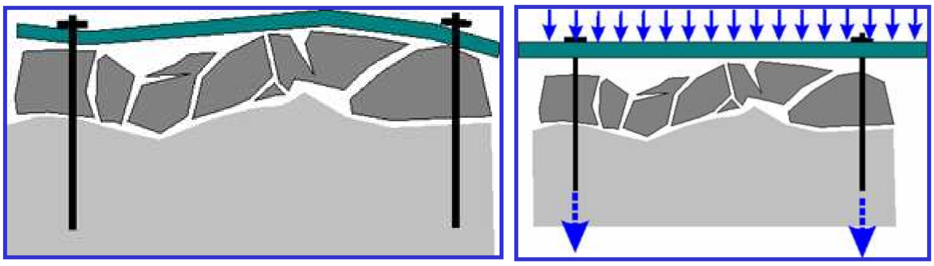
Cable net or ring panels



Cable net or ring panels

Cable net panels

Which is the real behaviour of a fixed mesh on a rock slope ?

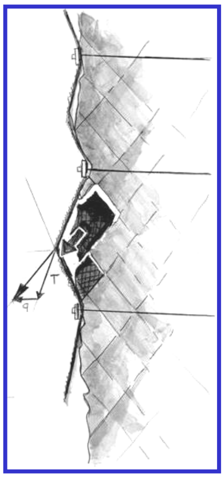


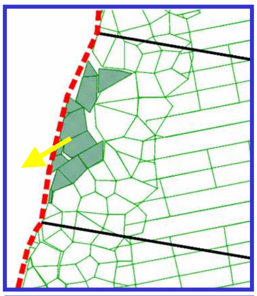
does the mesh act only when is loaded by a moving rock block by transferring the loads to the anchors ?

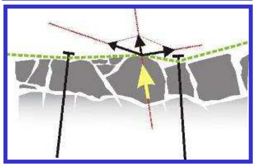
or

does the mesh apply a confining force on the rock face thus contrasting the moving rock block ?

Which is the real behaviour of a fixed mesh on a rock slope ?







The anchorages have an important role in the reinforcement of the rock mass
The mesh contains the blocks moving between the confining bolts

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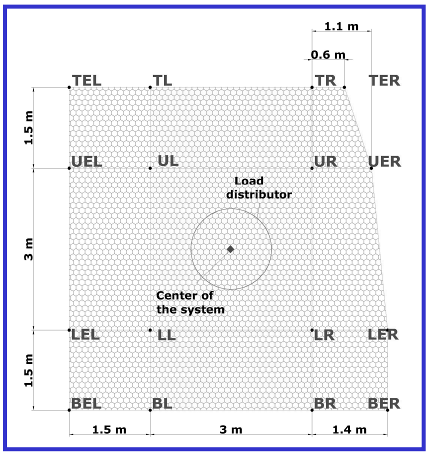
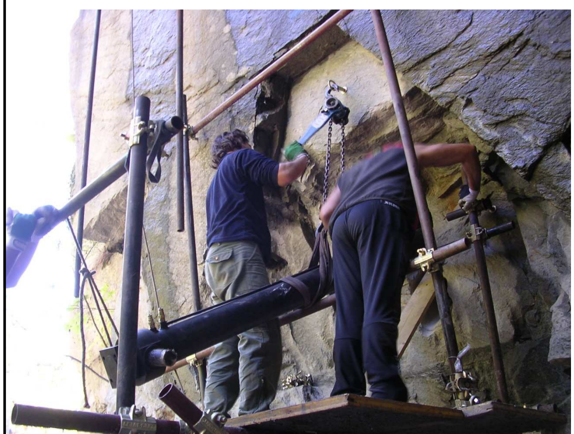


Front view of the full-scale test field and schematic drawing.

The rock mass irregular geometry imposed to reduce the size of the right upper corner of the lateral net. It has been verified that this change in geometry did not significantly altered the boundary conditions of the tested net panel.

Construction of the test site



Construction of the test site (installation of the jack)

Global view of the test site

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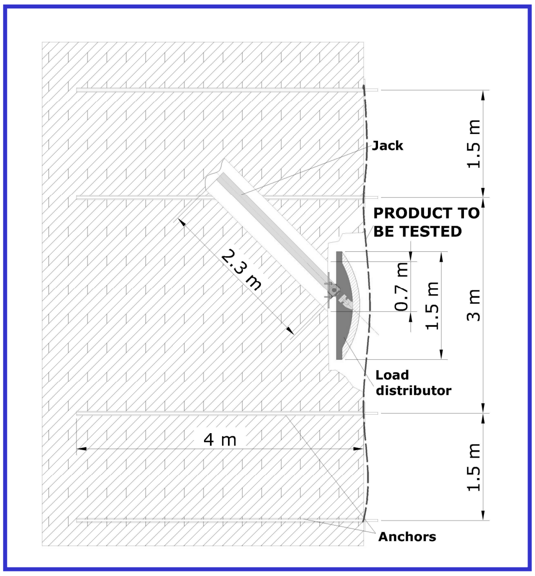
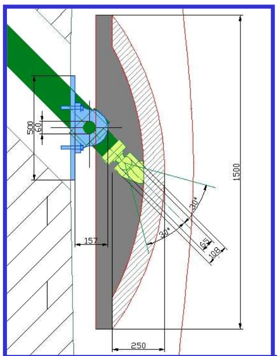
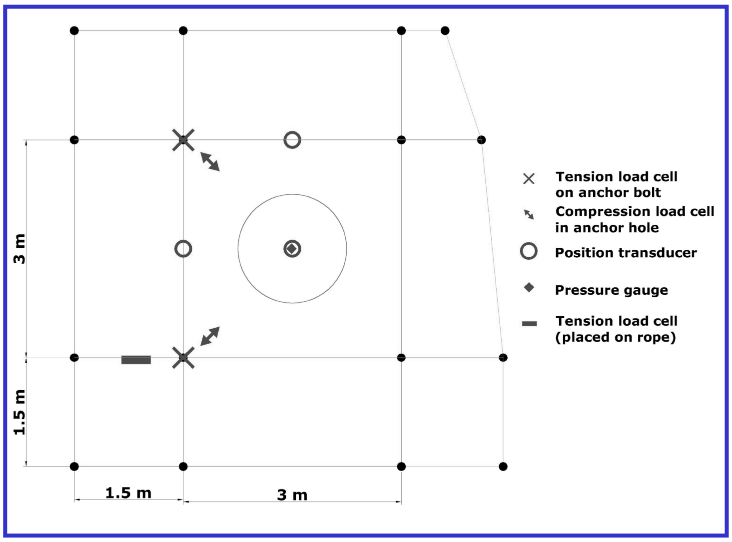
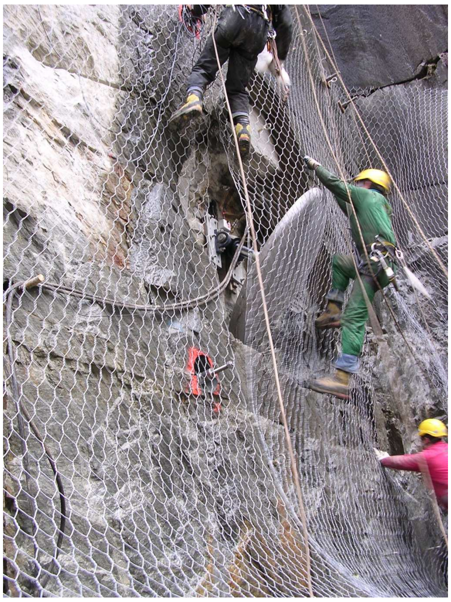






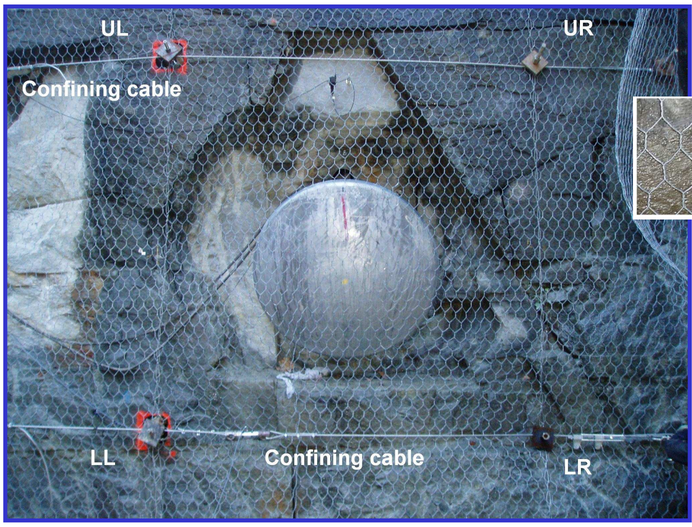
Test devices installed on the product to be tested



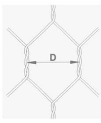
The installation is done exactly as on the job sites

Test n.	Type of mesh	Type of drapery system installed
1	HEA panel (3*3m panel), 10mm diameter cable. Mesh size 400mm*400mm, without edge cable	Cable net panels Anchorage bolt inserted in the edge mesh of the net panel
2	HEA panel (3*3m panel), 10mm diameter cable. Mesh size 400mm*400mm, without edge cable	Cable net panels The panel is the one already deformed after test 1 Anchorage bolt inserted in the edge mesh of the net panel
3	HEA panel (3*3m panel), 10mm diameter cable, mesh size 300mm*300mm, without edge cable	Cable net panels. Anchorage bolt inserted in the edge mesh of the net panel
4	HEA panel (3*3m panel), 10mm diameter cable, mesh size 300mm*300mm, without edge cable.	Cable net panels. Anchorage bolt inserted in the edge mesh of the net panel
5	HEA panel (3*3m panel), 10mm diameter cable. Mesh size 300mm*300mm, without edge cable	Cable net panels. Anchorage bolt inserted in the edge mesh of the net panel
6	HEA panel (3*3m panel), 10mm diameter cable. Mesh size 300mm*300mm, without edge cable	Cable net panels. Anchorage bolt inserted in the edge mesh of the net panel
7	double twisted wire mesh	Fixed drapery system with crossed reinforcing cables connected to the anchors (pattern 3m *3m) with a square 150mm*150mm plate
8	double twisted wire mesh	Fixed drapery system with sub-horizontal reinforcing cables connected to the anchors (pattern 3m *3m) with a square 150mm*150mm plate
9	double twisted wire mesh	Simple mesh drapery system

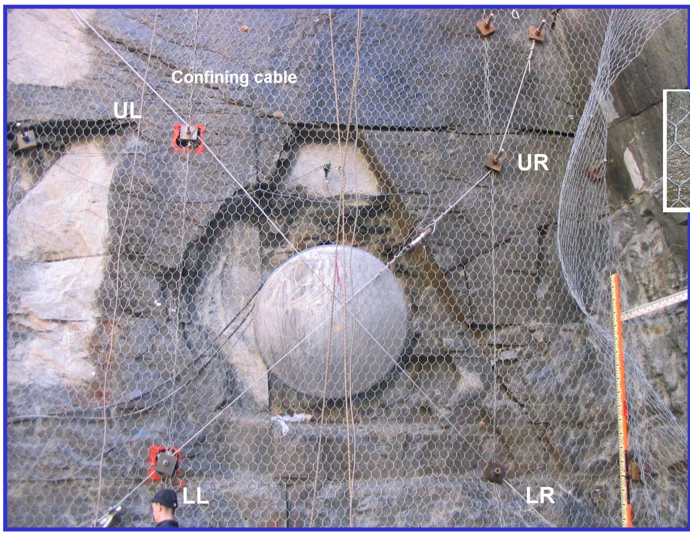
+ 1 test on cable net with clips



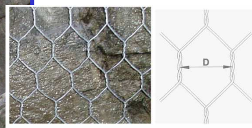
Test 8



Double twisted wire mesh with longitudinal cables



Test 9

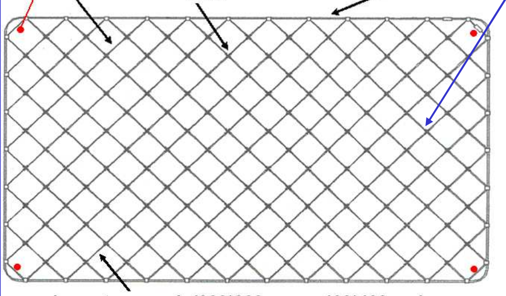


Double twisted wire mesh with cross cables

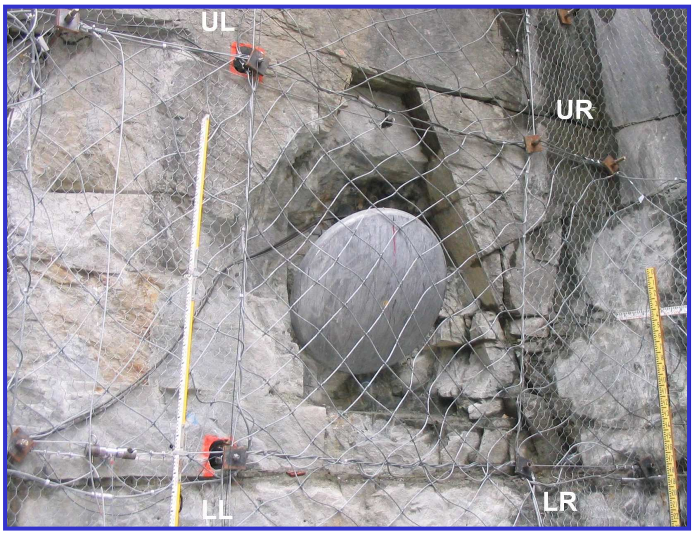
example of anchorage

in the hole the dynamometer is inserted

position of the anchorage in the mesh



Funi d'acciaio		
Fune di orditura		
Fune a trefoli (UNI EN 10264-2— UNI ISO 2408)	Diametro Ø(mm)	
	8	10 - 12
	6x7 WS	6x19 WS
	Classe B Classe A	
Resistenza nominale a rottura (N / mm^2)	1770	
Fune perimetrale (opzionale)		
Diametro Ø (mm)	10 - 12- 14 - 16	
Fune a trefoli tipo 6x19 WS (UNI EN 10264-2— UNI ISO 2408)	Classe B Classe A	
Resistenza nominale a rottura (N / mm^2)	1770	

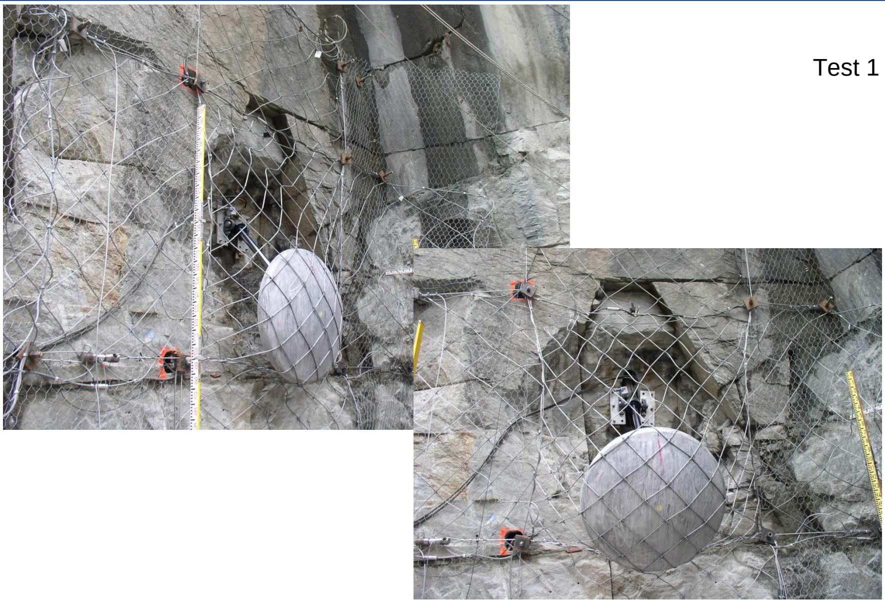


Tests 1-6

HEA panel installed on the test site

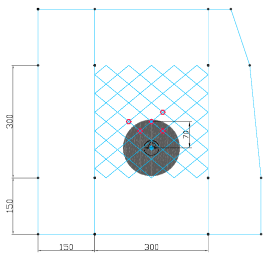
Test 8

Test 7

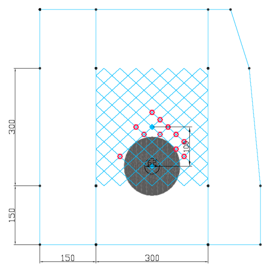


Test 1

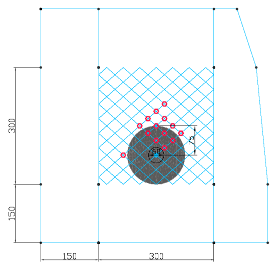
Position of the slipped knot after tests n. 1, 3 and 4.



Test 1



Test 3



Test 4

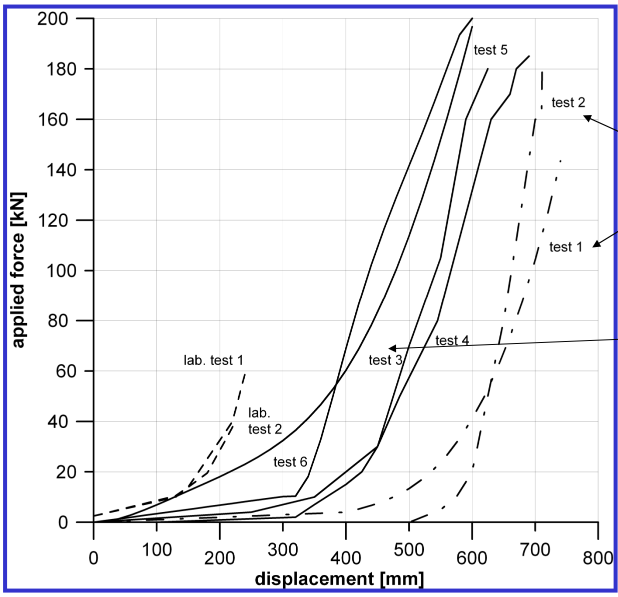
Example of the results of test carried out on a rope panel with clips

(diameter of the rope 8mm)

		Anchorage UL		Anchorage LL		Confining cables
Test n.	**Max. applied force [kN]**	**Max axial force [kN]**	**Max. tangential force [kN]**	**Max axial force [kN]**	**Max tangential force [kN]**	**Force [kN]**
1	143	Dynamometer broken (due to large displacement)	25	60	10	-
2	180	Dynamometers non installed		65-70	5-7	-
3	180	Dynamometer broken (due to large displacement)	37	70	10	-
4	185	70	30	Dynamometers non installed		
5	200	Dynamometers not installed				-
6	196	Dynamometers not installed				-
7	15	2-3	2-2.5	6-7	0	20 (pretension 3kN)
8	38	5	12	7	0	40 (pretension 3kN) Lower cable
9	14	5.5	0	1	0	0

Forces on anchorages measured during tests n. 1, 3 and 4.

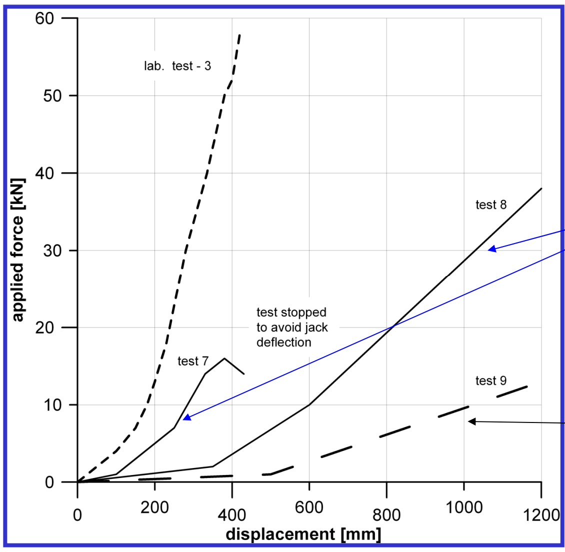
HEA panel behaviour during the test



Results of full scale tests on HEA panels as a system

mesh 400mm*400mm

mesh 300mm*300mm

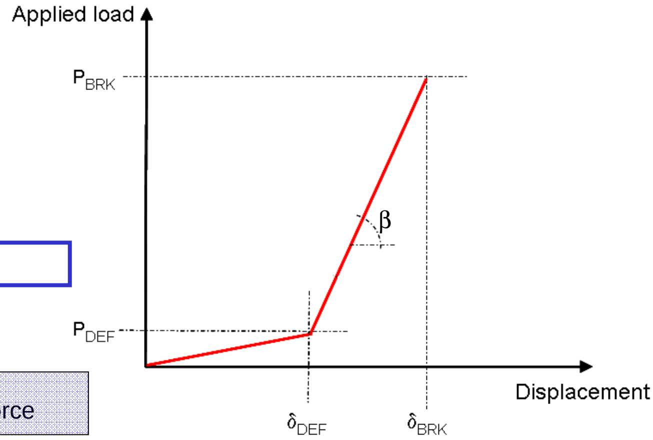


Results of the full scale tests on double twist mesh

Fixed drapery

Simple drapery

Behaviour of a mesh as installed on a slope



Is it important ?

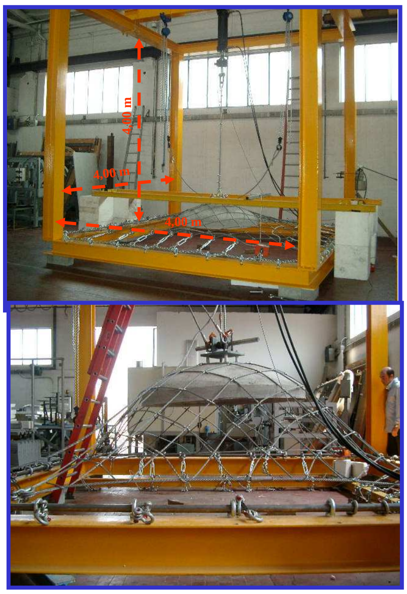
P_{BRK} = breaking force

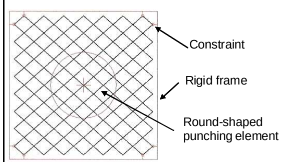
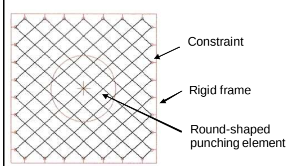
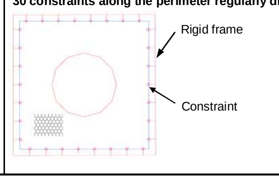
P_{DEF} = charge level at δ_{DEF}

$\beta = (P_{BRK}-P_{DEF})/((\delta_{BRK}-\delta_{DEF})$ = mesh system stiffness

δ_{BRK} = breaking displacement

δ_{DEF} = deformation at low carge

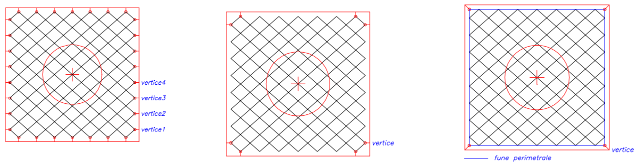


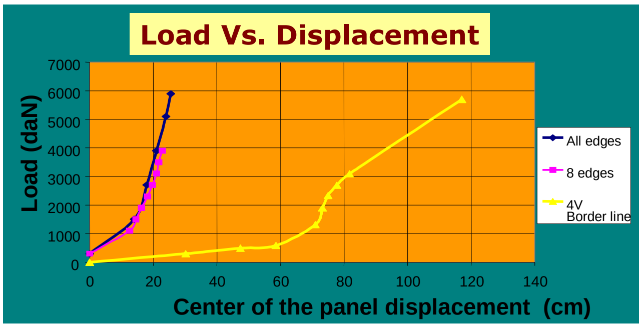
Lab. Test	Type of mesh	Test geometry and used constraints
1	HEA panel (3*3m size) 10mm diameter cable elementary mesh 300*300mm	**8 constraints on the panel edges** 
2	HEA panel (3*3m size) 10mm diameter cable elementary mesh 300*300mm	**All the mesh edges are connected with the rigid frame** 
3	Double twisted wire mesh (3*3m size)	**30 constraints along the perimeter regularly distributed** 

Laboratory test carried out by CNR-Maccaferri (Italy)

Results of the laboratory full scale test on HEA panel

Comparison of different boundary conditions: all edges fixed ; 8 edges fixed, 4 corners fixed





HEA Panel Φ 10 Mesh 300

MESH DESIGN – DRAPERY MESHES

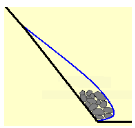
Actions of the system: weight of the mesh, weight of the debris and action of the snow

the **weight of the mesh**, in linear meter (W_m), is expressed by:

$$W_m = \gamma_m (H_o/\text{sen } \beta);$$

the **force acting in the direction of the slope** (S_{Wm}) is expressed by:

$$S_{Wm} = W_m \cdot \text{sen}\beta = \gamma_m \cdot (H_o/\sin \beta) \cdot \text{sen } \beta$$



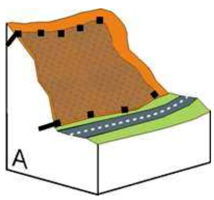
the **weight of the debris** at the base of the slope is expressed by:

$$W_d = V_d \cdot \gamma d = 0.5 \cdot \gamma d \cdot h_d \cdot [(h_d / \tan (\Phi_d)) - (h_d / \tan (\beta))]$$

the **force of the debris** along the average direction of the slope is expressed by:

$$S_{Wd} = W_d \cdot \sin \beta$$

H_o: height of the slope;



γ_m: weight per unit area mesh (kN/m²);
β: average slope angle;
γ_d: weight per unit of volume of debris (kN/m³);
V_d: volume of debris accumulated (m³/m);
h_d: respectively, the average height of the debris;
Φ_d accumulation angle of the debris.

The strength of the **snow load** acting on the direction of the slope:

- Snow load on mesh at temperatures below 0 °C:
$S_S = \gamma_s \cdot t_s \cdot L \cdot \text{sen}\beta - L \cdot c$;

- Snow load on mesh at temperatures above 0 °C:
$S_S = \gamma_s \cdot t_s \cdot L \cdot \text{sen}\beta$

γ_s mass density of the snow that may be assumed equal to about 2.70 kg/m^3
t_s thickness of the snowpack
L length of the slope on which snow has accumulated, in the first approximation can be considered equal to the length of the slope (l / sin β)
c cohesion snow, which depends on the density of the snow cover and temperature (for incompact snow can take values very close to zero).

Finally, the friction force directed in the direction of the slope, it is calculated considering only the weight of the debris on the mesh and is expressed by:

$$S_A = W_m \cdot \cos\beta \cdot \tan\delta + W_d \cdot \cos\beta \cdot \tan\delta$$

δ: angle of friction between the mesh and slope that should be taken as equal to 45° for very rough slopes and to 30° for slopes smooth and r egular.

VERIFICATION OF SYSTEM COMPONENTS

The verification should be performed to the ultimate limit state (ULS), Eurocode 7

Failure mechanisms:
- loosening or breakage of the anchor head;
- breaking of the rope attached to the superior longitudinal anchors;
- breaking strength of the mesh.

The actions per linear meter of mesh are:

$$\mathbf{T = \gamma_{A2} \cdot S_{Wm} + \gamma_{A3} \cdot (S_{Wd} + S_S) - \gamma_{A1} \cdot S_{Av}}$$

where: γ_{A1}= 1.0 multiplier for permanent loads pro-security;
γ_{A2}= 1.35 multiplier for permanent loads unfavorable safety;
γ_{A3}= 1.5 multiplier for varying loads unfavorable security

Verification of the breaking strength of the mesh

The traction test for the mesh:

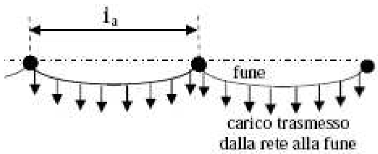
$$(T_m/\gamma_R) \geq T$$

T_m: tensile strength of the mesh per metro;
γ_R = 1.2 coefficient of safety

Verification the anchorages and head longitudinal rope

The verification of the anchors and the longitudinal head rope is made applying the action to anchorages meter (T), taking into account the interaction and the deformed of the rope under the action of the load, and the distance between the anchor (i_a)

Schematic indication of the geometry of the deformed head rope and anchors, the scheme adopted for the sizing and selection of the nails head.



The distance between the anchors is determined in an iterative way

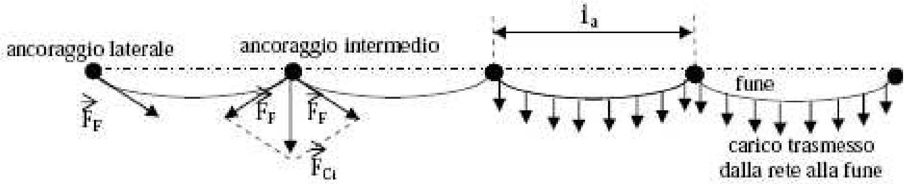
Usually it takes a distance of 3m and if the stresses induced on the structure are too high this value is improved.

The calculation of the rope head is made on the assumption that the piece of rope between two nails are deformed under the action of the load assuming a catenary shape and then the nail is stressed by a force of absolute value equal to F_F.

This force must be less than the tensile strength of the rope:

$$(R_F / \gamma_R) \geq F_F$$

where R_F: tensile strength of the rope and γ_R = 1.2 safety factor.



The anchoring nails have to endure actions induced by the mesh, they must be verified on the basis of the following failure mechanisms (ULS) :

1. shear failure;
2. breaking strain;
3. loosening of the portion anchored in the good rock (excluding the top 0.5).

Checks 1 and 2 : $R_{bar} / \gamma_R > R$

R_{bar}: resistance at break or cut of the material constituting the anchorage, γ_R the reduction coefficient of the bar resistance, equal to 1.20

Check 3: $(t_{cementation\text{-}rock} \cdot \phi_{hole} \cdot \pi \cdot L_{anchor}) / \gamma_R \geq R$

$(t_{cementation\text{-}bar} \cdot \phi_{bar} \cdot \pi \cdot L_{anchor}) / \gamma_R \geq R$

$t_{cementation\text{-}bar}$ and $t_{cementation\text{-}rock}$ are respectively the shear strength interface between bar and soil and between the rock and cementation, determined by tests or experimental data in the technical literature (Bustamante and Doix, 1985), while ϕ_{hole} and ϕ_{bar} are respectively the diameter of the hole and the bar of the nail, then assuming that the first anchor 0.5m are not reactive: $L_{anchorage}=(L_{nail}-0.5)$

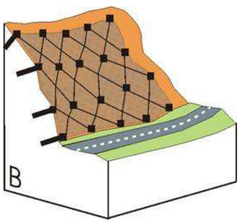
CORTICAL REINFORCEMENTS

The nails and the mesh must be calculated in a separate and interdependent way:

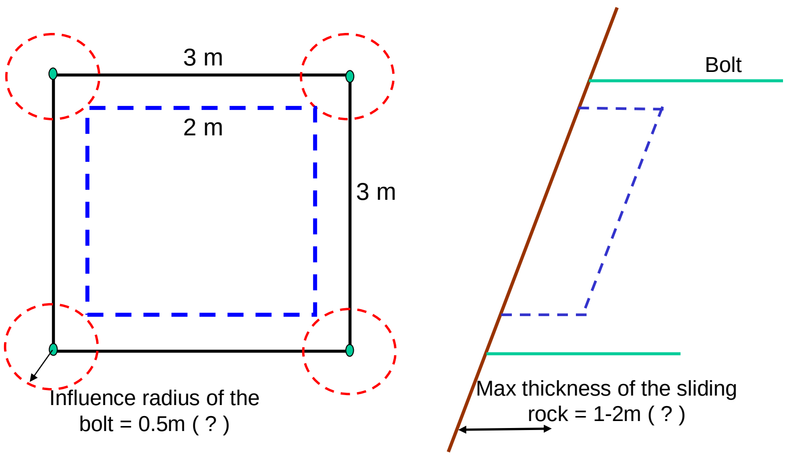
- the nails, taking into account the actions induced by unstable rock volumes fall within their influence area and action by the mesh for which they act as constraints;

- the mesh, holding the rock pieces freed from the nails.

In this approach it is assumed, in a simplified way but in favor of safety, that the single element consolidated from the anchors and rock blocks and the unstable elements located between the top of the nails are both in **equilibrium limit.**

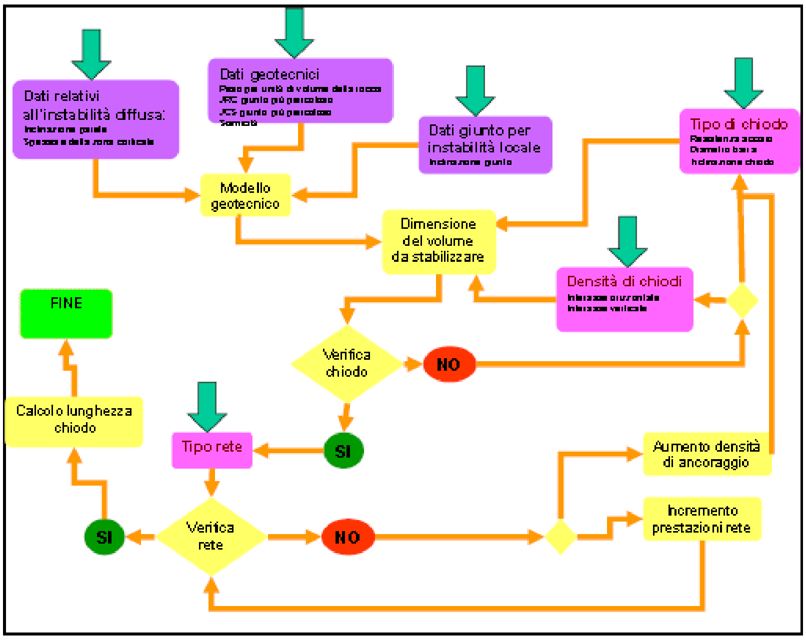


Simplified calculation



The maximum load that should be supported by the mesh is:

$$W = V^* \gamma = 2*2*2 * \gamma = 8*25 = 200 \text{ kN}$$



For the design of the nails, it is assumed, for security, that these support the entire thickness of the cortical part (s) considered to be unstable.

The contribution resistance of the nails can be calculated on the assumption that the considered cortical area is <u>in limit equilibrium conditions</u>

stabilizing forces = destabilizing forces = $W \cdot \text{sen}\beta$

β = inclination of the surface where may occur the slip,
W =weight of the volume to consolidate = $[i_x \cdot i_y \cdot \gamma \cdot s]$

i_x and i_y = horizontal and vertical spacing of the mesh of nail;
s = thickness cortical area of unstable;
γ = weight per unit volume of rock.

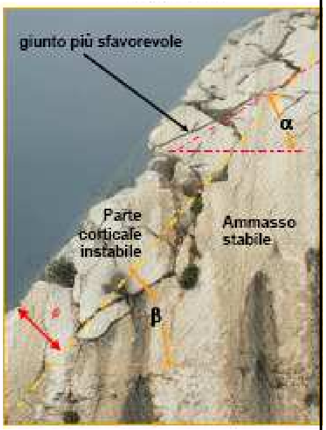


Diagram of a rock face with an indication of the main geometrical parameters

The stabilizing contribution (R) required to the single nail, is calculated by introducing appropriate safety factors for actions for the reactions:

$$[(W \cdot \sin\beta) / \gamma_{rw}] + R \geq (W \cdot \gamma_{DW} \cdot \text{sen}\beta)$$

γ_{DW} multiplier coefficient for permanent unfavorable loads, to safety we suggests that between 1.05 and 1.15,
γ_{RW} multiplier coefficient for permanent loads that can be safely expressed by:

$$\gamma_{RW} = \gamma_{RWs} \cdot \gamma_{RWg} \cdot \gamma_{RWa},$$

γ_{RWs}: coefficient which takes into account the reliability of the value of s, which can be assumed equal to 1.3 if the determination of s was made with geomechanical measurements in site and 1.5 if the evaluation of s is empirical;

γ_{RWg}: coefficient which takes into account the uncertainty in determining the weight per unit volume of loose rock. Can usually be set equal to 1.0, while in some uncertainty cases (flysch or marly rocks) it is suggested to take 1.05;

γ_{RWa} the coefficient that considers the environmental conditions and the degradation of the rock, usually equal to 1.0 - if the rock is very altered, it is suggested to adopt the value 1.05

Based on the previous formulation we can derive the resistance (R) request to the anchors to stabilize the rock slope (before we had to chose the design distance between the nails (i_a)

If the calculated values are too high you must repeat the calculation for smaller stressing distances, however the most commonly used range are from 2mx2m and 4mx4m.

In cases of seismic problem we must added to the active loads the seismic actions.

Determined the reaction (R) that must exercise the nails to stabilize the slope, these must be checked on the basis of the following mechanisms (ULS):

- shear failure at the interface between the portion of the rock unstable and the good rock
- breaking tensile deformation at the interface between the portion of the nail unstable and the good rock
- loosening of the portion anchored in the intact rock

The first and second check require that:
$R_{bar} / \gamma_R > R$

R_{bar}: resistance at break or cut of the anchoring material;
γ_R: reduction of resistance equal to 1.20.

The third check: $L_{anchor} = (L_{nails} - s)$

L_{anchor} must be enough length to ensure the impossibility of slipping of the nail both at the contact among cement - bar, and bar - cementation:
$(t_{cementation-\ rock} \cdot \phi_{hole} \cdot \pi \cdot L_{anchor})/\gamma_R \geq R$;
$(t_{cementation-\ bar} \cdot \phi_{bar} \cdot \pi \cdot L_{anchor})/\gamma_R \geq R$

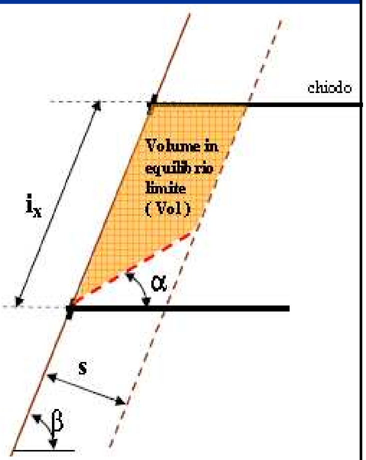


Diagram of the unstable rock volume taken for mesh verification.

We must always remember that the interventions being usually by hand-drilling, the maximum length that can be achieved, for operational reasons, is limited to 3-4m.

Mesh design

For the dimensioning of the mesh we must taking in account **full scale tests**, in fact some types of mesh have a high deformability even modest loads which changes the geometry of force application and facilitates the spread of instability within the slope.

The drapery mesh will be verified only if it responds to the load with a limited deformation, so as doesn't allow the spread of the collapse of into the slope.

It is not possible to verify the mesh only based on their mechanical resistance to traction or assume that the mesh can develop containment pressures against the rockside.

Mesh design

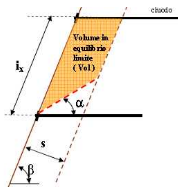
The design of the mesh must be developed to the ultimate limit state (ULS)

The condition for local instability (ie the separation of the rock block between the nails) is that the wedge of rock that is based on more critical joint among all those that have been measured on the rockside, is in equilibrium limit and:

$\beta > \alpha$

where β angle of slope

α angle of the most critical rock joint including in the mesh



Given this situation the maximum size of the sliding block can request a mesh meter, depends on the thickness of the altered band (s) and the spacing between the nails of the mesh.

This hypothesis is cautionary, but being in favor of safety, it is adequate in relation to the many uncertainties related to this problem

Mesh design

The force action of the rock volume on the mesh:

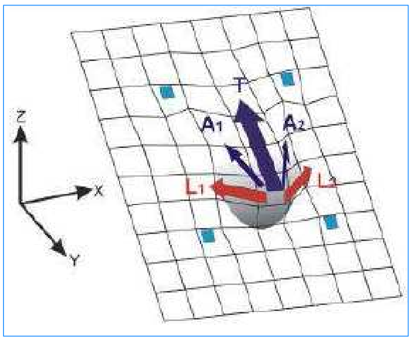
$$(W_i \cdot sen\alpha / \gamma_{RWi} + N_r) \geq (W_i \cdot \gamma_{DWi} \cdot (sen\alpha - cos\alpha))$$

$W_i = Vol \cdot \gamma \cdot i_x$ weight of the unstable mass retained by the mesh;
N_r: action that the mesh develops against the movement of the unstable block and that depends on the mesh characteristic curve

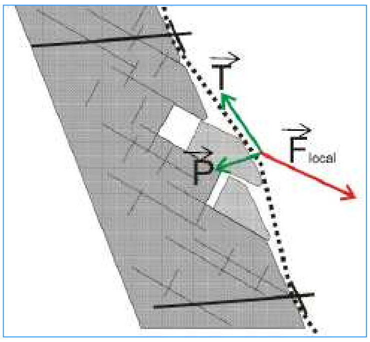
γ_{RWi}: reduction coefficient of the stabilizing forces, it taking in to account the progressive surface degradation, can be assumed equal to 1.15;
γ_{DWi} : multiplier coefficient for pro-safety permanent loads, it taking in to account the uncertainties of the adopted calculation model and it can be expressed by the product of three factors: $\gamma_{DWi} = \gamma_{DWis} \cdot \gamma_{DWig} \cdot \gamma_{DWia}$
dove γ_{DWis}coefficient which takes into account the reliability of the value attributed to s, it can be assumed equal to 1.3 if the s (thickness of the altered rock band) determination was made with geomechanical measurements in site, and 1.5 if the evaluation of s is empirical;
γ_{DWig} coefficient which takes into account the uncertainty determination of the loose rock. weight per unit volume it can usually be set equal to 1.0, while if there is uncertainty in some cases it is suggested take 1.05 (marly rocks or flysch)
γ_{DWia} coefficient taking into account the environmental degradation of the rock that can usually be equal to 1.0, whereas if the rock is altered, it is suggested to adopt 1.05.

Mesh design

The tensile strength of the mesh must be compared with the action of tensile actions resulting from vector analysis



Simplified diagram adopted for the resisting forces developed by the mesh. The blue squares indicate the position of the nails (Giacchetti and Bertolo, 2010).

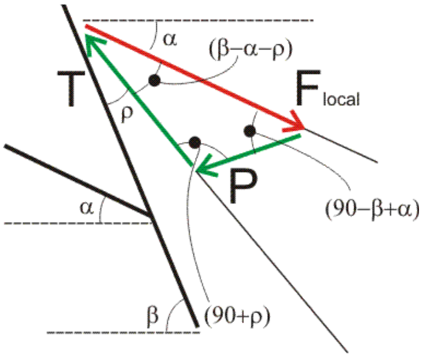


Two-dimensional scheme in the plane of sliding of the forces generated by the mesh between the nails (Giacchetti and Bertolo, 2010)

Mesh design

The geometry of the problem is three-dimensional

The action against the punching developed by the mesh is against:
- the tensile strength according to the tangential components (T) upward (i.e. in the vertical plane containing the direction of slip);
- the lateral direction perpendicular to the sliding plane (L1 and L2)



Mesh design

The resultant of forces L1 and L2 in the vertical plane containing the direction of sliding is represented by the force P

$$N_r = P + T = F_{Local}$$

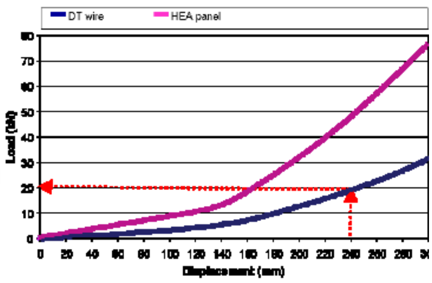
the module of the P force is equal to:

F_{Local}.sin (β–α);

the deformation induced is given by:

$\delta = \arctan \Delta_{pnz}/1.5$

Δ_{pnz} = f(P), function dependent by the experimental load-displacement curve of the mesh



Consequently, on the basis of these relationships we can be expressed the T_{force}:

$$\mathbf{T = (F_{Local} \cdot \sin(\beta-\alpha)) / (i_x \cdot \sin(\beta-\alpha-\delta))}$$

Mesh design

So the mesh is tested only when both the following conditions are verified:

$$\begin{cases} T_R / \gamma_R \geq T \text{ (ultimate limit state (ULS));} \\ D_{pnz} \cdot \gamma d\Delta \leq \Delta_o \text{ (serviceability limit state (SLS));} \end{cases}$$

T_R tensile strength of the mesh determined by experimental tests

γ_R factor to be applied to the resistance of the mesh, which is suggested to be chosen not less than 3 to take into account the many uncertainties of the problem;

$\gamma d\Delta$: factor to be applied to the deformation of the mesh, that takes into account the differences between the constraints under ideal conditions and those realize on site, is expressed by: $\gamma d\Delta = \gamma_{\Delta C} \cdot \gamma_{\Delta M}$

$\gamma_{\Delta C}$: coefficient of increase that takes into account the type of link between the mesh and anchors, that can be set equal to 2.0 in the case of mesh constrained only by the nails and plate or equal to 1.5 in the case of mesh also constrained by grid of ropes;

$\gamma_{\Delta M}$: coefficient that takes account of the morphology of the surface that can be placed equal to 1.3 for smooth surfaces and 1.5 for rough morphology, with poor adhesion between the mesh and the rock;

Δo: expresses the deformation of the project that value should take into account the working condition of the mesh and it depends on the geometric boundary conditions, we can be adopted it equal to 1.0m, if there aren't particular aesthetic and functional problems, instead equal to 0.5m in the case of interference with infrastructure

Additional checks

The additional verification regarding the resistance of the wires of the mesh at the anchors.

This analysis require the use of complex numerical methods, and in fact cannot be make a systematic check of all the load conditions in the neighbourhood anchor.

However you can take the pull out test results in full scale test and similar boundary conditions to what is usually constructed on site and compare it with stresses transmitted to the nails and then bound them to the mesh.

CONCLUSIONS

The behaviour of drapery mesh is greatly influenced by the boundary condition.

The “system” (mesh+cables+bolts) can show large displacements before starting to apply a a real confining force to the moving block.

It is very difficult (or impossible ?) to install the mesh in a way to apply a force on the slope before its start to move (more tests are necessary, also with other type of fabric, to completely evaluate this concept)

The displacement vs applied load curve is a key parameter for the design.