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Analysis of funicular structures and tests on materials for specialized arboreal cultivations

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ABSTRACT: An analysis is made with the finite elements method on spatial support systems (Pergola) for table grape and actinidia cultivations, which are currently expanding throughout Italy and other Mediterranean countries, by defining the global static-displacement behaviour of the structure subject to the service loads and to the wind action. Then the results of mechanical strength tests on samples of materials to be used for the structural members subject to tension are illustrated. On the basis of the achieved results, criteria of structure design and of choise of traditional and new materials, are obtained.

RESUME': L'analyse se fait par la méthode des éléments finis de systèmes spatiaux de soutenement pour la culture de raisin de table et d'actinidia, actuellement en expansion en Italie et en d'autres Pays de la Méditerranée, définissant le comportement statique et déformatif global de la structure exposée aux charges utiles et à l'action du vent. On régistre en outre les résultats des éssais mécaniques d'endurance sur des échantillons de matériaux à utiliser pour les éléments structuraux exposés à la traction. Sur la base des résultats obtenus on tire des principes de projet de la structure et de choix des matériaux nouveaux et traditionnels.

ZUSAMMENFASSUNG: Mit der Methode der finiten Elemente führen wir eine Analyse von räumlichen Trägersystemen (Pergolen) für den Tafeltrauben und Actinidiaanbau durch, der momentan in Italien und anderen Mittelmeerländern in zunehmendem Maße betrieben wird. Wir bestimmen dabei die Nutzlast und Windlasten ausgesetzten Konstruktion hinsichtlich statischer Verformungen. Wir erläutern weiterhin die Ergebnisse von mechanischen Festigkeitsprüfungen an Materialproben, die für die Zugkraft ausgesetzten Konstruktionselemente verwendet werden können. Aufgrund dieser Ergebnisse erhalten wir Kriterien für die Ausmaße der Konstruktion und die Wahl von traditionellen und innovativen Materialien.

1 FOREWORD

The present trend for table grape and actinidia (kiwi) arboreal cultivations tends towards forms of pergola growing, since they allow us to obtain the best quantitative and qualitative production results (Di Lorenzo 1987) and, at the same time, to rationalise the vegetable apparatus disposition in function of coltural, irrigation and crop operations.

Nowadays, in Italy, such a growing system for table grape production occupies an area of about $8.0 \cdot 10^4$ Ha, with a yearly production of about $1.7 \cdot 10^6$ tons, of which 40% is intended to be exported (Baccarella 1987); its diffusion is also worthy of note in other Mediterranean countries, particularly in Spain (Negueroles 1987). In Italy there is a remarkable expansion of actinidia cultivations, too, practised with pergolas in the Southern regions, with yearly productions of $4.5 \cdot 10^5$ tons on an area of about $9.0 \cdot 10^3$ Ha.

Since currently the realization of such plants is based on empirical constructive criteria, we studied the static-deforming behaviour of the structure in its whole by using the finite elements method applied to schemes close to the most representative real model, under several load conditions, and by giving design criteria of such structures for agriculture and choice of materials. To this purpose we carried out some laboratory tests on some traditional and new materials to be used to make the supporting structure members subjected to tension.
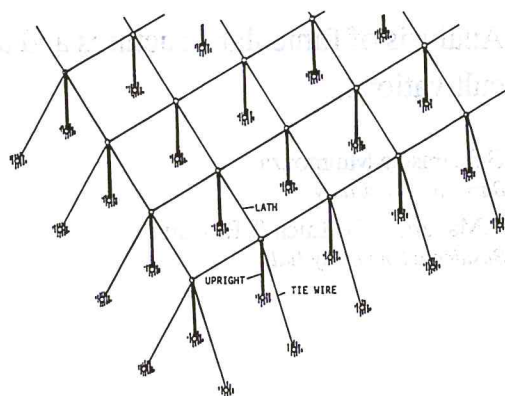

Fig. 1. Axonometric view of a pergola.

2 STRUCTURAL ANALYSIS

2.1 Geometry description

The adopted geometrical scheme is considered the most suitable to the agronomic demands (Liuni 1978), as can be remarked by results of the present realizations.

The pergola structure (Fig.1) can be seen as essentially composed of a suspended plane parallel to the soil, set at a certain height, made of two main orthogonal counterlaths of wires forming a square mesh grid supported by a series of uprights, or posts, connected with the mesh wires meeting points. The so-formed network is made, together with a secondary strings counterlath, to support the vegetation and the fruits deriving from the arboreal stems sited in correspondence of each upright. Vertical uprights and net-like mesh are in equilibrium by the border tie wires inclined and anchored to the soil.

The structure is sorted locating some members which can be traced back in two essential types. In fact, assuming that the meeting joints between two orthogonal wires and the post represent spatial pinned joints, and that the uprights and the inclined border tie wires are restrained to the soil by spatial hinges, the two kinds of members are: the upright, corresponding to a bar reacting to compression, and the string segment between two joints reduced to essentials

as a wire supported at the two ends, only reacting to tensile forces.

The adopted dimensions are the following: square mesh lenght: 3.00 m; uprights height: 2.20 m; ties inclination: 45°.

2.2 Calculation method

The structural analysis of the located scheme, made of bars and wires, is carried out with the finite elements method (Brebbia 1982) based on the following main operations:

– loads distributed on wires, on the basis of considerations of highly tension wires' compatibility and equilibrium (Belluzzi 1977), are traced back to equivalent energy joint loads collected in the $\underline{P}$ vector;

– the resolving system is expressed in the joint displacements unknown quantities, indicated by matrix $\underline{U}$, and it is obtained starting by a configuration already compatible, through the static equilibrium in each joint: $\underline{K} \cdot \underline{U} = \underline{P}$ (1) where $\underline{K}$ is the structure's global stiffness matrix.

Obtained the unknown quantities joint displacements $\underline{U}$, by solving the aforesaid equation system, we determine the joint loads through the relations of constitutive link of members, and eventually the resultant forces at the members end. With the word member we mean both the post and the wire segment; this

latter has a non-linear load extension law which is computable (Belluzzi 1977) by the taking on of a fictitious elasticity modulus of the wire equal to:

$$Ef = E/(1 + (q^2 L^2 EA/12 N^3))$$ (2)

where:
E = effective elasticity modulus ($2.1 \cdot 10^5$ N/mm²);
L = span of the wire segment;
q = uniform distributed load;
A = wire cross sectional area;
N = axial stress.

Regards the geometrical non-linearities induced in the structural complex, these are neglected because of the slight influence on the static condition.

The calculation pattern was expressed in operative terms in a resolutive algorithm and codified in a proper computer program (in Pascal language operating in MS-DOS).

2.3 Tie wire anchor system

Presently anchors are made in an empirical way, without any static trial-and-error test, with dead-man blocks embedded into the soil. After studies (Briassoulis 1984) on this subject, we inferred indications about the most efficient anchor systems and relative pull-out values by which we deduce allowable loads which can be assumed in the examined structures. In fact screw, arrowhead and dead-man type inclined anchors, have pull-out capacities of 533÷769 daN (for 15.2÷20.3 cm diameter), 159÷303 daN (20.3÷25.4 cm), 263 daN (for a cube with side 15.2 cm) respectively, calculated by the equation:

$$Qu = (Nu) c A + S$$ (3)

where:
Qu = pull-out load capacity;
Nu = pull-out coefficient;
c = undrained shear strenght of the soil;
A = projected area of the anchor plate;
S = shaft resistance.

For this case the values given above must be reduced by applying suitable safety coefficients.

2.4 Loads

While in a previous study (Scarascia 1988) we only considered the dead and the live loads, in this second study we also considered some combinations with additional loads. In fact, apart from the dead loads made by structure's weight and by vegetation, and the live loads represented by fruit, we considered the wind load which can cause a pergola collapse. In the evaluation of the wind load we only examined the static effect, neglecting the dinamic one, according to the Italian Standard (Min. LL.PP. 1982) on Constructions which, for wind blowing parallel to the soil and for rectangular plant plain covering, state a value of static pressure:

$$p = c q$$ (4)

where :
c = coefficient of shape or exposition (± 0.6);
q = kinetic pressure (60 daN/m²).

Such wind load is carried out for "sail effect"; the wind canalizes below the suspended plane supporting the pergola and, by bumping against the soil, acts as kinetic pressure or depression on the cover surface.

The vertical eolian action has been evaluated, not being a civil building, by considering the effective degree of covering of the surface itself and the non-contemporaneousness of the wind action on the whole structure; so, wind load (4) has been corrected with a coefficient 0.17, then the uniformly distributed wind load is p = 6 daN/m².

Concerning the wind horizontal loads the static pressure on vertical and horizontal border elements was evaluated as follows (Ministero LL.PP. 1982):

$$W = c q S$$ (5)

where:
c = shape coefficient of the exposed surface (1.1, 1.2 for horizontal and vertical border members respectively);
q = kinetic pressure (60 daN/m²);
S = exposed surface for lenght unit of the

member.

Such effects are taken back to horizontal forces concentrated on the border joints and, in this case, they contribute with a concentrated load of 14 daN .

Service and additional vertical loads due to the wind act on the network and are distributed on the wires, of span L = 3.00 m, considering the influence areas for the wires themselves.

We considered the following load condition combinations, referred to a spatial cartesian reference system with x–y axes, parallel to network's sides, and z vertical:

I) dead loads + live loads:

$qz = -7.5$ daN/m uniformly distributed;

II) dead loads + live loads +horizontal forces due to the wind:

$qz = -7.5$ daN/m;

$Fx = 14$ daN;

III) dead loads + live loads + wind loads uniformly distributed downwards + wind horizontal forces:

$qz = -16.5$ daN/m;

$Fx = 14$ daN;

IV) dead loads + wind loads uniformly distributed upwards + wind horizontal forces:

$qz = +7.5$ daN/m;

$Fx = 14$ daN .

3 TENSION TESTS ON MATERIALS

Within the members which can be distinguished in the pergola structures, namely the soil anchors, the compression and tension members, those latter, made of strings which could be made with several materials, were object of study and laboratory tests. In fact the tension members, together with the soil anchors, seem to be the structural components which are more susceptible to development from the point of view of a correct design and of the application of new materials.

3.1 Materials and methods

To obtain the allowable stresses to be introduced in the structural analysis, we conducted laboratory tension tests on specimens of materials largely used in the present tecnique of plants for pergola cultivations, as galvanized iron wires with ordinary superficial galvanizing, equal to 90 g/m^2 ,and a series of specimens of materials such as stainless steel wire AISI 304 and 310 which, together with AISI 430 and 434, are more and more affirming for their specific peculiarity as regards the characteristics of tensile properties and corrosion resistance (Melotti 1986).

The tests, carried out according to the Italian Standards (UNI 1979), were conducted in the laboratory for material tests of the Institute of Rural and Forest Engineering of the University of Basilicata; the specimens, obtained by wires of diam. 1.5, 3.0 and 4.4 mm (10 for each sample) had a total length of 300 mm with an useful length stretch of 200 mm. The tests were made on a Galdabini PMA 10 universal testing machine, with a speed of testing not superior than 30 $N/mm^2 \cdot s$.

3.2 Laboratory tests results

The results, expressed in average value, are reported in Tab. 1 .

Table 1. Tension tests results on galvanized iron and stainless steel wires specimens.

Material	fs	ft	A%
galvanized iron	37.122 ± 0.233	46.101 ± 0.212	12.625 ± 0.688
stainless steel	----- -----	84.920 ± 0.214	9.655 ± 1.259

where:

fs = tensile strength at yield (daN/mm^2);

ft = tensile strength at break (daN/mm^2);

A% = per cent elongation at break.

From Fig. 2, reporting the stress-strain diagram for a galvanized iron wire and a stainless steel one having the same diameter (1.5 mm), emerges the different behaviour of the two materials under the same test's conditions. From the achieved results, we inferred the tensile stress values (σ_c) to be introduced in the calculation for the tension members.

Referring to the Italian Standards and considering that materials for the examined structures don't have to satisfy to the same factors of safety of the materials adopted in buildings we can write:

$$\sigma_c = 0.75 \, ft/ \, \gamma_m \qquad (6)$$

where γ_m = 1.5, 1.33 in absence and in presence of wind action load conditions respectively. From (6) we obtained the following values in daN/mm^2 :

Table 2. Design unit stress values for galvanized iron and stainless steel wires without wind and with wind load combination.

Material	LOAD CONDITION	
	wind absen.	wind pres.
	σ_c	σ_c
galvanized iron	23.0	26.0
stainless steel	42.5	48.0

4 RESULTS AND DISCUSSION

In a previous study we only considered service loads and structure having constant wires cross section areas for any tension member. An analysis made in the aforesaid way allowed us to determine, with the same extensional stiffness of the tension members, the stresses differences in the characteristic members: tie wire, lath and upright.
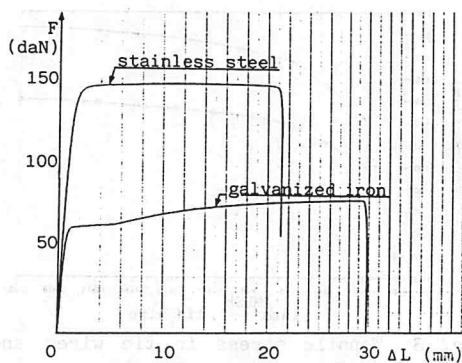
From the obtained results it has been

Fig. 2. Stress-strain diagram for 1.5 mm diameter wires in galvanized iron and stainless steel AISI 304.

possible to put the higher stress level of the border wires in evidence, compared to that one of the laths, because they were made to support the structure in its whole, so in this second study we used larger diameters for tie wires compared to those ones of the internal wires by admitting a theoric deflection to get a material saving. With the calculation program application we determined the stresses and the deformations of the members with the variation of:

– number of structure meshes;
– values of induced stress condition due to initial pretractions;
– combinations of acting loads;
– cross sections of members.

So the elaboration of the obtained results was done on the main members and drawn on the following diagrams.

The diagram of Fig. 3 was done to analyse the influence of the number of meshes constituting the structure on the stresses in the tension members; it is shown that curves (relative to tie wires and laths) are close to the stress asymptotic value in corrispondence to a pergola with 20 x 20 meshes: this allows the assimilation of the structure with an infinite number of meshes to the 20 x 20 mesh and so the generalization of the obtained results.

Comparing the stresses resulting in tie wires, for the loads combinations considered (Fig.4), we inferred that the heaviest combination is III, corresponding
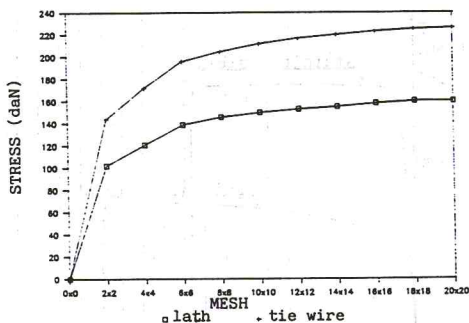
Fig. 3. Tensile stress in tie wires and laths, 4 and 3 mm diameter respectively, in function of the number of meshes. Load combination I, pretraction force 75 daN.

Fig. 5. Tie wire tensile stress-pretraction diagram for different cross section areas. 20 x 20 mesh structure, load combination III.
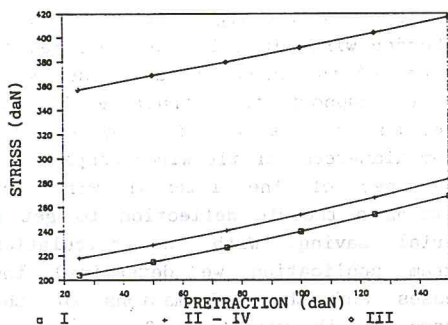

Fig. 4. Tie wire tensile stress-pretraction diagram for different load combinations. 20 x 20 mesh structure, diameter 4 and 3 mm for tie wires and laths respectively.

Fig. 6. Lath tensile stress-pretraction diagram for different cross section areas. 20 x 20 mesh structure, load combination III.

to the presence of fruits and to wind horizontal and vertical action. The resulting stresses in both loads combinations II and IV are the same.

Considering the stress developed in tie wires (Fig.5) and in laths elements (Fig.6), in function of pretraction and at the cross section variation, we see that the tensile force shows a sensible increase with the increase of the wires diameters, and in a lesser extent, with the increase of the imposed pretraction. So the use of wires made by materials with a better unit tensile stress –as stainless steel– allows us to reach the double objective of reducing cross sections and containing the forces resulting on the
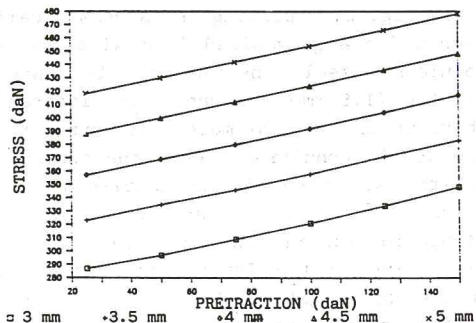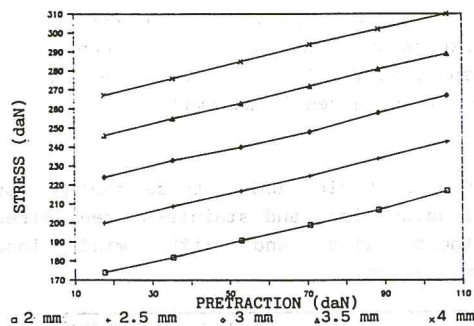
anchors. In fact, the increase of tensile stress allows us to reduce the cross sections, tensile forces being equal and, since the system is hyperstatic, such reduction developes a further stress decrement (Fig. 5 and 6)and wires section decrease. In this case, for example, the tensile stress difference between galvanized iron and stainless steel (Tab.2) would give, with the same tension, about a 45% reduction of stainless steel strings cross sections. This reduction reaches 55% if we carry out the global structural analysis with the effective required cross sections. The economies, reached in this way, in terms of savings of material in the tension members and in the soil anchors can counterbalance the

higher costs due to the use of stainless steel wires.

However pergola plants with galvanized iron wire, as used nowadays, need an oversizing of cross sections because of corrosive phenomena which cause dangerous reductions of the cross section areas. So it is suitable to study materials, alternative to the traditional ones, which, in spite of the higher unit costs, allow us to obtain high reductions of the quantities with further economies realizable in the construction phase, thanks to the better capacity of working on strings with little diameters. Moreover we must not neglect the longer endurance of the stainless steel wires compared to the galvanized ones, which because of the corrosions occuring during the pergola life, need frequent changes with consequent high maintenance costs.

The tendency of wires' deflections at centre of span, in function of the pretraction and for different used diameters (Fig.7), is inversely proportional to the strings cross sections and sparely dependent on the stress condition induced in the construction phase, so the material saving which can be obtained by using strings with little diameters involves, as negative effect, an increase of the maximum deflection under load that we'll consider for the functionality of the plant.

The structural global deflected form in load combination I and III, without support failures, puts in evidence that pinned joints on the uprights tops deflect within the network plane. The deflections -of few millimeters- are directed towards the internal part of the structure, with decreasing values passing from the external joints to those placed along the simmetry axes.

Concerning the stresses in posts, by the results of this analysis we deduced that corner posts are subject to compression by far superior than the others (Tab.3). Given the static importance, in the border upright design it will be necessary to consider the slenderness ratio and also possible impacts caused by farm tractors.

In the case of acting wind loads in depression on dead loaded pergola (loads
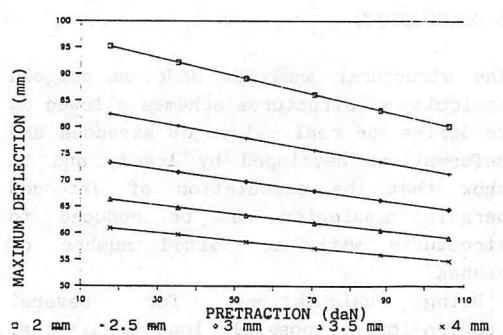

Fig. 7. Lath maximum deflection-pretraction diagram for different cross section areas. 20 x 20 mesh structure, load combination III.

combination IV) it is necessary to verify that the tension in uprights will not exceed the weight and the pull-out resistance of themselves. If the value is higher, the lift of the internal posts could occur, with consequent damages and possible collapse of the structure.

Table 3. Upright compression stress envelope for minimum and maximum pretraction force (25 and 150 daN).

DIAMETER CROSS SECTIONS		corner post	border post	int. post	int. post (tens.)
Lath (mm)	Tie wire (mm)	qIII (daN)	qIII (daN)	qIII (daN)	qIV (daN)
2.0	3.0	409÷490	254÷295	100	45
2.5	3.5	458÷540	279÷320	100	45
3.0	4.0	504÷585	302÷342	100	45
3.5	4.5	547÷627	324÷364	100	45
4.0	5.0	586÷663	344÷384	100	45

5 CONCLUSIONS

The structural analysis made on pergola agricultural structures schemes allowed us to define the real values of stresses and deformations developed by loads, and to show that the calculation of extended pergola tipologies can be reduced to structures with a limited number of meshes.

Doing calculations for several combinations of possible load conditions, we remarked that the heavier condition is given by the service loads and wind actions, so in windy areas it is suitable to use natural or artificial windbreak.

We also analysed the possibility of realizing tension members with several cross section areas in order to use new materials alternative with galvanized iron generally used. In fact, from the results of some tests, we saw that stainless steel strings have tensile strength higher than galvanized iron wires; then we must not neglect the higher resistance of stainless steel to corrosion which is a big problem for farmers. The use of such materials would allow us to obtain remerkable savings in cross sections sizing, in construction operations and in structure maintenance.

Eventually, we must underline that the correspondence of the theoric calculation pattern to the real structure depends, above all, on the effective assembly operations, which don't often correspond to the supposed ones. So it is necessary to pay attention to realizations of anchors and of uprights supports, compared to the lift for "sail effect" of wind, by orientating the design choices towards low values of pretraction stress in wires to obtain economies in soil anchors too.

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