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Case study

Structural design of an Italian bamboo house in an Italian regulatory context: Revisiting a small building built in Costa Rica with tropical bamboo

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ABSTRACT

A residential building designed by the architect Anna Zanetti and already built in Costa Rica using Guadua angustifolia and Dendrocalamus asper is redesigned, choosing the municipality of Bologna, in the north part of Italy, as the construction site and Moso (Phyllostachys edulis) cultivated in Italy, as structural material. The redesign is performed in the context of Italian standards. The paper demonstrates how it is possible to design a building using European bamboo, geometrically smaller in respect to the tropical one, in compliance with the Italian national standards NTC2018 with the aid of Bamboo International standards (ISO 22156). All the members are multi-culms and the connections are chosen without perforations.

1. Introduction

We are facing a wide range of challenges that claim sustainable choices. There is growing recognition and awareness that nature can help providing viable solutions. For this reason, we are looking with increasing interest at the use of natural materials for construction and, in particular, bamboo. Bamboo is one of the more effective resources which satisfy the requirement of green material for its amazing rapid grow, its capability to generate oxygen and its related low embodied energy. Bamboo is native of tropical and subtropical zones; however, it has also spread to temperate regions such as southern Europe where they can easily grow, if cultivated. In Italy, for example, nowadays there are about 2000 ha of bamboo that require the establishment of a real active supply chain. Cultivated bamboo of temperate zones has a smaller cross section than that cultivated in the tropics, it has good mechanical properties as shown by a recent experimental campaign on various Italian bamboo species [1–4].

The use of bamboo as a building material in many European countries, such as Italy, is limited by the lack of specific regulations. In Italy, for example, the national current standard for construction NTC2018 have guidelines only for steel, concrete, masonry and wood. However, ISO technical standard ISO22156 [5] on the design of bamboo culm structures, published in 2020, has given specific guidelines and can be used together with the national standard.

The crucial point of the paper is to show that it is possible to design a house in an Italian/European context using Italian/European bamboo. The design process can be helped by using together the ISO 22156 [5] and the National regulatory system for construction of each European country, and in this case the NTC2018 [6].

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2. Case study

The case study was inspired by a small bamboo building for residential use designed by architect Anna Zanetti and already built in Costa Rica, shown in Fig. 1. The structure of the building is a beam-column system joined with through culm wall dowels. In the original project, the bamboo culms were of the *Guadua angustifolia* and *Dendrocalamus asper* species which have average diameters of approximately 150 mm and average thickness of about 15 mm.

The building was redesigned using Italian bamboo belonging to *Moso* (*Phyllostachys edulis)* species. The dimensions of the culms are smaller with average diameter and wall thickness of 70 mm and 7 mm, respectively, and a maximum usable length of 6 m. According to recent experimental investigations [1–4], this bamboo species has good mechanical characteristics. The municipality of Bologna (north part of Italy) was chosen as the construction site. The requirements set out in Italian standards as NTC2018 [6] and CNR-DT 207/2008 [7] were adopted to calculate the loading combinations due to actions such as snow, wind and earthquake.

3. Characteristics of the material

The average values of tension ($f_{t,0}$), compression ($f_{c,0}$), and bending strength ($f_{m,0}$) parallel to the fiber and bending strength orthogonal to the fiber ($f_{m,90}$) and shear strength (f_v) of the *Phyllostachys edulis* bamboo are shown in Table 1 with the respective characteristic values calculated as suggested by ISO 22156 [5] (5% fractile with 75% confidence), using the data in [1–4]. Table 1 also shows the design resistance values for instantaneous, semi-permanent and permanent loads [5]. For the same cases, the modulus of elasticity is also reported in Table 2. The values of the shear strength are obtained considering the 8% of the tensile strength, as suggested in [8].

4. Structural model

The structure was designed using the finite element software SAP2000 [9] (Fig. 2). The material was modeled as an orthotropic elastic material with the same elastic constants in the radial and circumferential direction.

Internal forces in the elements of the structure depend on the stiffness of the joints, which is not well stablished. Thus, to consider extreme conditions for the verification of the columns and the beams two models of the same structure were analyzed. In the first one the connections between beams and columns were represented as rigid to maximize the stresses in the columns. In the second model, the connections between beams and columns were modeled as hinge to maximize the stresses in the beams. The highest stresses deriving from these two models were considered for each structural element to be in the most unfavorable possible condition. Bending, shear, compression, compression-bending, tension-bending verification were performed as reported in ISO 22156. The total bearing capacity of a structural element composed of various culms was calculated, as suggested by ISO 22156, by adding the bearing capacities of the individual culms constituting the element, which means that the culms are considered not to cooperate with each other, as confirmed in [10].

The structural elements were verified at ULS (Ultimate state) as reported in the Italian Standard (NTC2018).

The bending verification was carried out verifying that:

$$M_r \geq M_d$$

being M_d the bending moment, and M_r the flexural member capacity, obtained by summing up the flexural capacity of each culm. In particular, $M_r = \sum_i M_i$, where M_i is the flexural culm capacity equal to $f_m \cdot S_i$, where f_m is the bending strength parallel to fibres and S_i the elastic section moduli of the individual culm. As suggested by ISO 22156 a redundancy factor equal to 1.1 due to the multi-culm elements was considered in the calculation of the stress capacity of the structural elements, except for the single-culm purlins of the two

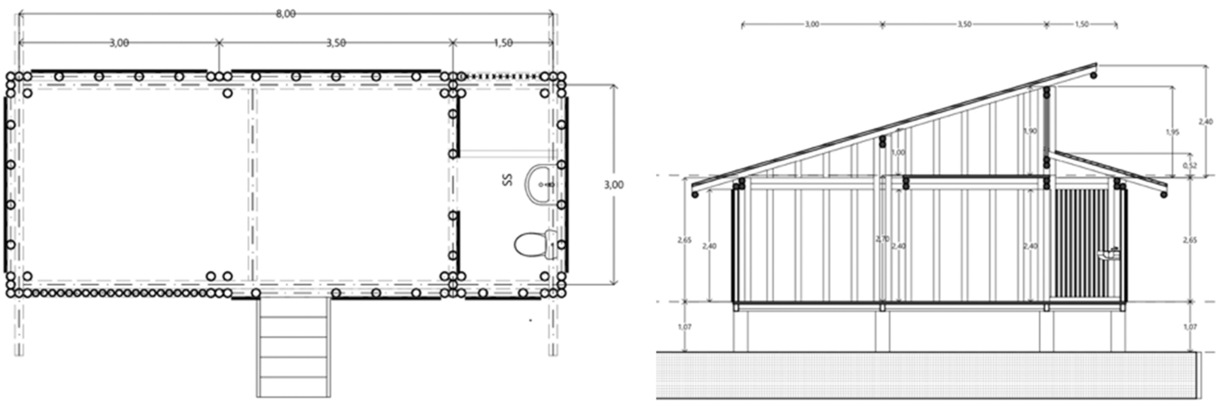

Fig. 1. Plan and section of the original building.

Table 1
Average, characteristics and design strengths of Italian Moso bamboo (expressed in MPa).

	Mean value [MPa]	COV [MPa]	Characteristic value (5% fractile with 75% confidence) [MPa]	Design value [MPa]		
				Instantaneous loads	Semipermanent loads	Permanent loads
Tensile strength	$f_{t,0,mean}$ 193,5	42,7	$f_{t,0,k}$ 98,4	$f_{t,0,d}$ 46,0	$f_{t,0,d}$ 35,2	$f_{t,0,d}$ 29,8
Compression strength	$f_{c,0,mean}$ 68,7	8,1	$f_{c,0,k}$ 49,7	$f_{c,0,d}$ 23,3	$f_{c,0,d}$ 17,8	$f_{c,0,d}$ 15,0
Bending strength parallel to fiber	$f_{m,0,mean}$ 97,3	13,6	$f_{m,0,k}$ 65,8	$f_{m,0,d}$ 30,8	$f_{m,0,d}$ 23,5	$f_{m,0,d}$ 19,9
Bending strength orthogonal to fiber	$f_{m,90,mean}$ 27,2	4,4	$f_{m,90,k}$ 17,2	$f_{m,90,d}$ 8,1	$f_{m,90,d}$ 6,2	$f_{m,90,d}$ 5,2
Shear strength	$f_{v,mean}$ 15,5	/	$f_{v,k}$ 7,9	$f_{v,d}$ 1,8	$f_{v,d}$ 1,4	$f_{v,d}$ 1,2

Table 2
Averages, characteristics and design values of elastic moduli of Italian Moso bamboo (expressed in GPa).

	Mean value [GPa]	COV [GPa]	Characteristic value (fractile 5%, 75% Confidence) [GPa]	Design value [GPa]		
				Instantaneous loads	Semipermanent loads	Permanent loads
Tensile Elastic modulus parallel to the fiber	$E_{t,0,mean}$ 15,2	2,9	$E_{t,0,k}$ 14,61	$E_{t,0,d}$ 14,61	$E_{t,0,d}$ 13,88	$E_{t,0,d}$ 6,57
Compression Elastic modulus parallel to the fiber	$E_{c,0,mean}$ 14	3,64	$E_{c,0,k}$ 13,26	$E_{c,0,d}$ 13,6	$E_{c,0,d}$ 12,60	$E_{c,0,d}$ 5,97
Elastic modulus parallel to the fiber in bending	$E_{m,0,mean}$ 13,2	1,0	$E_{m,0,k}$ 13,00	$E_{m,0,d}$ 13,00	$E_{m,0,d}$ 12,35	$E_{m,0,d}$ 5,85
Elastic modulus orthogonal to the fiber in bending	$E_{m,90,mean}$ 3,2	0,26	$E_{m,90,k}$ 3,2	$E_{m,90,d}$ 3,19	$E_{m,90,d}$ 3,04	$E_{m,90,d}$ 1,43
Tangential elastic modulus [7]	G_{mean} 2,88	0,1	G_k 2,52	G_d 2,52	G_d 2,39	G_d 1,13

roofs, for which it was set equal to 0.9.

The shear verification was carried out verifying that:

$$V_r \geq V_d$$

where V_d is the shear force on the structure members and V_r is the shear capacity of the member given by the sum of the shear capacity of each culm with the following formula:

$$V_r = f_v \bullet \sum_i \frac{3 \bullet \pi \bullet \delta}{8} \frac{D^4 - (D - 2\delta)^4}{D^3 - (D - 2\delta)^3}$$

where D is the diameter of the culm and δ is the wall thickness of the culm wall.

The verification of compression member was carried out verifying that:

$$N_{cr} \geq N_{cd}$$

where N_{cd} is the compression stress, while N_{cr} is the compression capacity of the member

$$N_{cr} = \frac{P_c + P_e}{2c} - \sqrt{\left(\frac{P_c + P_e}{2c}\right)^2 - \frac{P_c \bullet P_e}{c}}$$

which is function of the maximum compression axial force $P_c = f_c \bullet \sum_i A_i$ and of the maximum force considering instability $P_e = \frac{n \bullet \pi^2 \bullet E_d \bullet I_{min} \bullet C_{bow}}{(KL)^2}$ and $c = 0,8$, (Zahn 1993). Being f_c the design compressive strength of the bamboo parallel to the fibers, A_i the cross section area of the culm, E_d the design modulus of elasticity in bending parallel to the fibers, I_{min} the minimum moment of inertia, C_{bow} the reduction factor that takes into account the non-linearity of the culms that make up the element, L the free length of deflection of the element, K the effective length coefficient and n the numbers of the culm of the element.

For the verification of compression combined with bending the following expression was considered:

$$\frac{N_{cd}}{N_{cr}} + B \bullet \frac{M_d}{M_r} \leq 1$$

where B is the moment amplification factor $B = \left[1 - \frac{N_{cd}}{P_e}\right]^{-1}$.

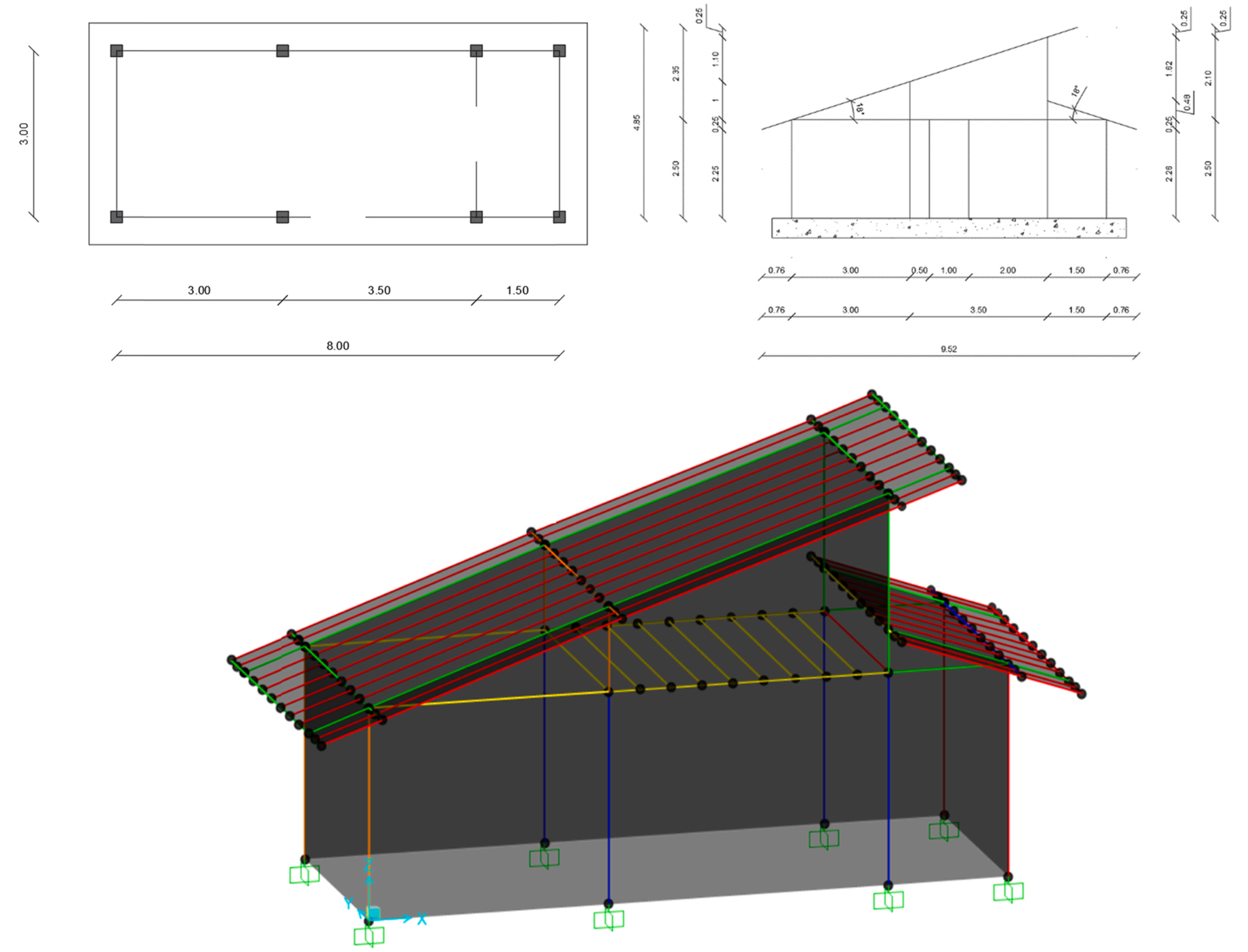

Fig. 2. The sketch of the structure and the model in SAP 2000.

Table 3
Moment, shear, and axial force for the different members of the structure (positive sign for compression axial force) and the required number of culms.

	Istantaneous loads			Short duration loads			
	M_d (kNm)	V_d (KN)	N_{cd} or N_{td} (KN)	M_d (kNm)	V_d (KN)	N_{cd} or N_{td} (KN)	minimum n. culms
Purlins 1	1	1,8	0	1	1,7	0	3
Edge Beam PC	1,25	1,83	4,7	1,2	1,8	3	7
Beam 1	4	7,2	2,9	3	6,6	2,6	8
Beam 2	3,9	8,7	3,2	3,4	7,5	3	9
Beam 3	3,8	8,4	2,3	3,4	7,3	2	8
Purlins 2	0,3	0,6	0	0,1	0,54	0	1
Edge Beam SP	0,6	1	8,9	0,4	0,9	5,6	2
Beam 4	2,4	3,7	1	1,6	3,5	0,8	5
Beam 5	1,8	2,2	22,1	1	1,6	-3	4
Purlins 3	1,64	2,2	0	1,64	2,2	0	4
Beam AB	2,1	2,1	4	1,73	1,8	0,64	5
Beam BB	2,2	2,1	1,1	1,2	1,5	-3,5	4
Beam BC	11	12	-10,3	11	12	-1,3	24
Beam CC	2,7	3,5	1,5	2,7	3,5	0,8	7
Beam CD	3,3	3,6	8,7	2,1	2	3,7	7
Column A	4,7	3,4	13,7	2	1,2	13,7	11
Column B	6	3,3	25,2	0,5	0,3	25,2	16
Column C	5,5	3,9	27,6	0,6	0,44	27,6	15
Column D	3,2	2,1	9,3	0,74	0,44	9,3	8
Column 1 A	2,54	3,5	15,7	2,2	3,1	13,3	6
Column 1B	5	8,8	15,2	3,3	2,9	15,2	10

For the verification at tension and bending stress the following expression is considered:

$$\frac{N_{td}}{N_{tr}}+\frac{M_d}{M_r}\leq 1$$

where N_{td} is the acting axial tensile stress, N_{tr} is the tension capacity of the structural element. In particular, $N_{tr} = n{\bullet}\mathrm{f_t}{\bullet}\mathrm{A_{min}}$ being f_t is the design tensile strength parallel to the fibers, A_{min} is the minimum cross section area of the culm.

The moments, the shears and the axial forces of the structure are reported in Table 3 for the different load classes. Considering these moments, shear and axial forces, the minimum number of culms needed for each member is calculated and it is shown in the last column.

The minimum number of culms required in some members like the beam BC is too high to be viable, so two main changes in the structural system are provided.

The first is to half the space between the beams of the main roof and between the columns which it becomes of about 1.5 m. The second is to insert X-bracing elements along the two main directions to have a better response in relation to the action of the wind. The modified structure is reported in Fig. 3.

The moments, shear and axial forces related to this modified structure are reported in Table 4 with the required number of culms in the last column.

For the structural elements associated with the intermediate floor slab, it is necessary to consider permanent load. In Table 5 the associated stresses.

5. Deformability verifications

The maximum displacements of the beams were calculated as the sum of the maximum displacements obtained for permanent (w_p) and temporary loads (w_Q). The seismic action was not considered. In case of simply supported beams the maximum displacements is:

$$w=\frac{\frac{5}{384}\,\frac{q{\bullet}l^4}{E_d{\bullet}C_v{\bullet}}I+\frac{1}{8}\,q{\bullet}l^2{\bullet}\chi}{G_d{\bullet}A}$$

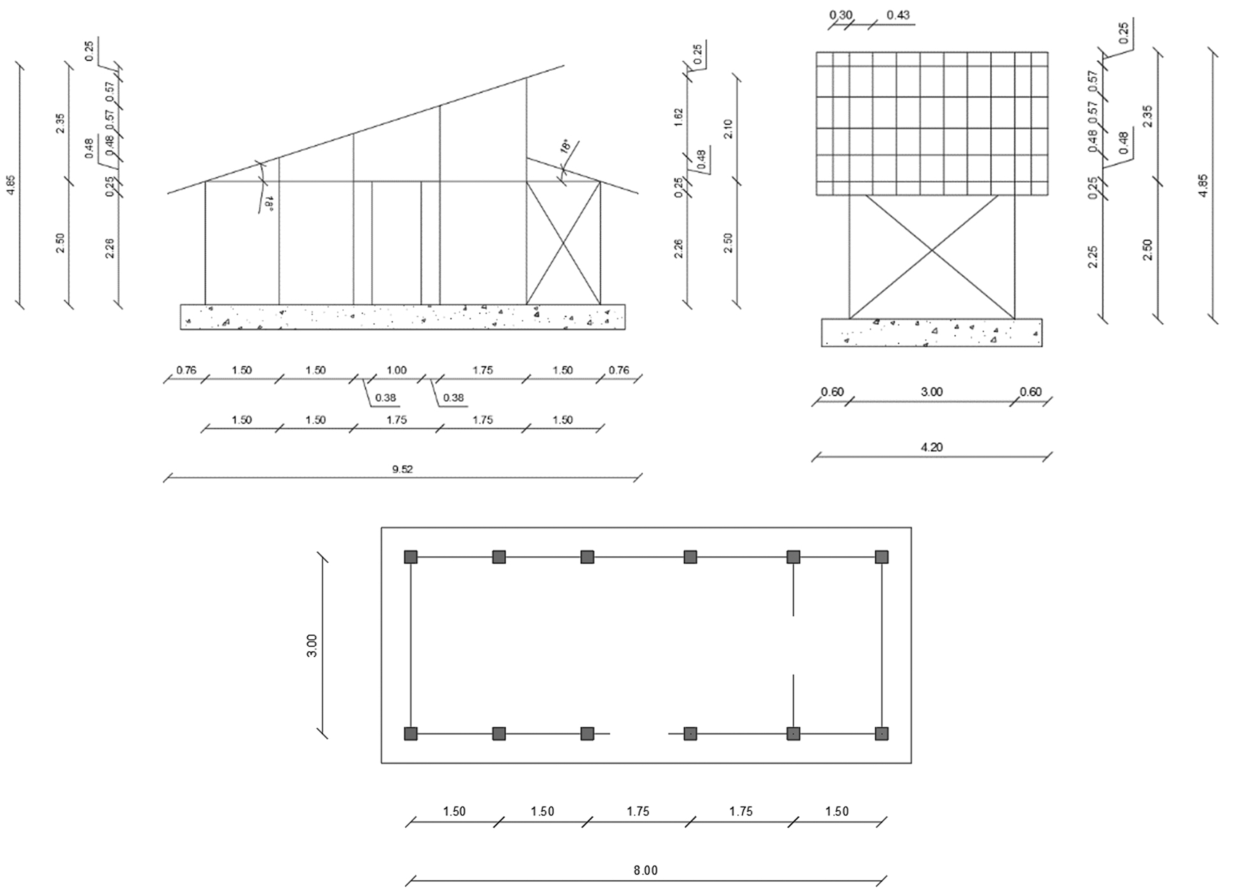

Fig. 3. The structural scheme of the modified structure.

Table 4
Moment, shear, and axial force for the different members of the structure (positive sign for compression axial force) and the required number of culms.

	Istantaneous loads			Short duration loads			Seismic action			minimum n. culm
	M_d (kNm)	V_d (KN)	N_{cd} or N_{td} (KN)	M_d (kNm)	V_d (KN)	N_{cd} or N_{td} (KN)	M_d (kNm)	V_d (KN)	N_{cd} or N_{td} (KN)	
Purlin 1	0,3	0,9	-4	0,3	0,8	0	0,2	0,2	0	1
Edg. Beam	0,9	1,2	6,5	1,1	2,3	1,6	0,6	0,8	3,2	4
Beam 1	1,8	4,4	-1,2	1,6	3,9	0,3	0,5	1,5	0,9	4
Beam 2	2,1	4,9	1,4	1,8	4,2	0,8	0,9	2,5	0,2	5
Beam 3	1,9	4,2	2,1	1,7	3,6	1,9	0,6	1,4	0,7	5
Beam 4	2,5	5,6	1,6	2,3	5	1,3	0,4	0,6	0,2	6
Beam 5	2,2	4,7	1,4	1,9	4,2	1,3	0,8	0,8	1	5
Purlin 2	0,4	0,6	0	0,2	0,6	0	0,5	0,6	0	1
Edg. Beam	0,5	1,3	1,9	0,4	1,2	0,5	0,5	0,6	0,4	2
Beam 6	1,7	3,8	0,7	1,4	3	0,1	0,8	1,8	0,6	4
Beam 7	1,1	2	3,7	1,1	1,9	1,3	0,7	1,7	2,8	4
Purlin 3	1,5	2	0	1,5	2	0	0,5	0,6	0	4
Beam AB	0,6	0,7	4,3	0,2	0,4	0	0,3	0,4	5,6	3
Beam BC	0,6	0,7	4,3	0,2	0,4	0	0,3	0,4	5,6	3
Beam CC	0,9	1,1	1	0,8	1	0	0,4	0,3	0	3
Beam CD	2,6	5,2	2,7	2,6	5,2	0	1,1	2,3	0,8	7
Beam DE	2,6	5,2	2,7	2,6	5,2	0	1,1	2,3	0,8	7
Beam EE	2,5	3,3	0,8	2,5	3,3	0	1,4	1,9	0	7
Beam EF	0,9	1,1	-11,8	0,1	0,2	0	0,6	0,8	4,2	3
Column A	1,3	0,8	8,5	0,9	0,6	7,3	0,2	0,1	4,2	5
Column B	1,1	1,9	8,6	1	0,8	7,6	0,5	0,3	1,4	5
Column C	1,2	0,9	11,4	0,3	0,1	11,4	1	0,7	3,3	5
Column D	1,6	1,1	19,1	0,2	0,1	19	1,6	1,1	6	7
Column E	0,9	0,7	19	0,3	0,2	16,2	0,6	0,9	12,4	6
Column F	0,7	0,4	6,2	0,5	0,3	3,3	0,1	0,1	6,7	5
Col. 1A	1,6	2	7,4	1,4	1,9	6,4	0,8	1,1	0,9	5
Col. 1B	1,6	1,5	9,4	1,4	1,3	8,4	0,7	0,7	1	5
Col. 1C	1,6	3,8	11,2	1,5	2,5	10,7	1	3	2,3	6

Table 5
Moment, shear, and axial force of the beam of the first floor subjected to permanent loads.

	Permanent loads		
	M_d (kNm)	V_d (KN)	N_{cd} or N_{td} (KN)
Purlins	1,5	2	0
Beam CC	0,8	1	0
Beam CD	2,6	5,2	0
Beam DE	2,6	5,2	0
Beam EE	2,5	3,3	0

In case of fixed ended beams, the displacement is

$$w = \frac{\frac{1}{384}\,\frac{q\bullet l^4}{E_d\bullet C_v\bullet}I + \frac{1}{8}\,q\bullet l^2\bullet\chi}{G_d\bullet A}$$

being q the uniformly distributed load; l the length of the beam; χ the shear factor (=2 for a circular crown cross section), E_d is the Young modulus which takes into account the load duration; Gd is the shear modulus; C_v takes into account for the shear deformation:

$$C_v = 0,5 + 0,05\bullet\frac{\mathrm{a}}{\mathrm{D}} \leq 1,0$$

where a is the length of the element cutting light.

The limit of the displacements is reported in NTC2018:

$$w_Q \leq \frac{l}{300}$$

$$w_{tot} \le \frac{l}{200}$$

where w_{tot} is the total displacement obtain by the summation w_p and w_Q; $w_{tot} = w_p + w_Q$.

6. Resulting structure

The resulting structure is shown in Figs. 4 and 5 with the number of culms in each column, beam, and purling. The column layouts were modified in respect to those permitted by ISO 22156 which requires columns to be symmetric about two axes or radially symmetric to simplify the connections. Fig. 6 shows three-dimensional images that give the overall idea of the structure.

7. Design of the joints

A joint which does not need the perforation of the culms was chosen to avoid axial inducing cracking. In particular, the connections which use steel semi-ring studied in [11–14] were adopted with and without the steel diagonal stiffener.

The verification of the connection was conducted as suggested by ISO 22156. In particular, the bearing capacity of the bamboo culm and the circumferential bearing capacity of the culm were verified. Also, the bending capacity was verified even if not required by standard. These verifications were conducted at the ultimate limit state.

Regarding the bearing capacity the following expression was considered:

$$P_b = C_{EB} \bullet \mathrm{f_c} \bullet \mathrm{A}$$

being C_{EB} the coefficient for the different types of cut of the final part of the culm. It assumes the value of 0.4 in the case of fish mouth, while it assumes the value of 0.8 in the case of straight cuts.

Regarding the circumferential stress capacity, the following expression was considered:

$$\frac{P_{cir} = p_{cir} \bullet \sin\left(\frac{\beta}{2}\right) \bullet \mathrm{D} \bullet \mathrm{L_{cir}} \le 2\mathrm{f_{m90}} \bullet \mathrm{L_{cir}^2} \bullet \delta^2 \bullet \sin\left(\frac{\beta}{2}\right)}{3D \bullet K_m}$$

being P_{cir} the resultant circumferential force, p_{cir} the circumferential pressure associated with P_{cir}; β the central angle that describes the portion of the circumference on which the pressure is applied; L_{cir} is the length along the culm on which pressure is applied; f_{m90} is the design resistance to circumferential bending of bamboo; K_m is a factor that depends on the angle β, it assumes the value of 0.5 for $\beta = 180^\circ$. Following the ISO22156 standard, P_{cir} has to satisfy the following inequality:

$$P_{cir} \le 0,5 \bullet \mathrm{L} \bullet \delta \bullet \mathrm{f_c}$$

The verification of the bending capacity was added even if not suggested ISO standard to verify the connection at bending:

$$M_{yd} \ge M_{d,i} = \frac{M_d}{n}$$

where M_d is the maximum external moment acting on one of the elements convergent in the node; $M_{d,i}$ is the bending moment acting on the i-culm of the member; n is the numbers of culms constituting the member; M_{yd} is the load capacity of each.

Three nodes shown in Fig. 5 are reported in detail in the following: the node between a column and the roof (node 1), the node between a column and the intermediate floor slab (node 2) and a node at the foundation (node 3).

Node 1 connects beam 4 with column 1B and the edge beam as shown in the figure. Beam 4 has 6 culms, column 1B has 5 culms

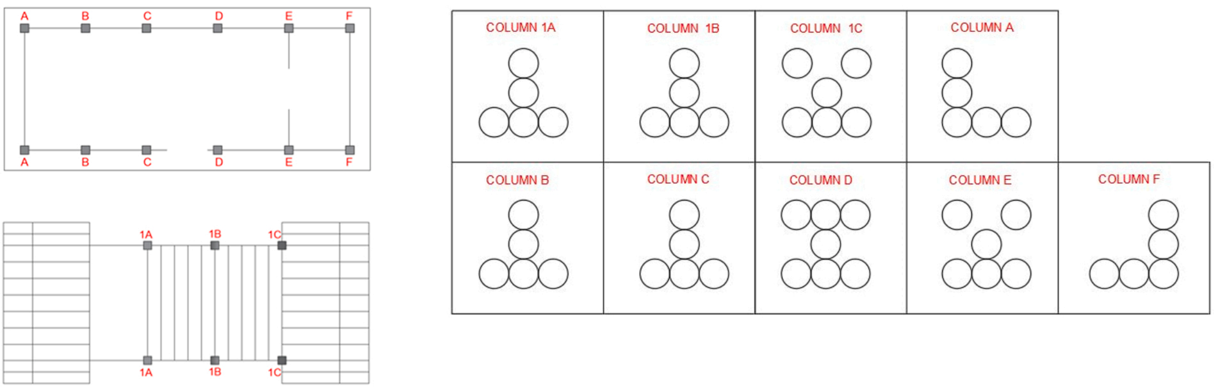

Fig. 4. Cross section of columns.

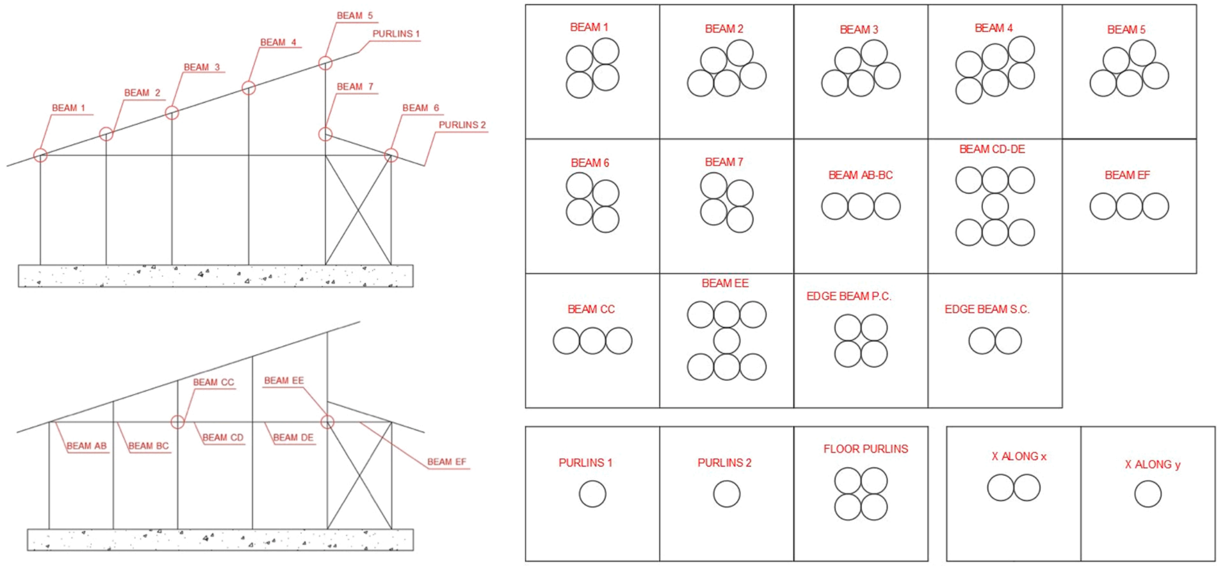

Fig. 5. Cross section of beams.

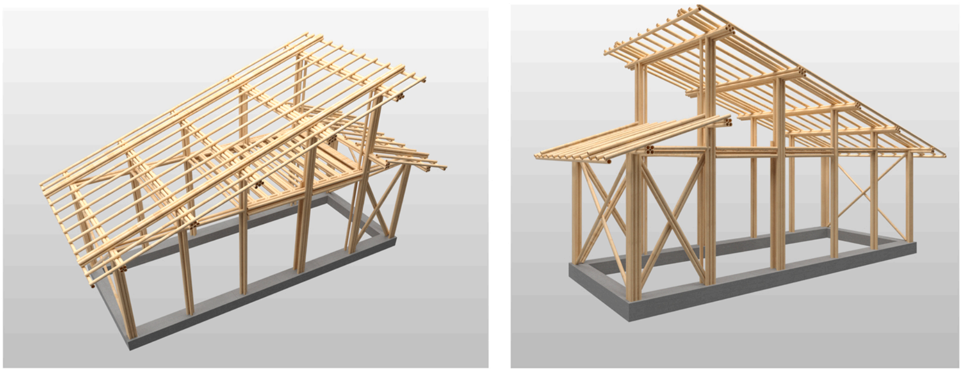

Fig. 6. Rendering of the structure.

while the edge beam has 4 culms. The lower culms of beam 4 are those directly connected with the culms of pillar 1B while the upper outer culms provide a connection between beam 4 and the edge beam. Metal rings were inserted with a pitch of 300 mm in beam 4.

Node 2 connects the beams CD and DE with the columns 1B and D and with the beams of the floor. As it is possible to observe in Fig. 7 the culms of the beams in correspondence of the edges are interrupted near the columns except the central one. The culms of the columns continue up to the roof except the central one. This solution was chosen to ensure continuity of the culms of the column; to minimize the connection while keeping beams of the external frame and of the floor on the same plane. The beams of the floor discharge their forces directly on the column and not on the beams of the external frame. The circumferential bearing capacity verifications are satisfied while for the verification of the bending moment is necessary to insert diagonal elements. It is necessary to introduce the diagonal reinforcement also between the beams of the floor frame and the column D.

Node 3 connects the pillar E and the bracing element with the foundation. The column has 6 culms, and the bracing element has 2 culms. The connection between the pillar culms and the foundation is guaranteed by inserting a threaded bar with a length of 300 mm inside the culm itself with a diameter equal to 14 mm of the terminal part of the culm then filled with concrete. To avoid or reduce any splitting phenomena of the terminal part of the culms, a metal clamp with a wall thickness of 3 mm and a width of 10 mm was inserted together with steel wires.

The lower culm of the bracing element is connected directly to the foundation, while the upper culm is connected through a joint to the central culm of the column.

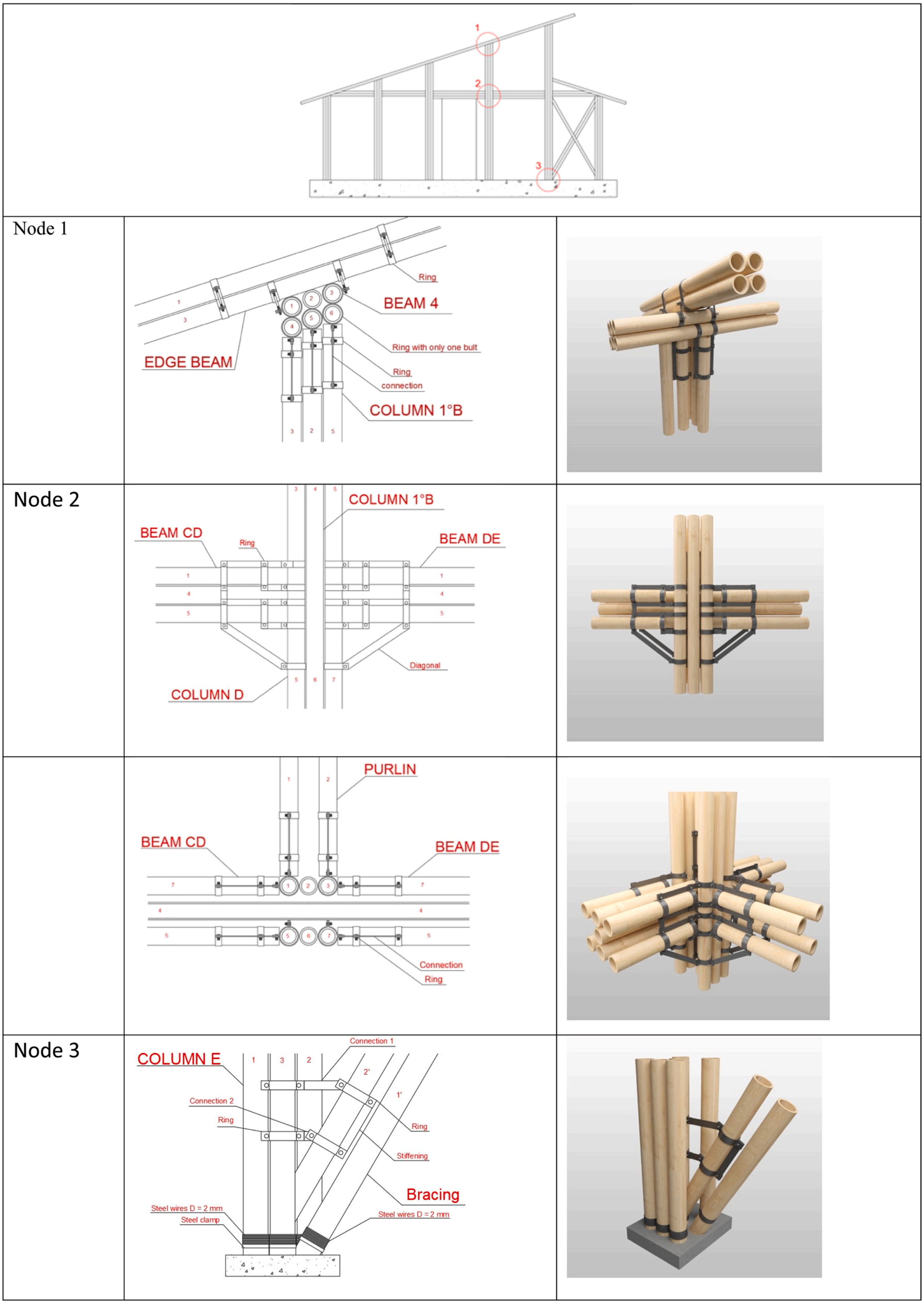

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Fig. 7. Geometry of node 2.

8. Conclusions

The paper shows the feasibility of the design of a building using bamboo culms grown in Italy in compliance with the Italian standard context (NTC2018) supported by the international standard on structural bamboo culms (ISO 22156).

The use of the ISO22516 standard is very important because it regulates the characteristic and design values of bamboo and because it gives indications on the verifications to be performed for culms and connections.

However, the standards show the need to better detail the design in seismic zone or in relation to fire, furthermore it is necessary to increase the material knowledge in respect to the long-term behavior and the durability of the culm. Another crucial and critical point is related to the joints between the elements, mainly the joints not drilled or filled with concrete which are not exhaustively treated.

Another issue that deserves further investigation is the behavior of multi-culms beams. The standard requires that bamboo culms constituting a single structural element are not composite (for example for beams and columns). The moment of inertia of a multi-culm element, therefore, is calculated as the sum of the moments of inertia of the individual culms constituting it, in this way the resulting moment of inertia is considerably smaller.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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