buildings

MDPI

Article

Robustness Analysis of Historical Timber Roofs: A Case Study of the Gaggiandre Shipyard at the Arsenale of Venice

Francesca Ferretti [1], Luca Pozza [1,*] and Diego Alejandro Talledo [2]

[1] Department of Civil, Chemical, Environmental and Materials Engineering, University of Bologna, Viale Risorgimento 2, 40136 Bologna, Italy
[2] Department of Architecture and Arts, University IUAV of Venice, Dorsoduro 2206, 30123 Venezia, Italy
* Correspondence: luca.pozza2@unibo.it

Abstract: In this work, a deterministic approach is adopted to analyze the robustness of the timber roof of the Gaggiandre shipyard at the Arsenale of Venice. The capacity of the traditional timber truss to withstand the design loads as a result of the damage in the strut-tie node is evaluated according to the alternative load path method. Two layouts of the trusses are analyzed and compared: before and after the Austrian retrofitting intervention, performed in the late 1800s. For both configurations, robustness analyses are carried out by using linear 2D numerical models that consider the effective rotational capacity of the structural nodes in relation to the construction methods of the timber joints. For the configuration subject to the 19th-century restoration, the 3D response of the roof is also analyzed to verify the additional contribution provided by the longitudinal bracing system to the activation of alternative load paths (bridge effect). The results obtained with the different analyses are thoroughly evaluated, providing an indication of the deterministic robustness index of the roofing system based on different assumptions. The outcomes of this work allow to draw some general considerations on the method that could be used for the robustness assessment of historical wood systems.

Keywords: timber truss systems; historical buildings; robustness analysis; numerical modeling

Citation: Ferretti, F.; Pozza, L.; Talledo, D.A. Robustness Analysis of Historical Timber Roofs: A Case Study of the Gaggiandre Shipyard at the Arsenale of Venice. *Buildings* **2022**, *12*, 1773. https://doi.org/10.3390/buildings12111773

Academic Editor: Wen-Shao Chang

Received: 6 July 2022
Accepted: 20 October 2022
Published: 22 October 2022

1. Introduction

Robustness against an accidental action, as reported by the CNR Italian Guidelines [1], indicates the ability of a structure to avoid damages disproportionate to the entity of the action which causes an initial damage. The robustness assessment of large span timber roofs is usually performed on newly built roofs to evaluate the effect of local damage to structural joints or elements [2,3]. The methodologies available for the implementation of these assessments on new structures are well defined and consolidated in design practice [1,4,5]. Otherwise, the robustness assessment of historical large span roofs requires specific in-depth studies and a different methodological approach that has not yet been codified and defined in the literature. In fact, it is necessary to acquire an adequate level of knowledge of the structure [6,7], in particular with reference to reinforcement and consolidation interventions that the structure has undergone during its life as they could significantly affect the structural response. In addition, the evaluation of geometry, acting loads, and material properties is more difficult than for new structures, with significant implications on the reliability of the analyses, e.g., Ref. [8].

The present paper proposes a simplified procedure to assess the robustness characteristics of existing timber roof structures, belonging to cultural heritage, which could provide a basis for developing an engineer-oriented method to be used in robustness analyses.

This approach is applied to the very challenging case study represented by the roof structure of the Gaggiandre shipyard in Venice. It consists of two very long sheds, called "Tezoni", built between 1568 and 1573 by Sansovino and belonging to the complex "Arsenale" in Venice, which was the hub of the naval industry of Venice since the beginning of the XII century [9,10]. The Arsenale of Venice is a huge complex of docks and sheds, subject to several restoration and strengthening interventions in the past, performed mainly due to changes in the use of the buildings over the centuries or, more recently, due to material degradation phenomena and damages caused by the lack of maintenance of the complex [11].

For the robustness assessments, a deterministic approach is adopted [5,12] aiming to estimate the ability of the roof to withstand the design loads after the damage of some key structural nodes, according to the Alternative Load Path (ALP) method. The assessments are performed with numerical models capable of considering the actual rotational capacity of the structural nodes in relation to building procedures of the timber carpentry joints.

In this study, the robustness of the roof is evaluated for two layouts of the trusses: the original version and the one subject to Austrian retrofitting works, performed at the end of the 1800s. In both configurations, reference is made to the response of the single truss in its own plane. For the retrofitted case, the 3D system of the entire roof is also considered. The analysis of the obtained results allows us to understand the effects of the 19th-century restoration interventions on the robustness of both the truss and the entire roof, and to define scenarios and configurations to be analyzed with more refined methods to perform probabilistic robustness assessments.

2. The Gaggiandre Shipyard

In the 16th century, the shipbuilding activity in Venice was concentrated in the Arsenale area. The need of the Venetian Senate was to guarantee the regular navigation of the merchant ships by establishing a shipyard aimed at the construction of Galleys. During the expansions of the Arsenale Novissimo, Jacopo Sansovino was the builder of the dry dock called "Gaggiandre" (Figure 1). The Gaggiandre (or Gagiandre) are "two aquatic canopies–only built out of a planned series of three–constituting one of the most significant achievements of the vast sixteenth-century expansion undergone by the Arsenale of Venice" [13].

Figure 1. The Gaggiandre shipyard, Venice (from Google Earth).

The roof of the Gaggiandre shipyard, supported by continuous clay brick masonry walls, consists of a series of composite type timber trusses, with a length of about 25 m, which are among the largest of the 16th century (Figure 2). Figure 3 shows some construction details of the roof, i.e., the suspension of the tie element (Figure 3a) and detail of the support of the truss on the masonry, realized with a stone cantilever and barbican (Figure 3b).

Figure 2. Bottom view of the roof.

(**a**) (**b**)

Figure 3. Construction details of the roof: (**a**) suspension of the tie element; (**b**) support on the masonry by a stone cantilever and barbican.

2.1. Historical Evolution of the Gaggiandre Roof Structure

The current configuration of the trusses of the Gaggiandre roof is the result of the 19th-century static restorations, carried out by Austrian soldiers for structural strengthening purposes, as thoroughly described in Ref. [13]. In more detail, the original truss was reinforced by adding a trapezoid system consisting of lateral King Posts, Struts, and a Horizontal Strut. Figure 4 shows the scheme of the composed truss, highlighting the original configuration and the structural elements added during the restoration. These interventions have significantly modified the static scheme in the plane of the truss, adding a reinforcement substructure and modifying some structural nodes typical of the construction methods of Venetian timber carpentry. Furthermore, globally, a system of diagonal rods has been introduced, which stabilizes the trusses out of their plane and connects them together, serving as a transverse bracing system (Figure 5).

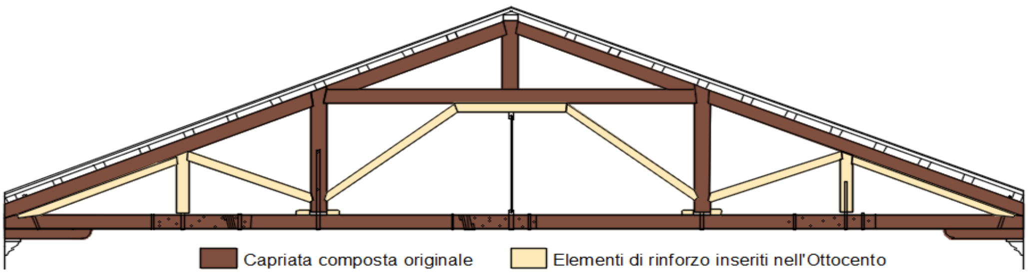
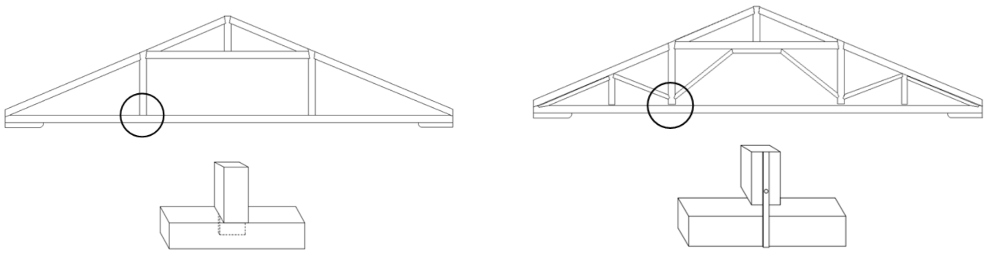


Figure 4. Schematization of the retrofitting intervention.

Figure 5. Detail of the transverse bracing system of the Gaggiandre roof.

With regard to the 19th-century interventions, it is noted that the dimensions of the cross section of the elements belonging to the reinforcement system are not consistent with those of the original elements, which respected the unit of measurement of the Venetian foot (equal to 34.77 cm).

The most significant change made with the retrofitting interventions is the one that involved the elements called King Posts: in the original scheme, these were connected to the ties, most likely using wrapping tie or special joints. Currently, the King Posts appear to be detached from the ties, giving rise to an open King Post-tie connection (Figure 6).



(**a**) (**b**)

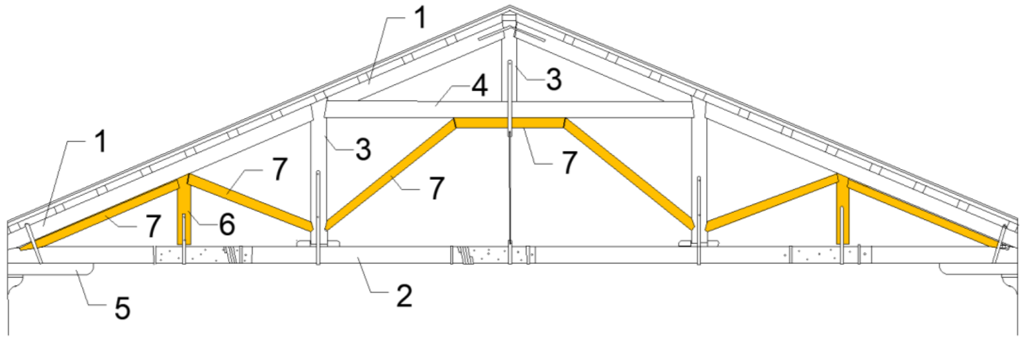
Figure 6. King Post–bottom tie joint configuration: (**a**) original (closed connection type); (**b**) after retrofitting intervention (open connection type).

Furthermore, the addition of the lateral struts has halved the span of the lower struts by significantly reducing the bending action as well as the effective length with reference to the buckling phenomenon in the vertical plane of the truss. The tie in the centerline turns out to be supported by a metal tie rod, connected to the central King Post.

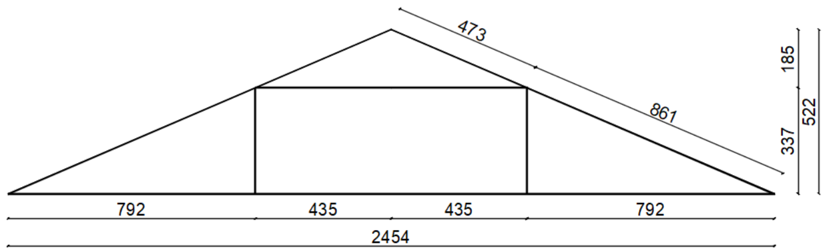
2.2. Geometric Configuration

The two roofs belonging to the Gaggiandre shipyard consist of a truss structure, with a span approximately equal to 25 m, made of struts, ties, and King Posts. The dimensions of the structural elements that compose the truss are reported in Table 1 and follow the ancient Venetian measure called foot, "piede", which corresponds to 34.77 cm and represents the main reference measure of the time. In fact, the tie, the four struts, and the King Posts have a section of about one foot by $\frac{3}{4}$ foot. The bottom tie is divided into three segments joined by particular double step nodes named "Dardo di Giove". The angle between the strut and the bottom tie is equal to 24° with a truss height of about 5 m. The length of the truss elements is reported in Figure 7 both for the original and the retrofitted configuration. The spacing between the trusses is variable, ranging from 206 to 210 cm, equal to about 6 Venetian feet.

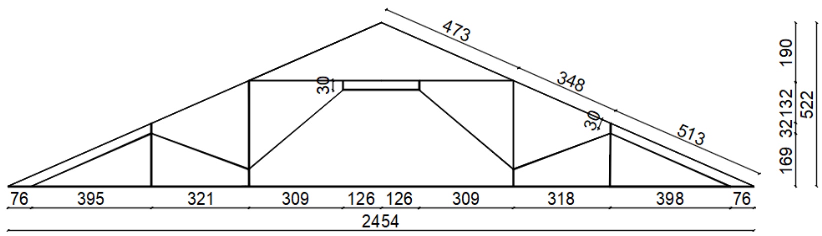
Table 1. Size and nomenclature of the main structural elements.



Roof Truss-Structural Element ID and Nomenclature		Cross Section Dimension B × H [cm]
1	Inclined Strut	26 × 35
2	Tie	26 × 35
3	King Post	26 × 35
4	Horizontal Strut	26 × 35
5	Shelf	26 × 35
6	Additional King Post	26 × 28
7	Additional Strut	26 × 20



(**a**)



(**b**)

Figure 7. Scheme of the roof truss (dimensions reported in cm): (**a**) original configuration; (**b**) retrofitted configuration.

3. Robustness Analysis of the Individual 2D Timber Roof Truss

This section reports the analysis of the single roof truss in the two different configurations, i.e., pre- and post- 19th-century restoration. The 2D linear static analyses are performed with the software "Strand 7" [14].

Preliminarily, the response in the initial configuration of the trusses is analyzed for the vertical loads (self-weight, permanent, and snow loads) and, subsequently, the response of the trusses subject to a strut-tie node damage is analyzed in order to obtain information on the structural robustness. The analyses are performed in a comparative way considering the effect of rotational stiffness of the carpentry structural nodes.

3.1. 2D Model

Two different configurations of the wooden roof trusses, the original (model 1) and the retrofitted (model 2) one, respectively, are analyzed by using 2D models and adopting beam elements (Figure 8). Particular attention is paid to the nodal joints and boundary conditions.

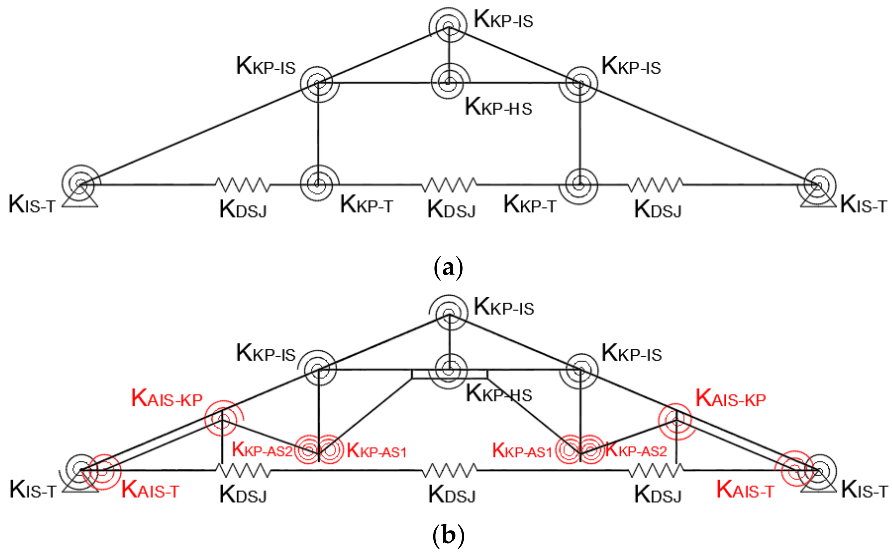


Figure 8. Modeling of the truss accounting for the nodal stiffness: (**a**) 1—original; (**b**) 2—post intervention.

Each element of the roof trusses was modeled according to the geometry of the section derived from the historical surveys given in Table 1. Due to the uncertainties concerning the material characterization, which is typical of existing constructions, especially of the historical ones, the effect of a variability in the timber mechanical properties was considered in the analyses with the objective to evaluate its effects on the robustness characteristics. Therefore, in the following, four different strength classes of the wooden material are assumed, corresponding to the categories C22, C24, C27, and C30, in accordance with EN 14081-2016 [15], and the obtained results are compared in terms of robustness analyses. It is worth mentioning that the same material was considered both for the elements in the pre-intervention configuration and for the elements added with the 19th century restoration. This is justified by the fact that usually, in the retrofitting interventions on historical constructions, materials compatible with the existing ones were adopted [16].

Two comparative modelings of the nodes of the structures were performed: the first neglects the rotational stiffness of the nodes, therefore all the nodes are treated as hinges, while the second (hereinafter referred to as the letter "R") considers the nodal stiffness (Figure 8), the value of which has been adequately calibrated on consolidated results available in literature. In particular, in the more refined modeling, the interactions between the structural elements are defined as follows: (i) the double step-joints of the segments of the bottom tie were modeled as equivalent springs whose axial stiffness is defined in accordance with Refs. [17,18]; (ii) the step-joints of the strut-tie and King Post-strut connections are modeled considering a rotational stiffness defined in accordance with Ref. [19]; (iii) the joint between the bottom tie and the King Post in the original configuration of the truss is modeled considering the rotational stiffness defined in accordance with Ref. [20]; (iv) the interaction between the original struts and the new additional reinforcing struts is modeled with rigid elements (trusses), having an axis perpendicular to the axis of the strut elements in order to reproduce the flexural coupling but allowing for the reciprocal sliding. This modeling criterion is also used to reproduce the interaction between all the horizontal strut elements as well as the tie-barbican connection.

The input parameters considered in the models are reported in Table 2, where information about the mechanical properties is included for each strength class in terms of mean elastic modulus parallel (E_0) and perpendicular (E_{90}) to grain, and in terms of mean shear modulus *G*. In Table 2, the values considered for the nodal stiffnesses are also reported, considering the nomenclature introduced in Figure 8.

Table 2. Input material properties and nodal stiffnesses.

Model	Symbol	Unit	Description	C22	C24	C27	C30
All the models	E_0	MPa	Mean elastic modulus parallel to grain	10,000	11,000	11,500	12,000
	E_{90}	Mpa	Mean elastic modulus perpendicular to grain	6700	7400	7700	8000
	G	Mpa	Mean shear modulus	630	690	720	750
1—Original model with nodal stiffnesses (Figure 8a)	K_{DSJ}	N/m	Axial stiffness double step joint	9.10×10^2	1.00×10^3	1.05×10^3	1.09×10^3
	$K_{IS\text{-}T}$	Nm/rad	Rotational stiffness inclined strut-tie	8.02×10^6	8.93×10^6	9.22×10^6	9.67×10^6
	$K_{KP\text{-}T}$	Nm/rad	Rotational stiffness king post-tie	3.05×10^6	3.41×10^6	3.50×10^6	3.67×10^6
	$K_{KP\text{-}IS}$	Nm/rad	Rotational stiffness king post-inclined strut	2.63×10^6	2.92×10^6	3.02×10^6	4.27×10^6
	$K_{KP\text{-}HS}$	Nm/rad	Rotational stiffness king post-horizontal strut	1.80×10^6	2.00×10^6	2.07×10^6	2.17×10^6
2—Post-intervention model with nodal stiffnesses (Figure 8b)	K_{DSJ}	N/m	Axial stiffness double step joint	9.10×10^2	1.00×10^3	1.05×10^3	1.09×10^3
	$K_{IS\text{-}T}$	Nm/rad	Rotational stiffness inclined strut-tie	8.02×10^6	8.93×10^6	9.22×10^6	9.67×10^6
	$K_{KP\text{-}T}$	Nm/rad	Rotational stiffness king post-tie	0.00	0.00	0.00	0.00
	$K_{KP\text{-}IS}$	Nm/rad	Rotational stiffness king post-inclined strut	2.63×10^6	2.92×10^6	3.02×10^6	4.27×10^6
	$K_{KP\text{-}HS}$	Nm/rad	Rotational stiffness king post-horizontal strut	1.80×10^6	2.00×10^6	2.07×10^6	2.17×10^6
	$K_{AIS\text{-}T}$	Nm/rad	Rotational stiffness additional inclined strut-tie	6.76×10^6	7.52×10^6	7.77×10^6	8.15×10^6
	$K_{AIS\text{-}KP}$	Nm/rad	Rotational stiffness additional inclined strut-king post	2.23×10^6	2.48×10^6	2.56×10^6	2.69×10^6
	$K_{KP\text{-}AS1}$	Nm/rad	Rotational stiffness king post-additional strut 1	4.60×10^5	5.11×10^5	5.28×10^5	5.54×10^5
	$K_{KP\text{-}AS2}$	Nm/rad	Rotational stiffness king post-additional strut 2	6.59×10^5	7.32×10^5	7.57×10^5	7.94×10^5

The models described above are used to obtain the stress state on each structural element for linear static analysis (considering only vertical forces in symmetrical and non-symmetrical configurations) and robustness analysis (in the hypothesis of failure of the strut-tie node due to degradation phenomena). The two different load configurations analyzed in the linear static analysis include the snow load, as follows: for the first configuration (hereinafter referred to as the letter A), the snow loads are applied symmetrically on the two pitches of the roof, neglecting the accumulation phenomenon, which may occur in the concave portion between the two roofs of the Gaggiandre shipyard (Figure 1), while for the second configuration (hereinafter referred to as the letter B) an asymmetrical load condition due to the phenomenon of snow accumulation is considered. The robustness analyses are instead performed considering the combination of the accidental loads defined by the Italian Building Code [21], i.e., with a symmetrical load pattern. For all the configurations examined, the permanent and variable loads were defined and applied in compliance with the Italian Building Code [21].

3.2. Demand–Capacity Ratio (DCR) Evaluation for Undamaged Configurations

In this section, the main results obtained from the linear static analyses (LSA) of the different configurations, in terms of the Demand–Capacity Ratio (DCR) of the element or of the carpentry joint, are reported. In particular, the demand is expressed as the stress component obtained by the internal forces (i.e., bending moment, axial, and shear forces) resulting from LSA, while the corresponding capacity is evaluated by the relationships reported in Section 6 of Eurocode 5 [22]. The safety verification of each element/joint is satisfied if DCR $\leq$ 1. The results of the most relevant verifications are reported in Figure 9 for the different models and loading conditions considered, i.e., original (1) or retrofitted (2) configuration, symmetrical (A) or asymmetrical (B) snow load, accounting for the nodal stiffness (R) or not.

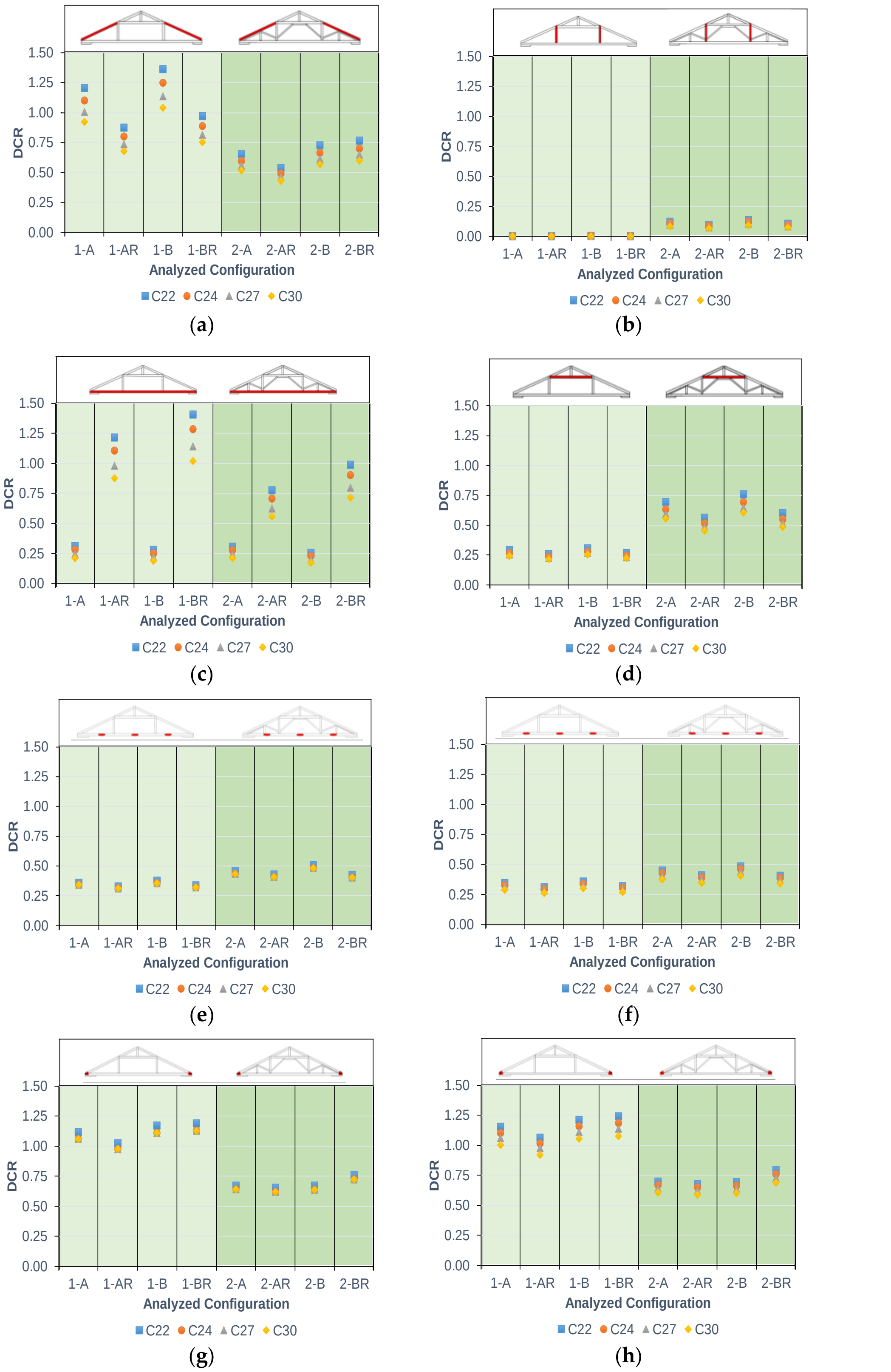


Figure 9. DCR for the most significant structural elements and nodes: (**a**) Strut-bending/compression; (**b**) King Post-traction; (**c**) Bottom tie-bending/traction; (**d**) Strut-bending/compression; (**e**) "Dardo di Giove" joint-shear; (**f**) "Dardo di Giove" joint-inclined compression; (**g**) Step joint-shear; (**h**) Step joint-inclined compression.

It can be observed that in the original configuration some elements and joints of the truss are not verified (DCR > 1), even considering a high strength class timber.

The 19th-century reinforcement intervention has resulted in different stress distributions on the elements of the truss, which show a DCR less than 1, even for the lowest timber strength classes. In particular, it can be observed that: (i) the reinforcement system added during the 19th-century retrofitting interventions reduces the effective length of the lower strut; (ii) in the original configuration, the King Post has a low level of stress, while in the retrofitted configuration it shows a traction state of stress; (iii) the horizontal strut is more stressed after the retrofitting intervention due to the redistribution of the stresses that involves the lower struts system; (iv) the DCR of the step joint in the main strut-tie connection exceeds 1 in the original configuration, except for the cases in which high strength timber classes and semi-rigid node are considered, while the addition of an inclined strut and the metal support allow for a better redistribution of stresses reducing the DCR of the joint; (v) the DCR of the "Dardo di Giove" joint is less than 1 in all the considered configurations; (vi) the asymmetric load condition implies, in general, a greater DCR of the structural elements for both the considered configurations.

3.3. Demand–Capacity Ratio (DCR) Evaluation for Damaged Configurations and Robustness Analysis

The robustness analysis is conducted using a deterministic approach [5] in order to verify the ability to redistribute the stresses inside the truss as a result of the damage to a structural element or node. The adopted procedure verifies the possibility of activating an alternative load path in the plane of the truss.

The damage mechanism taken into consideration in the robustness analyses consists in the shear failure of the step joint between the main strut and the bottom tie, since this is actually the most significant mechanism for this type of structure. Indeed, on the one hand, this node is highly stressed as a result of the static analysis, and, on the other hand, experience demonstrates that trusses typically exhibit collapse phenomena due to the damage of this node, which is often caused by degradation phenomena (e.g., infiltration of water from the roof covering) or constructional defects.

In the numerical model, the shear failure of the step joint between the main strut and the bottom tie is simulated by unlocking the axial translation of the strut so that the transfer of axial forces between the bottom tie and the principal strut is no longer possible. In this damaged configuration, the bottom tie works mainly as a beam over a span of 25 m while all the other elements participate in the bearing mechanism only after they have come into contact with the bottom tie itself. This aspect is particularly significant for the 19th-century retrofitted configuration, in which the King Post-tie joint was modified by creating a gap between the elements. To take this aspect into account, the analyses for the 19th-century configuration are carried out assuming the King Post resting in support on the tie but applying a dynamic amplification factor of the forces, here assumed equal to 1.5 according to Ref. [5]. Figure 10 shows the schematic deformed shapes of the models used for the robustness analyses in the two configurations of the roof truss.

Figure 11 shows the results of the verifications of the elements that are found to be the most stressed in the robustness analyses, where an accidental load combination is considered, for both the original and post 19th-century intervention configurations (the analyses with dynamic amplification are identified with the label "Dyn" in the graphs). It is observed that both configurations of trusses are not robust in their plane. In particular, for all configurations, the bottom tie is not verified due to the excessive bending stress (which particularly exploits the sections weakened by "Dardo di Giove" joints) even if with lower DCR for the original configuration. With regard to the strut, it can be observed that the verification against bending/compression is satisfied in the original configuration, while it is not satisfied in the retrofitted configuration due to the bearing mechanism activated by the reinforcing trapezoid substructure.

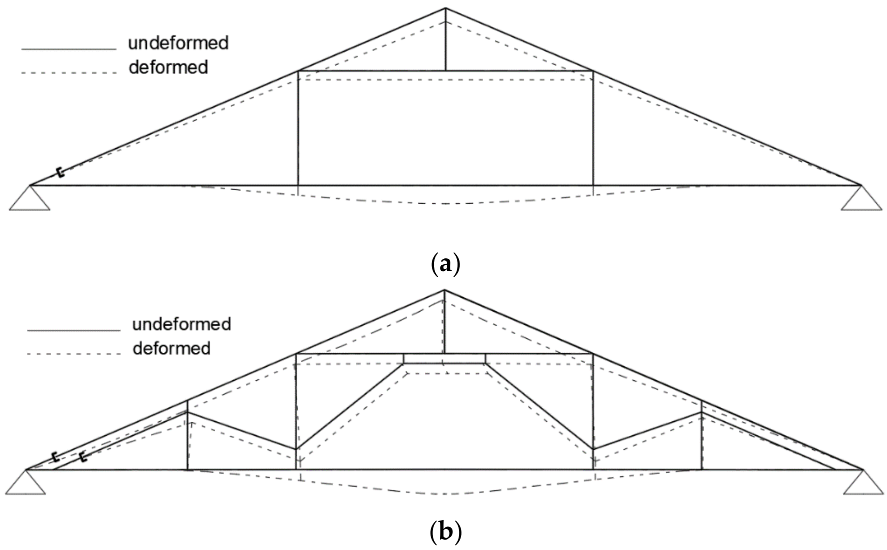


Figure 10. Deformed configuration of the roof truss: (**a**) 1—original; (**b**) 2—post intervention.

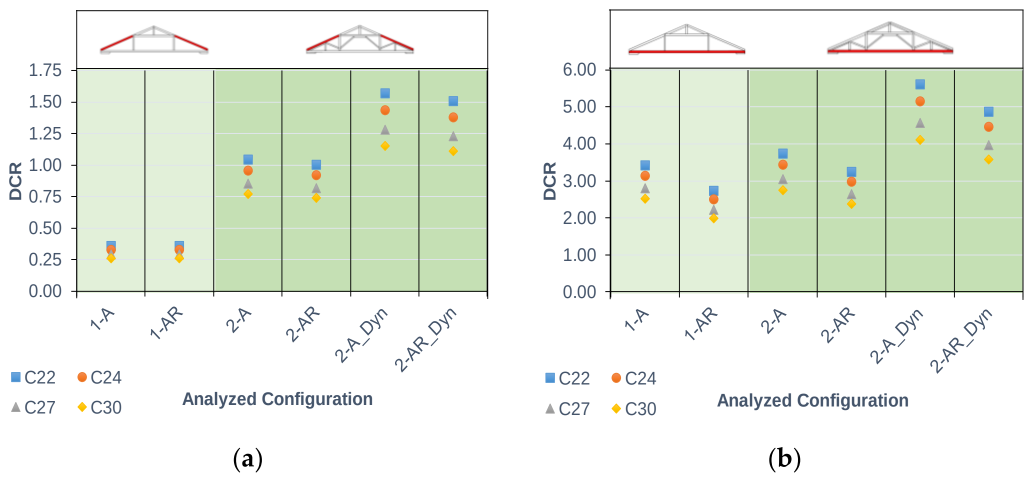


Figure 11. Robustness analysis—DCR for the most significant structural elements: (**a**) Strut-bending/compression; (**b**) Bottom tie-bending/traction.

The models that consider the nodal stiffness provide lower values of the DCR for the structural elements, as a demonstration that the degree of interlocking provided by the carpentry joints contributes positively to the overall strength of the truss.

Finally, when considering the dynamic effects caused by the fact that, in the retrofitted configuration, the bearing mechanism can only be activated after the contact between the King Post and the bottom tie, it can be, in general, concluded that the 19th-century retrofitting intervention did not improve the robustness of the truss in its plane. The "closed" type King Post-bottom tie typical of the Venetian carpentry appears to be particularly effective in giving overall robustness to the truss.

The results of the analyses are then processed for the purpose of defining the robustness indexes RI reported in Table 3. They are evaluated, for each timber strength class, as the Robustness Index (RI), defined as the ratio between the maximum DCR for the undamaged configuration and the maximum DCR for the damaged configuration (RI = max $DCR_{undamaged}$/max $DCR_{damaged}$) [23].

Table 3. Robustness Indexes (RI) for the different analyzed configurations.

Robustness Index (RI)		C22	C24	C27	C30
1-A	$DCR_{undamaged}$	0.79	0.72	0.65	0.60
	$DCR_{damaged}$	3.43	3.14	2.79	2.51
	RI	0.23	0.23	0.23	0.24
1-AR	$DCR_{undamaged}$	0.57	0.52	0.48	0.44
	$DCR_{damaged}$	2.72	2.49	2.22	1.99
	RI	0.21	0.21	0.22	0.22
2-A	$DCR_{undamaged}$	0.51	0.46	0.41	0.36
	$DCR_{damaged}$	3.74	3.43	3.05	2.74
	RI	0.14	0.13	0.13	0.13
2-AR	$DCR_{undamaged}$	0.35	0.32	0.30	0.28
	$DCR_{damaged}$	3.25	2.98	2.65	2.38
	RI	0.11	0.11	0.11	0.12
2-A_Dyn	$DCR_{undamaged}$	0.51	0.46	0.41	0.36
	$DCR_{damaged}$	5.61	5.14	4.57	4.11
	RI	0.09	0.09	0.09	0.09
2-AR_Dyn	$DCR_{undamaged}$	0.35	0.32	0.30	0.28
	$DCR_{damaged}$	4.87	4.46	3.97	3.57
	RI	0.07	0.07	0.08	0.08

From the results it is possible to observe that the original configuration showed higher RI than the post-intervention configuration. It is worth noting that the contribution of the rotational stiffness of the joints does not significantly affect the RI since for both undamaged and damaged cases the DCR reduces, as expected. Nevertheless, the 2D analyses does not account, in the post intervention configuration, for the effects of the transversal bracing system and, therefore, of the possible activation of 3D alternative load path resistant mechanisms.

4. Robustness Analysis of the 3D Timber Roof Truss

In this section, the 3D robustness analysis of the Gaggiandre truss timber roof is presented. Only the configuration subject to the 19th-century retrofitting intervention is analyzed in order to evaluate the effect of the longitudinal bracing system, inserted during the restoration itself, on the overall structural robustness. In particular, the purpose is to verify whether the longitudinal bracing elements allow, in the event of a local damage to a truss, the activation of a 3D "alternative load path" (ALP) type bearing mechanism or if they introduce a progressive collapse of the entire roof.

4.1. 3D Model

The 3D FE model is implemented by reproducing the actual geometry of the roofing system, composed by 18 trusses arranged parallel with a distance of about 210 cm. The end portion of the roof, having a "pavillon" type structure on one side, is not directly modeled as it is not of interest for the purposes of robustness assessments. Even so, the degree of constraint provided by this roofing portion has been considered, in particular with regard to retention in the longitudinal direction provided by the triangular conformation of the pitches. The trusses are modeled with the same criteria adopted for the analyses in the 2D model, also considering the rotational stiffness of the individual structural nodes. The longitudinal bracing system, consisting of a double alignment of diagonals that intercept four trusses each, is modeled by means of truss elements rigidly connected to the structural nodes of the trusses. The system of pitch purlins has been neglected as they are simply supported on the roof trusses and are, therefore, irrelevant for evaluation of the 3D robustness of the system. The input parameters are the same used for the 2D models and reported in Table 2. Figure 12 shows the 3D model of the Gaggiandre truss timber roof, with an indication about the number of trusses.

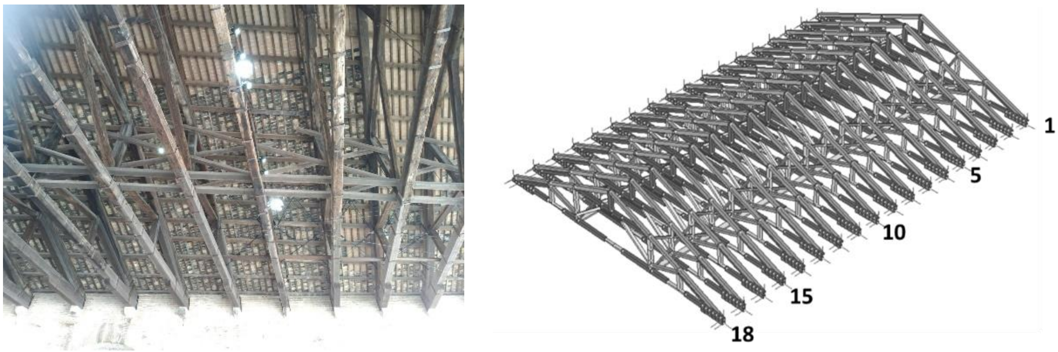


Figure 12. 3D model of the Gaggiandre truss timber roof.

4.2. Multi-Failure Analysis

The robustness analysis is performed by considering three different Local Failure Scenarios (LFS), typical of historical roof systems, all of them concerning the damaging and subsequent failure of the lateral node of one of the timber trusses, i.e., the step joint between the main strut and the bottom tie:

1. LFS1—Shear failure of the strut-tie joint: this is the same failure mode considered in the 2D analyses, in which the support of the truss is guaranteed but the ability to transfer the axial force between the bottom tie and the strut is lost, and the bottom tie results are inflected;
2. LFS2—Failure of the support barbican: in this case, the truss remains functional and assembled but it loses its support on the wall;
3. LFS3—Failure of the support node and of the strut-tie joint: this situation corresponds to the combination of the two previous failure scenarios.

These scenarios are compatible with the main degradation phenomena occurring over time, i.e., the degradation of the end part (at the support zone) of the roof system due to water infiltration.

In the analyses, the damage is considered to be localized on truss nr. 5 (Figure 12), chosen so as not be too close to the extremities of the roof structure and to limit the number of trusses involved in the potential redistribution mechanism. Indeed, the longitudinal bracing system is composed of diagonal elements crossing four trusses (i.e., from the bottom chord of the first truss to the top chord of the fourth truss) and anchored on them. The three LFS are schematized in Figure 13.

4.3. Robustness Analysis

For the three failure scenarios, the variation of the axial force on the main elements of the roof trusses is evaluated with respect to the undamaged configuration. All the analyses and related structural checks are carried out considering the lowest timber strength class, i.e., C22 in accordance with EN 14081 [15]. As an example, Figure 14 shows the percentage changes, calculated as the ratio between the axial force in the damaged and the undamaged configuration, related to the inclined struts and the bottom tie for all the roof trusses.

The results show that the trusses next to the damaged one evidence a significant increase in the axial forces. This increase is reduced by moving away from the damaged truss; the furthest trusses are practically not affected by any variation in the axial force; thus, they do not participate in the mechanism of redistribution of the loads.

To verify the possibility of activating an ALP-type bearing mechanism, the maximum DCR of the trusses that are the most stressed (i.e., trusses 4 and 6) is evaluated since they are next to the damaged truss. The results are reported in Figure 15 for the different LFS, in terms of DCR of the inclined struts and bottom tie for trusses 4 and 6. For both the elements, the values of DCR are always lower than 1 for all the considered LFS.

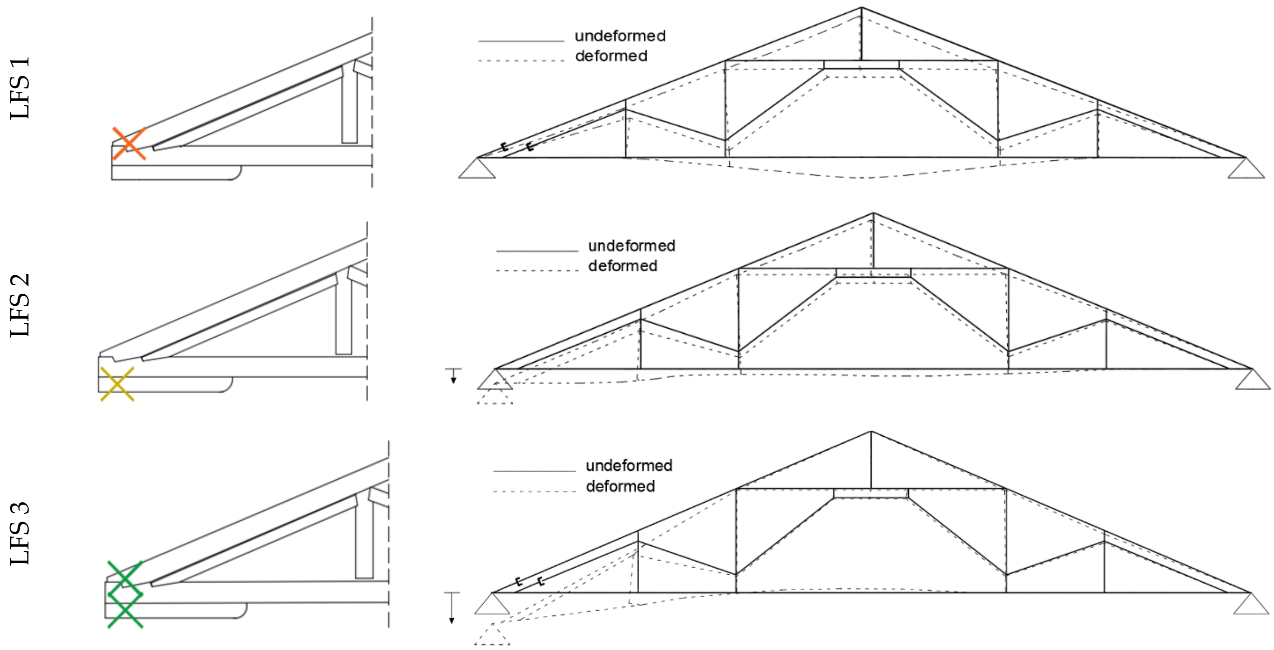


Figure 13. Scheme of the LFS considered in the robustness analyses and associated schematic deformed configurations.

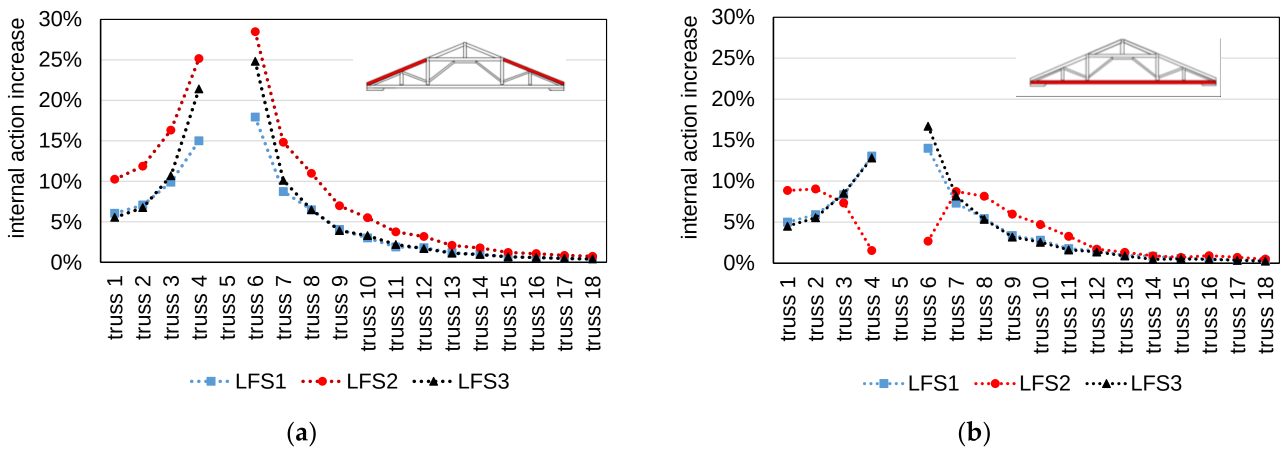


Figure 14. Variation of the internal actions on the main elements of trusses for the different LFS: (**a**) inclined struts; (**b**) bottom tie.

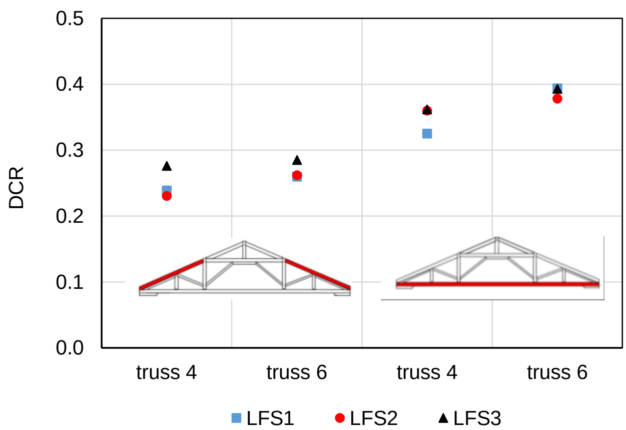


Figure 15. DCR of the struts and of the bottom tie of trusses 4 and 6 for the different LFS.

The results of the verifications show that, although there is a significant increase in the DCR of the elements of the trusses with respect to the undamaged configuration, the elements are always verified. The bracing system is also suitable to withstand the actions induced by the local damage to a truss, with DCR always lower than 1. The LFS3 turns out to be the most severe scenario for the bracing system as it must compensate for both failure modes, i.e., internal disassembly phenomena and loss of support of the damaged truss.

For the worst configuration examined, the corresponding robustness index RI, calculated as reported in Section 3.3, is equal to 0.88. Therefore, a significant robustness increase is registered, due to the beneficial effects of the transversal bracing system, allowing for the activation of a 3D alternative load path resistant mechanism. Indeed, the RIs, which estimated neglecting the 3D behavior, were equal to 0.21 and 0.11 for the original and the retrofitted configurations, respectively (Table 3).

The obtained results allow us to state that the roofing system is able to absorb an increase in internal actions due to a localized failure of a truss and that it can therefore be considered robust as it is capable of activating an alternative load path, avoiding a progressive failure of the roof.

5. Conclusions

In this work, a simplified method, based on a deterministic approach, for the assessment of the robustness of existing timber roof structures is applied to a case study of the Gaggiandre shipyard at the Arsenale in Venice. The analyses are carried out considering two structural configurations of the trusses composing the roof, i.e., the ones before and after the Austrian retrofitting intervention, performed in the late 1800s. Moreover, the robustness assessment is performed both on 2D models, i.e., considering the in-plane behavior of the single trusses, and on a 3D model of the entire roof, taking into account the presence of the bracing system. In the analyses, the variability of the mechanical properties of the timber elements, which represent a significant uncertainty when dealing with existing buildings, is taken into account by performing the structural analyses considering four different timber classes. In addition, the carpentry joints of the trusses are modeled both as hinges and by considering their proper rotational stiffness. The results obtained show that both the pre- and post-intervention configurations do not guarantee structural robustness if reference is made to the behavior in the plane of the single truss. In fact, the 19th-century intervention has not improved the robustness in the truss plane due to the fact that the resistant configurations can be activated only when the King Post comes into contact with the bottom tie, with consequent dynamic effects. A positive impact on the overall response of the truss, both in operation and in accidental conditions, was observed when considering the appropriate nodal stiffness of the carpentry joints.

The 3D analysis of the roof allows us to consider the effect of the transverse bracing system in the evaluation of the structural strength. Different scenarios of local failure of one single truss are considered, and the possibility of activation of an Alternative Load Path type redistribution mechanism is verified. The results confirm that the bracing system is effective and that the trusses are also able to withstand the increase in stresses due to the redistribution of the actions as a result of a local damage. It can therefore be concluded that the roof system, in its current configuration, is robust, i.e., no progressive failure happens if a local damage to a truss occurs.

On the basis of the obtained results, the method applied here and adapted to the case study of the Gaggiandre shipyard could be used as a simplified engineering-oriented tool to assess the robustness characteristics of historical wooden roof systems.

Author Contributions: Conceptualization, L.P. and D.A.T.; methodology, L.P. and D.A.T.; software, L.P. and F.F.; investigation, F.F.; writing—original draft preparation, L.P.; writing—review and editing, F.F. and D.A.T. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Some or all data presented in this study are available from the corresponding author upon reasonable request.

Acknowledgments: Special thanks are due to Mario Piana for his valuable contribution in the assessment of geometry, constructive methodologies, and intervention phases of the Gaggiandre shipyard roof.

Conflicts of Interest: The authors declare no conflict of interest.

References

1. National Research Council (CNR) (Ed.) *CNR-DT 214/2018 Guide to Design of Structures for Robustness*; National Research Council (CNR): Rome, Italy, 2018.
2. Sørensen, J.D.; Dietsch, P.; Kirkegaard, P.H.; Köhler, J. (Eds.) *Guideline—Design for Robustness of Timber Structures: COST Action E55 "Modelling of the Performance of Timber Structures"*; Shaker Verlag: Aachen, Germany, 2011; ISBN 978-3-8322-9949-1.
3. Dietsch, P. Robustness of Large-Span Timber Roof Structures—Structural Aspects. *Eng. Struct.* **2011**, *33*, 3106–3112. [CrossRef]
4. Ellingwood, B.; Smilowitz, R.; Dusenberry, D.; Duthinh, D.; Lew, H.; Carino, N. *Best Practices for Reducing the Potential for Progressive Collapse in Buildings*; NIST Interagency/Internal Report (NISTIR); National Institute of Standards and Technology: Gaithersburg, MD, USA, 2007.
5. Robustness of Structures – COST Action TU0601. Available online: https://www.cost-tu0601.ethz.ch/Documents/Final%20Report/COST_TU0601_structural_robustness_design_practising_engineers_Version1_2-11Sept11.pdf (accessed on 1 July 2022).
6. Ministry for Cultural Heritage and Activities. *Guidelines for Evaluation and Mitigation of Seismic Risk to Cultural Heritage*; Moro, L., Neri, A., Eds.; Gangemi: Rome, Italy, 2007; ISBN 8849212690.
7. Berto, L.; Doria, A.; Faccio, P.; Saetta, A.; Talledo, D. Vulnerability Analysis of Built Cultural Heritage: A Multidisciplinary Approach for Studying the Palladio's Tempietto Barbaro. *Int. J. Archit. Herit.* **2017**, *11*, 773–790. [CrossRef]
8. Rodrigues, L.G.; Aranha, C.A.; Sousa, H.S. Robustness Assessment of an Ancient Timber Roof Structure. *Lect. Notes Civ. Eng.* **2016**, *1*, 447–457. [CrossRef]
9. Bellavitis, G. *L'Arsenale Di Venezia, Storia Di Una Grande Struttura Urbana*; Cicero Editore, Ed.: Venice, Italy, 2007.
10. Bettiol, G.; Valluzzi, M.R.; Garbin, E.; Modena, C. Structural Analysis of the Timber Roofs of the "Arsenale" of Venice. In Proceedings of the 10th World Conference on Timber Engineering, Miyazaki, Japan, 2–5 June 2008.
11. Valluzzi, M.R.; Bondì, A.; Da Porto, F.; Franchetti, P.; Modena, C. Structural Investigations and Analyses for the Conservation of the 'Arsenale' of Venice. *J. Cult. Herit.* **2002**, *3*, 65–71. [CrossRef]
12. Baker, J.W.; Schubert, M.; Faber, M.H. On the Assessment of Robustness. *Struct. Saf.* **2008**, *30*, 253–267. [CrossRef]
13. Piana, M. Il Consolidamento Della Carpenteria Lignea Delle Gaggiandre Nell'Arsenale Di Venezia. TEMA. *TEMPO Mater. Archit.* **1994**, *2*, 5–13.
14. Strand7 Software Verification Manual Release 2.4.6. Available online: https://www.strand7.com/downloads/Strand7%20R246%20Verification%20Manual%20TOC.pdf (accessed on 25 June 2022).
15. *EN 14081-1*; Timber Structures—Strength Graded Structural Timber with Rectangular Cross Section—Part 1: General Requirements. European Committee for Standardisations: Brussels, Belgium, 2016.
16. Scotta, R.; Trutalli, D.; Marchi, L.; Pozza, L. Seismic performance of URM buildings with in-plane non-stiffened and stiffened timber floors. *Eng. Struct.* **2018**, *167*, 683–694. [CrossRef]
17. Drdacky, M.F.; Walcf, F.; Mares, J. Modelling of Real Historic Timber Joints. *Trans. Built Environ.* **1999**, *39*, 171. [CrossRef]
18. Parisi, M.A.; Piazza, M. Mechanics of Plain and Retrofitted Traditional Timber Connections. *J. Struct. Eng.* **2000**, *126*, 1395–1403. [CrossRef]
19. Parisi, M.A.; Cordié, C. Mechanical Behavior of Double-Step Timber Joints. *Constr. Build. Mater.* **2010**, *24*, 1364–1371. [CrossRef]
20. Branco, J.M.; Descamps, T. Analysis and Strengthening of Carpentry Joints. *Constr. Build. Mater.* **2015**, *97*, 34–47. [CrossRef]
21. Ministry of Infrastructures and Transportations NTC. *Aggiornamento Delle «Norme Tecniche per Le Costruzioni»*; Ministry of Infrastructures and Transportations NTC: Rome, Italy, 2018. (In Italian)
22. *EN 1995-1-1*; Eurocode 5: Design of Timber Structures—Part 1-1: General—Common Rules and Rules for Buildings. European Committee for Standardisation: Brussels, Belgium, 1995.
23. Sørensen, J.D. Framework for Robustness Assessment of Timber Structures. *Eng. Struct.* **2011**, *33*, 3087–3092. [CrossRef]