Historic roof structures: life-cycle assessment and selective maintenance strategies

E. Garavaglia
Politecnico di Milano, Department of Civil and Environmental Engineering, Milan, Italy

N. Basso
Waseda University, School of Creative Science and Engineering, Department of Architecture, Tokyo, Japan

L. Sgambi
Université catholique de Louvain, Faculty of Architecture, Architectural engineering and Urbanism, Louvain-la-Neuve, Belgium

ABSTRACT: Many industrial and military constructions built between the end of the IXX century and the first years of the XX century show long span roof structures constructed using slender steel truss members. This characteristic makes the structures almost invisible and the interior space appears larger than it is. After 100 years or more, structures like these are still in good conditions although they may have fallen into disuse and neglect. Usually these buildings are protected by Cultural Heritage regulations. Therefore, it would be interesting to define suitable intervention strategies to maintain, conserve and possibly reuse them. This paper compares three different steel roof trusses, covering some buildings of a military installation in Pavia (Northern Italy), built from the end of the IXX century. The residual capacity has been probabilistically evaluated in order to check the efficiency of the most vulnerable members of the structures and investigate systems' robustness in case of sudden failure of the critical components. The method proposed can be considered as a reliable approach to recognise which structural element is most like to fail, and schedule proper selective maintenance interventions.

1 INTRODUCTION

In the last two decades there has been growing interest in the conservation and adaptive-reuse of disused military and industrial buildings (Butterfield, 1994, Neaverson & Palmer, 1998, Lambert Surendra & Gupta, 2005, Wang & Nan, 2007).

Especially in Europe, the historic heritage includes several good examples of rehabilitation and reuse of industrial archaeology buildings (Houser, 2001, Garcia & Ayuga, 2007, Madgin, 2010).

When it comes to transform old constructions for a new or different use, social and economic factors as well as preservation and structural reliability levels must be deeply investigated (Cigni, 1978, Avramidou, 1990, Baruchello & Assenza, 2004, Binda et al., 2011). Whenever those aspects are not well considered and analysed, there is no possibility to define proper, successful intervention scenarios.

Many industrial and military constructions built between late 19th and early 20th century show long span roof structures with slender steel truss members. This characteristic makes the structures almost invisible and the inner space appears larger than it is. Despite disuse and negligence, after 100 years or more, these structures are still in good conditions. Because of their architectural and historic relevance, these buildings are usually protected by Cultural Heritage regulations. In this regard, three examples are here introduced and analysed in order to evaluate their residual reliability and robustness.

A Monte Carlo simulation implemented with a damage law has been applied to investigate, member by member, the current structural reliability of three roof truss systems of military buildings in Pavia, Northern Italy (Garavaglia et al. 2016, 2018).

Once the members most damaged were identified, a sudden collapse simulation has been run to estimate the structural response associated with each of those members' failure, and evaluate the residual load-bearing capacity in terms of structural robustness. This analysis provides an initial overview on where and when maintenance actions must be performed in order to extend the structural lifespan and ensure its possible reuse responding to an acceptable safety level.

2 CASE STUDIES

The Arsenal complex in Pavia (Northern Italy) covers a wide area in the city centre, next to Rossani barracks. Most of the buildings in the Arsenal are basilica-type masonry constructions with timber-and-iron Polonceau roof system and tile roof covering. They were built between 1865 and 1900.

The complex was significantly expanded during the two World Wars, but neither the late 19th century asset nor the original structural identity changed despite the addition of new RC constructions.

Since disuse in 2010, the Arsenal area has become State property, and because of its architectural and historical significance, it has been protected by Cultural Heritage Authority.

The strategic position and the extent of the site (75,000 sm, with a floor area of 25,000 sm) have attracted several rehabilitation and reuse proposals.

The three case studies analysed here refer to the roof structures of two pavilions of the Arsenal complex and one pavilion of Rossani barracks (Fig. 1).

As well as the Arsenal buildings, Rossani barracks (disuse, 1992) are protected by cultural heritage regulations. The complex is located on the monastery and basilica of St Salvatore’s area (1500 aC). Some expansion has been made over the centuries. In particular, Case study 3 building was added to the complex at the same time of the Arsenal construction.

Damage level, residual mechanical property and structural performance investigations are necessary to evaluate the current condition of the roof truss elements.

All three case studies have in common a very light structure, despite a sometimes significantly wide span.

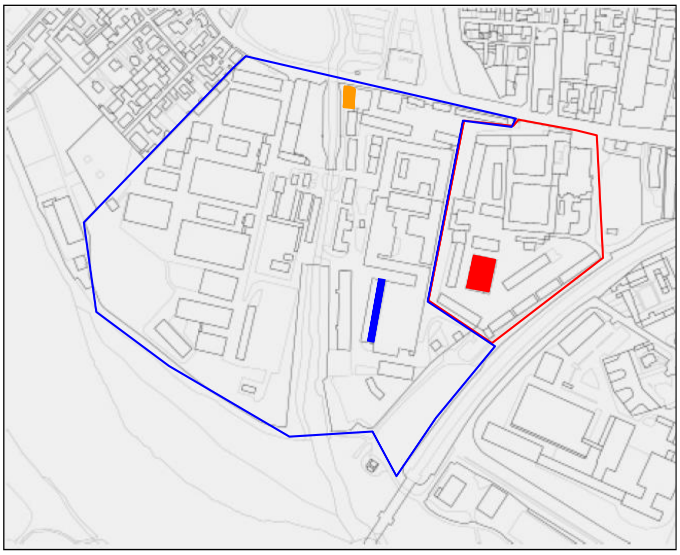


Fig. 1 Arsenal (blue perimeter) and Rossani barrack complex (red perimeter) planimetry in Pavia. Case study 1, Rossani’s Pavilion in red; Case study 2, Arsenal Building 65 in yellow; Case study 3, Arsenal Building 15 in blue.

2.1 *Case 1 Pavilion in Rossani barracks*

The case study structure is a Polonceau truss made by small tension and compression iron and timber bars (Fig. 2; Tab. 1).

The original load on the structure has been estimated as approximately 9.7 kN/m^2 on a 60-sm influence area. Figure 3 shows the truss geometry and the forces applied to each node. Monte Carlo simulation refers to the static model in Figure 3(b). Members 1-4 in timber and members 5-11 in iron have different geometries.

Figure 2 Pavilion in Rossani barracks.

Tab. 1 Case 1) Geometry and mechanical properties of truss elements

Member	Cross section	Length L_0 cm	Area A_0 cm^2	Vol. V_0 cm^3	Inertia I_0 cm^4
1, 4	□□	624.00	400.0	249600	13333.3
2, 3	□□	548.00	400.0	219200	13333.3
5, 7	O	667.00	19.15	12773.1	30.55
6	O	650.00	19.15	12447.5	30.55
8, 11	O	236.00	37.60	8873.6	117.81
9, 10	O	596.00	19.15	11413.4	30.55

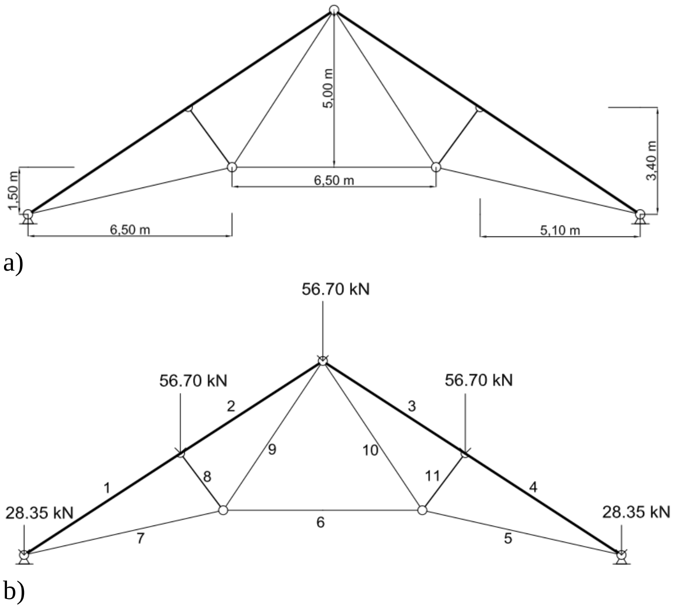


Fig. 3 (a) Truss geometry (in metres); (b) Truss static model

2.2 *Case 2 Arsenal Building 65*

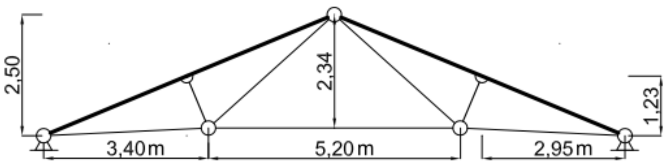
The structure is a Polonceau truss made by small tension and compression iron and timber bars (Fig. 4; Tab. 2).

The original load on the structure has estimated to be approximately 2.0 kN/m^2 on a 42-sm influence area. Figure 5 shows the truss geometry and the forces applied to each node. Monte Carlo simulation refers to the static model in Figure 5(b). Members 1-4 in timber, and members 5-11 in iron present different geometries.

Figure 4 Arsenal Building 65

Table 2 Case 2) Geometry and mechanical properties of truss elements

Member	Cross section	Length L_0 cm	Area A_0 cm^2	Vol. V_0 cm^3	Inertia I_0 cm^4
1, 4	□□	320.00	400.0	128000	13333.3
2, 3	□□	330.00	400.0	132000	13333.3
5, 7	O	340.40	6.91	2349.4	3.98
6	O	520.00	6.91	3593.2	3.98
8, 11	O	116.00	37.60	4361.6	117.81
9, 10	O	350.00	6.91	2418.5	3.98



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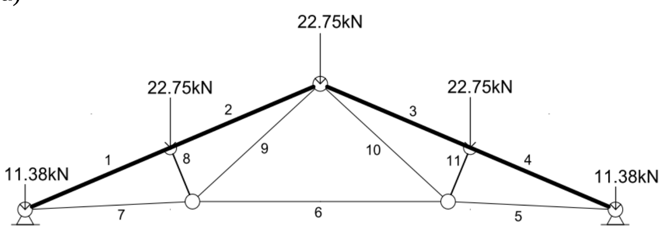


b)

Figure 5 (a) Truss geometry (in metres); (b) Truss static model

2.3 *Case 3 Arsenal Building 15*

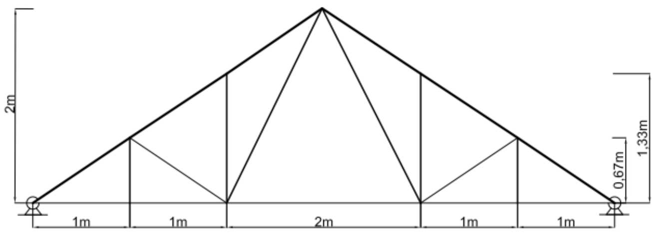
The structure is composed by a classic truss with small tension and compression iron bars (Fig. 6; Tab. 3).

The original load on the structure was about 2.0 kN/m^2 on an influence area of 35 m^2. Figure 7 shows the truss geometry and the forces applied to each node. Monte Carlo simulation refers to the static model in Figure 7(b).

Figure 6 Arsenal Building 15

Table 3 Case 3) Geometry and mechanical properties of truss elements

Member	Cross section	Length L_0 cm	Area A_0 cm^2	Vol. V_0 cm^3	Inertia I_0 cm^4
1-6	O	120.00	19.15	2298.00	30.55
7,8,10,11	O	100.00	6.09	609.00	3.97
9	O	200.00	6.09	1218.00	3.97
12,19	O	67.00	12.25	820.75	12.51
13,18	O	120.40	3.07	369.63	0.785
14,17	O	133.00	12.25	1629.25	12.51
15,16	O	224.00	12.25	2744.00	12.51



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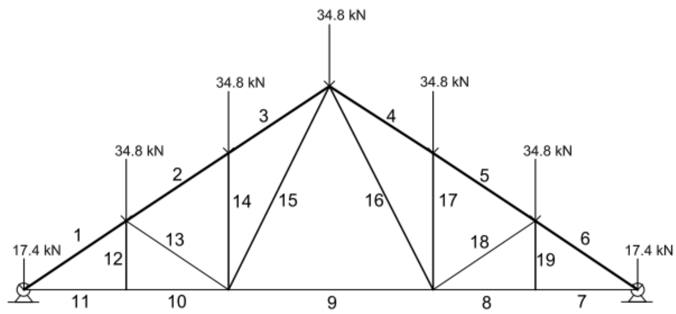


b)

Figure 7 (a) Truss geometry (in metres); (b) Truss static model

3 MODEL AND SIMULATION

According to Biondini et al. (2008), structures and their material properties are naturally affected by a time-dependent deterioration process:

$$\Theta(t) = \Theta_0[1 - \delta(t)] \tag{1}$$

where $\Theta(t)$ is a generic material property, Θ_0 is the undamaged material property at initial time, $\delta = \delta(t) \in [0;1]$ describes the time-dependent deterioration process using the law (Biondini et al. 2008):

$$\delta(t) = \begin{cases} \omega^{1-\rho}\tau^{\rho} & , \quad \tau \leq \omega \\ 1-(1-\omega)^{1-\rho}(1-\tau)^{\rho} & , \quad \omega < \tau < 1 \\ 1 & , \quad \tau \geq 1 \end{cases} \quad (2)$$

where $\tau = t/T_f$, T_f is the *n*-instant of time at which the failure threshold δ=1 is reached, ρ and ω are parameters defining the damage curves (Fig. 8).

The current case studies assume the initial area A and the material strength $\bar{\sigma}$ of each structural element to be affected by the deterioration process:

$$A(t) = A_0[1-\delta(t)] \quad (3)$$
$$\bar{\sigma}(t) = \bar{\sigma}_0[1-\delta(t)] \quad (4)$$

The damage parameters ρ and ω in (2) must be chosen according to the deterioration process considered. Aggressive environments as well as variations in loading condition can reduce each structural element performance. The following relationships are assumed:

$$\rho = \rho_a + (\rho_b - \rho_a)\xi \quad (5)$$
$$\omega = \omega_a + (\omega_b - \omega_a)\xi \quad (6)$$

where the coefficient $\xi = \sigma/\bar{\sigma}$ describes the ratio between the stress level σ at *n*-instant and the design limit state for a generic structural element. Subscript *a* refers to damage associated with environmental aggression, while subscript *b* refers to loading-associated damage. When the damage parameters are properly estimated, the relationships (5) and (6) prove the law (2) to be successful in representing damage mechanisms related to aggressive environment and material natural ageing.

Based on historic evidence from Building Regulations (Colombo 1890, Sandrinelli, 1905) $\bar{\sigma}$ equal to 308,7 MPa and 240 MPa are most likely to be the design limit state values for iron and timber respectively. The parameters are obtained by identification, where the consequent damage law must agree with the deterioration value observed during 2016's survey. The identification process provides a mean value for each parameter's behaviour; the standard deviation was chosen in accordance with examples in publications on similar topics (Ciampoli, 1999, Ceravolo et al. 2009). Assuming the causes of the degradation process as random variables, the deterioration law (2) describes the cross section decrease over time. Since visual and photographic observations have shown comparable cross-sectional area reduction, the same damage law was applied to both steel and wood structures. Further investigations are in progress.

Figure 8 describes the decay law affecting the structure. Deterioration is assumed uniform on the whole element surface. Environmental aggressions and variation in loading conditions are both considered.

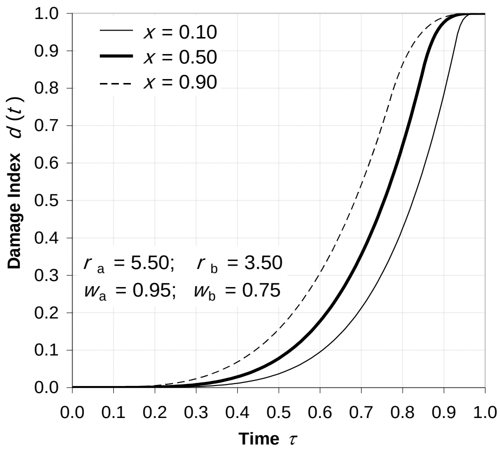


Figure 8 Damage index δ versus normalized time $\tau = t/T_f$ related to different performance loss percentage ξ.

3.1 *Mote Carlo simulation*

In order to investigate the life-cycle of the structure over the building lifespan along with the deterioration process affecting it, a Monte Carlo simulation implemented with the decay law (2) is run. Damage parameters ρ_a, ρ_b, ω_a, ω_b, and failure time T_f are modelled as random variables with assigned probability distributions: Normal distribution for parameters ρ and ω, Gamma distribution for failure time T_f (Tab.4).

Tab. 4 Input parameters and related standard deviation for Monte Carlo simulation

Parameters	Distribution	Mean	Stand. Dev.
ρ_a	Normal	5.50	0.20
ρ_b	Normal	3.50	0.20
ω_b	Normal	0.95	0.02
ω_b	Normal	0.75	0.02
T_f	Gamma	185.00	15.00

Using RDM.m random choice function of MatLab code (Matworths Inc. 2005), 1000 simulations have been performed. For each run, a random value from the assigned probability function is given to the variables, and implemented in law (2). Then the degradation law is applied to a time-dependent structural analysis for the evaluation of the structural response in terms of decrease in stiffness, member by member, and the estimation of the failure time for the whole structure.

This procedure provides considerable data: a variety of failure time samples for the evaluation of the average failure time T_{fail} and the estimation of their probability density function $F_{fail}(t)$, and the variation affecting geometry and mechanical properties of each structural member.

4 RESULTS

In the present section, Monte Carlo simulation results of the three case studies have been collected and discussed.

4.1 *Case 1: long span timber-iron Polonceau truss system*

Monte Carlo simulation shows a 10% to 15% damage spread over the entire structure. Although the overall deterioration level doesn't seem to compromise the structural stability yet, member 6 critical conditions shouldn't be underestimated (Fig. 9)

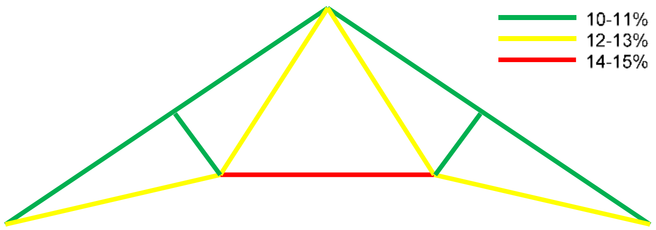


Fig. 9 Damage percentage member by member

4.2 *Case 2: short span timber-iron Polonceau truss system*

The simulation resulted in an 11% to 15% loss of cross section.

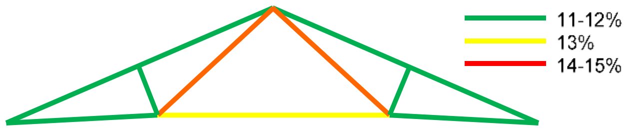


Fig. 10 Damage percentage member by member

As in Case 1, the structural stability is not compromised yet, but the critical members are now the longest, diagonal ones (9; 10).

4.3 *Case 3: steel English truss*

Once again, there is an overall deterioration. However, comparing Case Study 3 with the previous ones, because of the smaller dimension of its elements, the damage appears to be more consistent and it has been estimated in the range from 11% to 22% (Fig. 11).

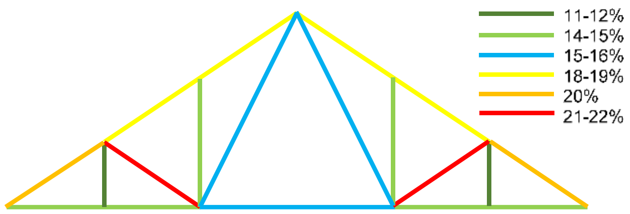


Fig. 11 Damage percentage member by member

Monte Carlo simulation proves the structure to be still strong enough to withstand current loading and environmental aggressions, though the slenderest members (i.e. elements 13th and 18th) have lost a significant portion of their original cross sectional area.

4.4 *Additional remarks*

Despite the conditions of the three case studies are not optimal, from the above-mentioned results it appears that they might have a still long residual life expectancy (Tab.5).

The comparison of the case studies reveals that the trusses are subjected to a different deterioration process depending on the elements' dimension and structural geometry, though the decay law applied is the same. For instance, Case 1 shows the horizontal member as the critical one, while mid-diagonal members (i.e. elements 9 and 10) are the weakest in Case 2.

When it comes to a sudden failure of majorly damaged elements, it is necessary to consider not only the structural resistance to unexpected hazards (e.g. impact loading, blast loading, terrorist attack, etc.), but also the possibility of losing the support of a whole structural component (Maes et al., 2006, Okasha & Frangopol, 2009). That would drastically redirect the maintenance decision-making process towards repairing/replacing the key members most likely to be the cause of collapse.

5 ROBUSTNESS-RELATED MAINTENANCE SCENARIOS

This study, particularly, shows the risk some structural elements have to reach inadequate levels of reliability faster than others.

Index μ (Bondini et al. 2008) describes the decrease of structural reliability over time:

$$\mu(t) = [1 - \delta(t)] \cdot \frac{[\sigma(t) - \bar{\sigma}]}{\bar{\sigma}} \tag{7}$$

where $\delta(t)$ is the damage obtained by Eq. 2, $\sigma(t)$ is the level of stress at time t and $\bar{\sigma}$ is the ultimate stress (i.e. allowable/permissible stress). Figure 12 compares the time evolution of index μ for the three case studies.

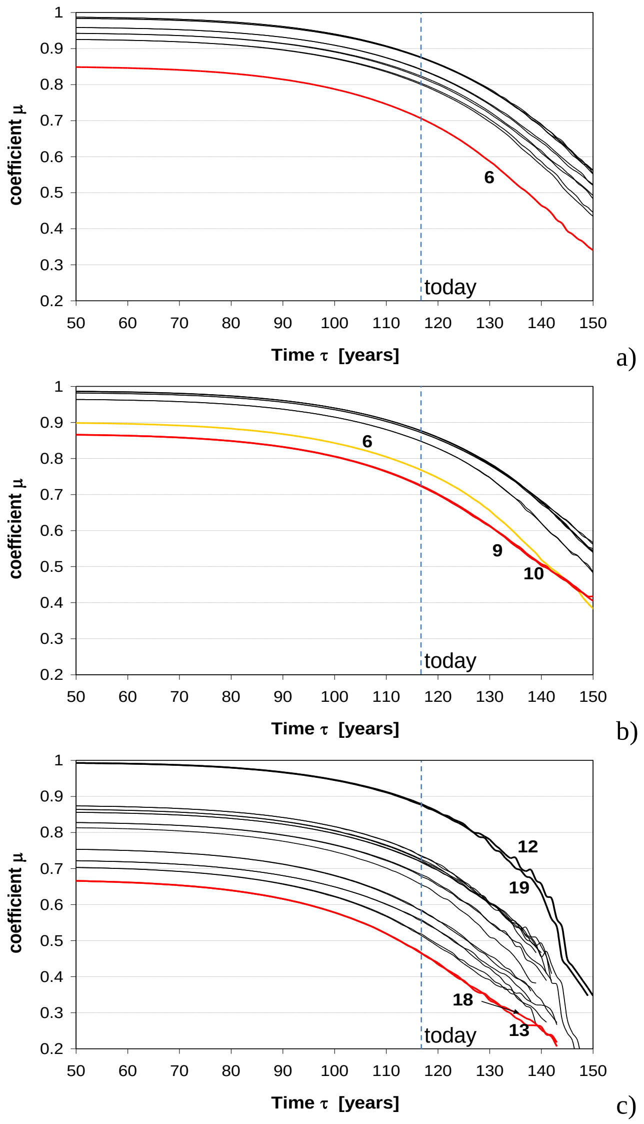


Fig. 12 Comparison between index μ evolution over time for all the members of each roof truss: a) Case 1; b) Case 2; c) Case 3. The most vulnerable components have been highlighted in red and yellow.

Figure 12 shows which component is most likely to fail. Furthermore, it appears clear that Case 3 truss system (Fig. 12c) is characterised by a lower reliability level than the other structures (Fig. 12a and b).

A selective maintenance would require an urgent intervention to restore the volume loss or replace the critical elements and the members with an index μ close to 0.4. Otherwise, when it comes to components characterised by a higher index μ, maintenance actions may be postponed.

Assuming the most vulnerable elements of the three case studies suddenly fail at the current time (i.e. instant time t equal to 117 years), it is important to understand whether their robustness would be enough to resist or they would miserably collapse, and if the constraint redundancy (i.e. Case 3) has any role in affecting the structural strength. In order to investigate this specific condition, the case studies have been evaluated considering 117 years as actual instant time, and without the critical members' support (Fig. 13).

Coefficient μ evolution can identify possible maintenance scenarios and support the definition of the structural robustness for each case study.

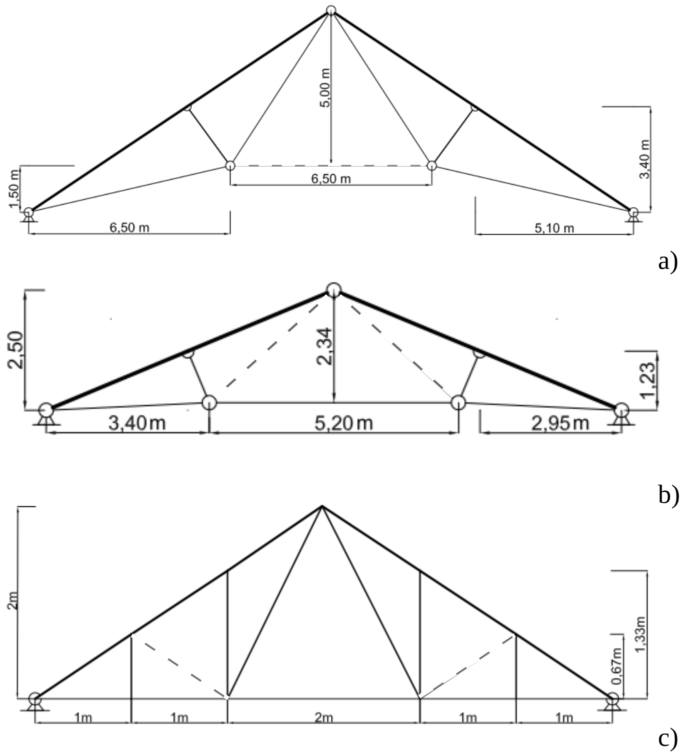


Fig. 13 Case studies analysed without considering the members affected by possible failure (dashed lines): a) Case 1; b) Case 2; c) Case 3.

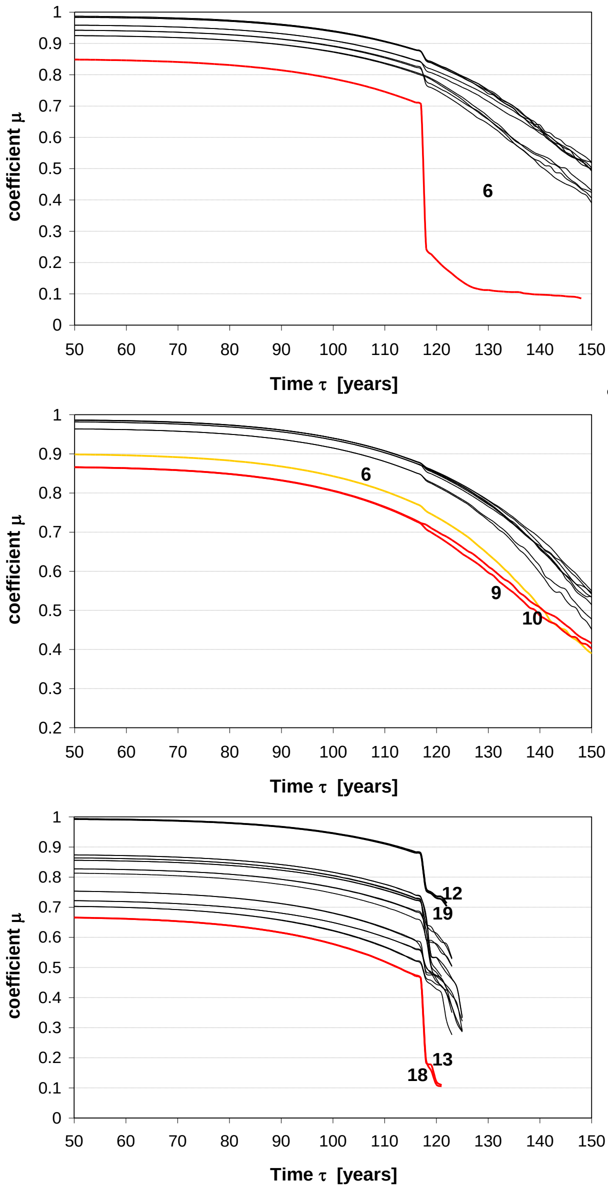
Figure 14 shows the results of the second analysis. In Case 3, index μ evaluation reveals that the system redundancy has not a significant impact on the stress distribution in the undamaged members, while a diffuse deterioration affects the whole structure with a failure time estimation of approximately 125 years (Fig. 13c; Tab. 5). Therefore, extensive maintenance actions over the whole roof structure would be necessary to protect the truss system and prevent it from failures.

In Case 1, member 6 failure has a low impact on the overall deterioration. Index μ evolution suggests that element 6 should undergo urgent maintenance, while the rest of the structure could be maintained in the long term (Fig. 14a; Tab. 5).

In Case 2, members 9 and 10 failure doesn't affect the overall structural response very much. The time evolution of index μ in Figure 14b is close to the one in Figure 12b. The failure time results in Table 5 again confirm the previous assertions. Case 2 truss system appears to be characterised by appreciable robustness values, which indicates no urgent maintenance is needed.

Tab. 5 Comparison between failure time up to today (117yo) in real structures and simulated structures (i.e. with some failed element).

Case Study	Failure time	
	Real	Members Failed
1	34.00	29.46
2	33.00	33.00
3	12.55	2.94



a)
b)
c)

Fig.14 Comparison between index μ evolution over time for all the members of each roof truss: a) Case 1; b) Case 2; c) Case 3. The most vulnerable components have been highlighted in red and yellow

6 CONCLUSIONS

This paper proposes a method for simulating the deterioration process affecting existing structures when the degradation typology can be assumed known.

This procedure allows the investigation of damage evolution over the whole structural life. Furthermore, it locates the vulnerable points where structural diagnostics should be focused on, and identifies the elements to be repaired/replaced.

The results here presented confirm the method as a reliable approach to recognise which structural element is most like to undergo failure, and schedule proper selective maintenance actions.

Furthermore, this methodology can be very helpful in defining the robustness level of a structure. In particular, in Case 3 (i.e. 33 constraints), Table 5 and Figure 14 show that constraint redundancy has no positive effects on robustness. On the other hand, proper cross section and thickness design of roof truss components ensure a better structural response to sudden collapse of the most vulnerable members.

Because of the importance of robustness in relation to maintenance strategies, this topic will be further investigated in future researches.

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