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# Capacity Design Criteria of 3D Steel Lattice Beams for Applications into Cultural Heritage Constructions and Archaeological Sites

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**Abstract.** Three-dimensional lattice beams are highly efficient technical solutions to cover large spans, especially when the single members do not have intermediate restraints able to prevent lateral-torsional buckling phenomena. Hence, lattice structures are widely applied in any field of civil and industrial engineering. In the present paper, in the framework of a research project funded by both the Campania Region and the European Community, a compound structure made of welded lattice beams and structural glass slabs is proposed as a structural system for valorisation and protection of monumental constructions and archaeological sites. Due to both the risk exposure of monumental heritage to be protected and the use of structural glass, the definition of an appropriate design criterion is mandatory in order to avoid development of brittle collapse mechanisms, mainly under static and dynamic vertical loads. The attention is herein paid to the design procedure, with a brief description of basic ideas behind the project itself and the main focus on the parametric capacity design of structural members. The proposed procedure, whose validity is quite general, has been subsequently verified by linear and non-linear numerical analyses calibrated on the basis of experimental investigations carried out on both the beam material and full-scale beam prototypes.

## Introduction

Spatial lattice beams are elements with longitudinal dimensions prevailing on transverse ones, belonging to the largest family of 3D structures [1], which are typically characterised by both a three-dimensional behaviour (spatial behaviour) and axial stress regimes (lattice behaviour). The spatial trusses, unlike the reticular box sections, are achieved by the repetition of tetrahedral and/or hemi-octahedral modules made of metal members, generally with pipe sections, mutually connected either by welding each other or through nodes characterising the constructive system. The high torsional and out-of-plane flexural stiffnesses make the 3D solution more convenient than the 2D one in all cases where self-support during construction is required and/or any lateral restraints to prevent flexural buckling system are absent [2].

The first applications of pre-fabricated 3D lattice structures date back to the early 1900s by Alexander Graham Bell, who experimented the use of tetrahedral and octahedral modules, obtained by assembling metal bars and joints, to produce gliders and wing structures. Bell immediately understood the enormous potential of such a constructive system, even for other uses, although several decades passed to see developed on industrial scale technologies capable of having reliable connection systems combining high structural performance with the easy of erection [3]. From the post-war period many patents, especially in the construction sector, were developed with the purpose to affirm industrialised systems in the field of steel structures [4], which began to oppose to the classical solutions of prestressed reinforced concrete beams for covering large spans [5]. The 70s represent the "golden age" of prefabricated 3D lattice structures [6], that nowadays are finding a new impetus for the construction of complex geometries made of steel or aluminium alloys [7, 8], which are used in the civil field for the construction of large roofing and towers [9] or in the industrial one for automotive, lifting and offshore applications.

In this paper, the use of spatial lattice beams in combination with structural glass is evaluated to erect coverings for valorisation of monumental constructions and protection of archaeological sites [10]. Despite the extensive use of steel for structural and energy retrofitting interventions [11], also including applications on artefacts with artistic and historical interest [12, 13], and even though ductility of steel members [14] is an important topic in the Seismic Engineering field, few studies have been devoted to analyse the dissipative behaviour of structures made of lattice beams [15], for which seismic design criteria have been not yet defined by the main international standards [16]. Contrary, the high exposure of goods to be protected, often located in medium-high level seismic areas, and the presence of structural glass [17, 18], require the design of a suitable project methodology matching the well-known “capacity design” principle with the “fail-safe" one. Indeed, both criteria should be used together synergistically to achieve, under vertical static or dynamic actions, members with ductile and robust behaviour.

In what follows, once briefly described the research project and the concept of the constructive system under investigation, the design procedure will be discussed, with focus on the parametric dimensioning process based on the capacity control of components of 3D lattice beams and also giving attention to the problem of connections, that is very important for steel [19] and aluminium [20, 21, 22] structures.

Indeed, for the sake of dimensioning, a “coarse-grained” approach [23], useful for both performing parametric analyses and making comparison with standard *I* beams having the same weight [24, 25], has been used as a design tool. The proposed procedure, whose validity is quite general, has been subsequently verified by linear and non-linear numerical analyses, which have been calibrated on the basis of experimental investigations carried out on both material and real beam prototypes.

## The Research Project Sketch

The herein presented study is part of a larger research project titled “Development and industrialization of innovative welded steel beam systems for lightweight floors and roofs for applications in monumental buildings and archaeological sites”, funded by the Campania Region by using the POR FESR 2007-2013 European funds for small and medium-sized enterprises and research organizations. The project partners were the Sideredil enterprise, which played the role of the project leader, and the Department of Structures for Engineering and Architecture (DiSt) of the University of Naples “Federico II”, which worked as the project co-proposer. The main objective of the project was to conceive, develop, validate and certify, for the purpose of subsequent marketing, an innovative floor system to be used for the realization of low weight roofs within cultural and archaeological heritage constructions. The project, lasting 12 months, was organised through five Work Packages (WPs) as follows:

WP1: Analysis of the state-of-the art of researches and applications on steel welded beams and steel-glass floors;

WP2: Definition of structural and technological requirements of beam prototypes;

WP3: Implementation of beam and floor prototypes;

WP4: Design and optimization of theoretical-experimental prototypes;

WP5: Definition of system requirements for industrialization and marketing.

The results of the study have been reported into appropriate deliverable documents related to the above five WPs.

The present paper describes in details the results of the second WP and part of the fourth one, which concerned respectively the system concept and the parametric design of spatial lattice beams. This activity was carried out according to a multilevel approach (Fig. 1), by adopting different modelling and analysis methods, for five different complexity levels of the analysis: the higher was the level, the more sophisticated was the adopted method.

The levels of analysis were conventionally labelled as Levels 0, 1/2, 1, 2 and 3. They ranged from the sizing of elements (Level 0), passing through the system performance evaluation (Levels 1/2, 1 and 2), up to the validation and calibration of the proposed methodology (Levels 2 and 3).

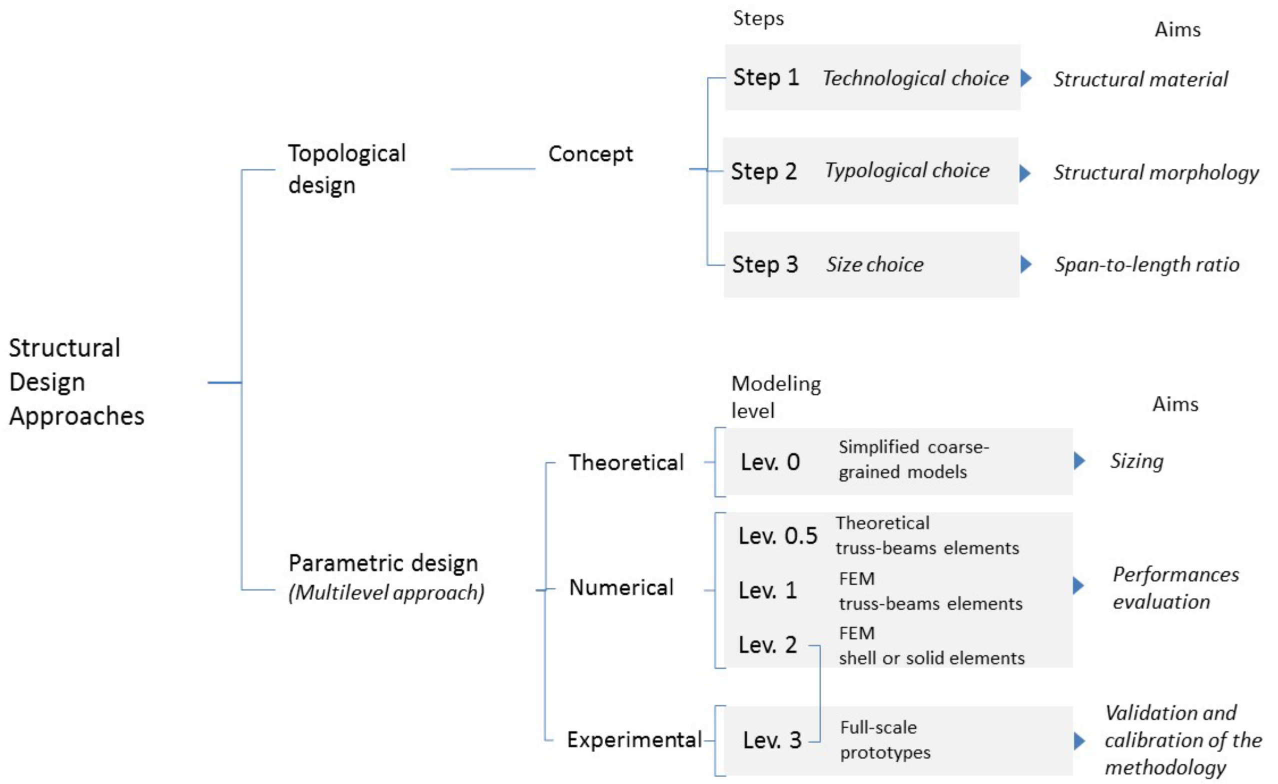


Fig. 1. The design process and research activities on investigated 3D lattice beams

Specifically, the Level 0 analyses are based on simplified theoretical models, whose closed-form solutions allowed for the evaluation of primary stresses to be used for parametric design. Level 1 analyses, referred to as linear elastic and buckling investigations, were performed using FE beam models able to capture secondary stresses. Structural checks were carried out using the capacity models provided by Eurocode 3 [26] and considering also the case of bending-torsional instability. Structural behaviour was also studied with more sophisticated analysis methods, referred to as Level 2, based on FE solid models. In particular, analyses taking into account geometric and material nonlinearities and explicitly defined imperfections were performed and suitably calibrated on the basis of experimental tests performed on full-scale prototypes (Level 3). Such a multilevel approach was used to hierarchically control the entire design process. In order to both validate the simplified formulas proposed for the Level 0 and to check the results obtained from Level 1, the intermediate Level 1/2 analyses, based on the Principle of Minimum Potential Energy, were also performed. In their turn, the effectiveness of Level 1 analyses was proved by Level 2 ones so that the formers, thanks to their low computational cost, were used to characterise the performance of beam prototypes. After the validation phase, such computations were used to elaborate the so-called Initial Type Calculations, which allow for the compilation of the Performance Statement document, used to guarantee the conformity assessment for getting the CE mark of the structural components under investigation, as required by UNI EN 1090-1 code [27].

## The Constructive System

The protection and the possible musealization of archaeological artefacts is usually made by using either shelters or protection enclosures, which can be profitably employed especially in case of extra-urban archaeological sites, where high intervention potentialities are noticed [10]. In addition to ensure an adequate protection level and the usability of goods to be protected, the main problems concern both the integration of protection systems with the context and the invasiveness of anchorage systems and foundations, which may interfere with the archaeological ruins [28]. Therefore, the main requirements of the structural system are the lightness, to minimise loads applied to foundations, and the ability to cover large spaces with limited intermediate supports, to ensure a proper utilization of goods. Other than these, flexibility, low maintenance and ease of assembly and dismantlement, as well as the possibility of reusing them in other environments [29], are important prerequisites of these protection covering systems.

A good alternative to timber, widely used for the protection of cultural heritage, is provided by high-strength low-alloy steels with improved corrosion resistance (weathering steels), which can be well integrated into the environment, thanks to their brownish colouring, delivering also a high durability. The current study proposes the use of S355J2W (numerical designation 1.8965) steel spatial lattice beams, in combination with laminated structural glass, for the construction of roofs for the protection of archaeological sites. The spatial lattice solution is justified by the need to cover significant spans without intermediate supports through high lateral stability members, which are needed due to the absence of secondary beams and/or a horizontal bracing system.

The beams, hereinafter referred to as BB.CC. (Italian acronym of *cultural heritage*), although obtained from standard profiles, have several innovative process and product aspects. These innovations are particularly related to the top chord, which is made of a tubular alveolar profile (Cellular Hollow Sections C-HS) obtained by longitudinal cutting and subsequent welding of the two resulting semi-parts, which are suitably staggered.

In order to cover spans variable from 6 up to 30 m and possibly more, it has been necessary to differentiate the member height by identifying three product families (Fig. 2), characterised by the height $H$ of 600 mm (low beams), 900 mm (medium beams) and 1200 mm (high beams).

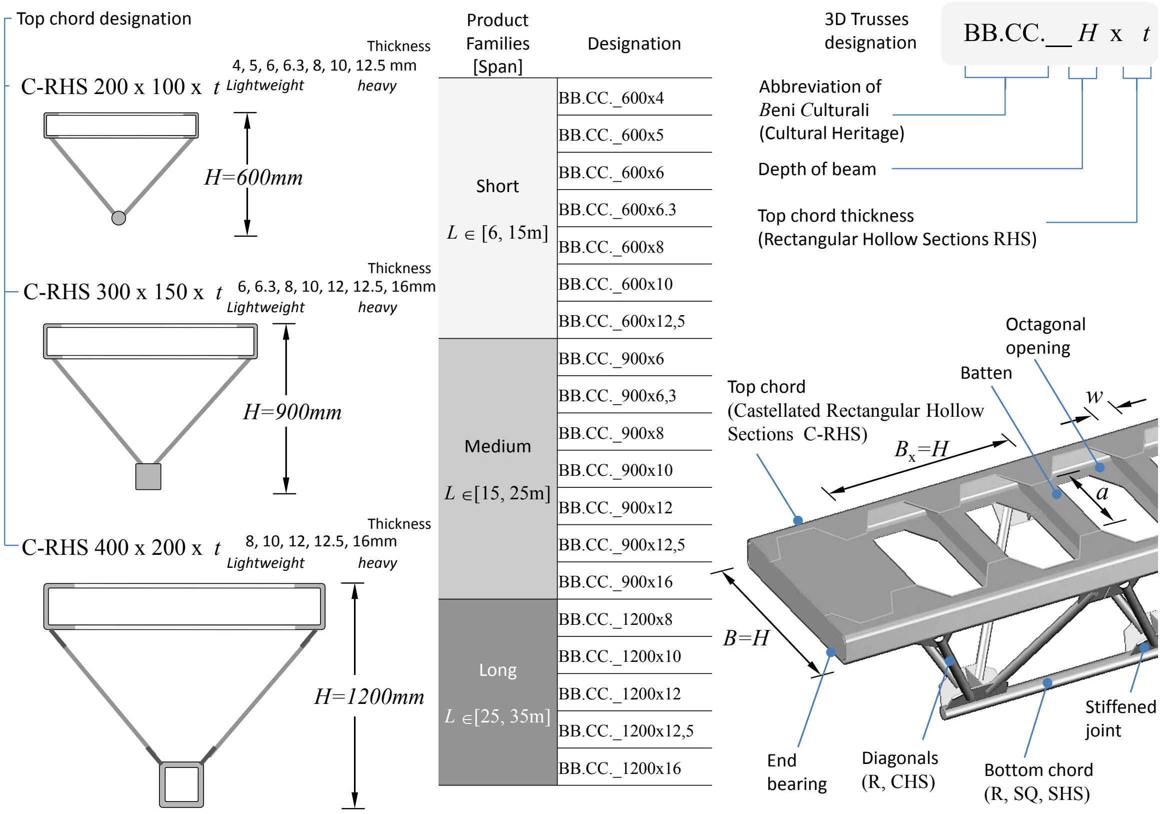

Fig. 2. BB.CC. beams: nomenclature and families of products

Once defined the steel type, within the same product family and through a capacity control procedure (see next Section), the performance of members has been modulated by modifying the size of the defined component elements as a function of the thickness $t$ of the Rectangular Hollow Section (RHS) profile used to achieve the top chord. Consequently, the generic beam is uniquely identified by the acronym "BB. CC._$H$ x $t$".

## Design Criterion

The high exposure of cultural assets to be protected, which are often sited in medium-high seismicity areas, and the use of structural glass as a roofing system material require the definition of an adequate design methodology based on the capacity control criterion. The used criterion is applied to avoid the occurrence of premature brittle collapse mechanisms, favouring at the same time a ductile failure of beams under exceptional vertical actions.

The favourite ductile mechanisms are given by the plasticization under tensile actions of the bottom chord. The brittle mechanisms to be avoided are, for example, due to either the buckling failure of end diagonal members (global shear collapse) or the crisis for instability of the compressed chords which, in the absence of torsional restraints, can lead towards the flexural-torsional phenomena of the whole beam, compromising the integrity of the structural glass floor directly placed on the members. Other brittle mechanisms are those induced by the failure of joints and end supports, the latter being prevented by the extension and stiffening of the top chord.

Once defined the design material and loads, both permanent ($g_k$) and variable ($q_k$), the parametric design process is divided into six distinct phases (see Fig. 3) by changing both the beam span $L$ and the centre-to-centre distance among beams $i_T$ into ranges compatible with the performance requirements of each BB.CC. family of beams, whose height was defined during the pre-design phase (see previous Section).

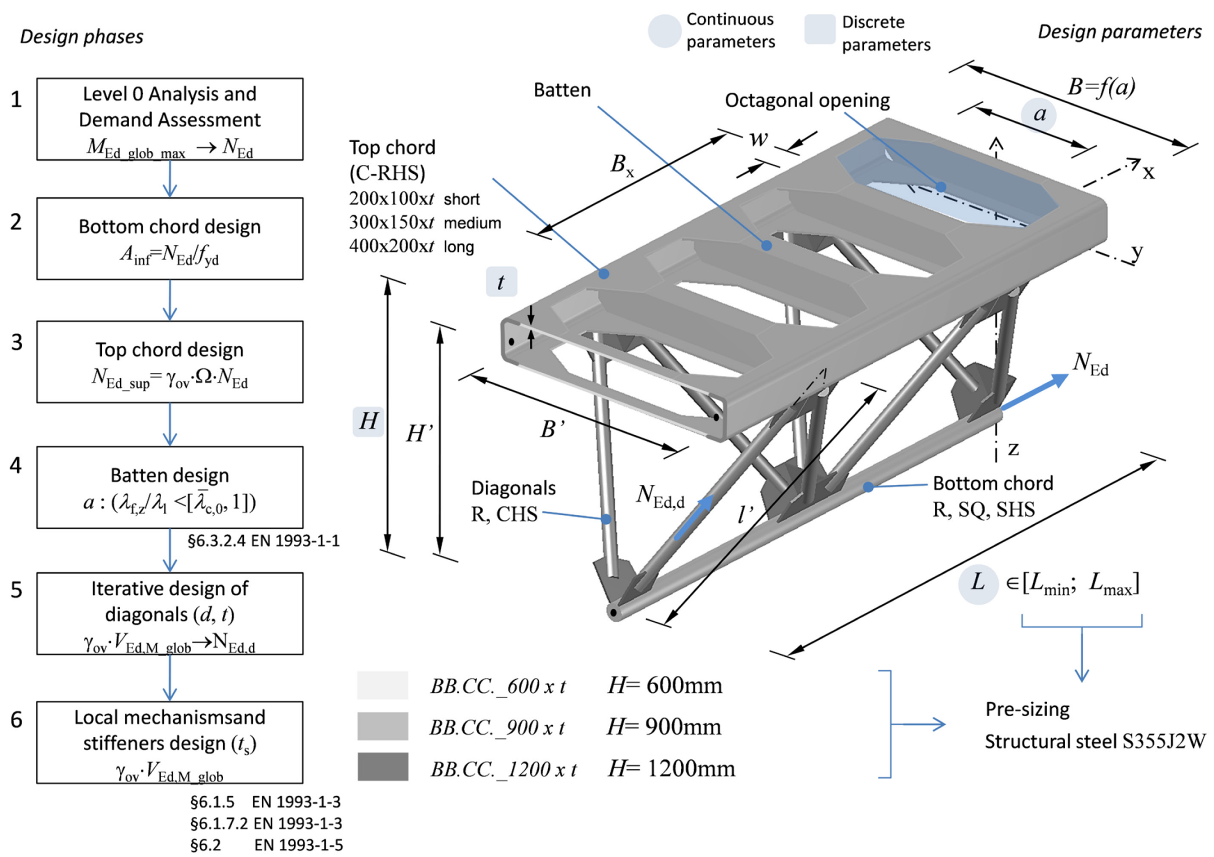


Fig. 3. Capacity control design methodology for spatial lattice beams

The first step (Phase 1) consists in determining the maximum axial load (demand) in the bottom chord ($N_{Ed}$). The primary stress regimen, of axial type for lattice systems, is evaluated in the design phase by means of simple (coarse-grained) modelling and analysis.

Referring to the case of a simply restrained beam subjected to vertical loads applied at the shear centre, the maximum stresses in the chords are deduced either from the Schwedeler's formulations [30] or by relating the maximum global moment $M_{Ed,glob}$ in the beam middle to the theoretical height *H'* of the beam schematised as an isostatic truss, that is with bars connected by hinged joints. The maximum axial loads in the diagonal bars $N_{Ed,d}$ at their ends are instead calculated by using a formulation appropriately conceived for spatial lattice beams with hemi-octahedral modules.

Once evaluated the demand in terms of stress in the beam critical areas, the design of the bottom chord, whose profile type (round R, square box SQ or rectangular box SHS) is selected according to the beam family considered, is carried out (Phase 2). Later on, the design of the top chord is done (Phase 3), so that it has an adequate over-strength with respect to the bottom chord. Therefore, the demand axial load is amplified through the coefficient $\Omega$ to both favour the bottom chord plasticization and compensate for any performance decrease that may arise in the compressed chord.

The expression of the over-strength coefficient is provided by the following equation:

$$\Omega = \frac{N_{pl,Rd}}{N_{Ed}} + \Omega_0 \tag{1}$$

where: - $N_{pl,Rd}$ is the plastic axial load of the bottom chord in the middle section;
- $\Omega_0$ is an additional over-strength factor.

In the design phase, referring to the gross area *A* of the RHS profile to be used for the top alveolar chord (C-RHS), the additional over-strength coefficient takes into account the decrease in capacity associated with the section reduction (net area $A_n$) generated by the profile cutting and offsetting, as well as to possible local or global (flexural) buckling phenomena in the xz-plane not properly evaluated during the sizing of members.

The flexural-torsional buckling of the beam, associated with the instability of the compressed chord in the *xy*-plane, is instead controlled (Phase 4) by searching for the optimal value of the parameter *a*, that defines the batten plate size and, then, for the width B of the member. By adopting the simplified assessment method provided in [31], also reported in the § 6.3.2.4 of the Eurocode 3 – Part 1.1 [26], the beam lateral buckling has been limited by containing the normalised slenderness of the top chord net section $\bar{\lambda}_{f,z}$, evaluated with reference to the *z*-axis, in the range between the threshold value $\bar{\lambda}_{c,0}$ and 1. Subsequently, by imposing the plastic failure of the bottom chord (plastic hinge in the middle section), both the limit load $F_{pl,Rd}$ and the associated global shear $V_{Ed,M\_glob}$ are determined, they being used to iteratively design the end compressed diagonals subjected to instability (Phase 5) for different values of the beam span comprised in a plausible interval ($L_{min}$; $L_{max}$) within the range of each BB.CC. beam family.

The procedure ends (Phase 6) by checking that no local mechanism appears in the end bearings due to shear and crippling crisis of the top chord web. To this purpose, the member is appropriately stiffened transversely in such areas with adequate thickness plates. In order to ensure the activation of the ductile collapse mechanism produced by the plasticization of the bottom chord, the sizing of the elements that must not collapse prematurely (compressed chord, end-diagonals and bearings) should be performed considering the uncertainties associated with the effective yield stress, i.e. by multiplying the design stresses $E_d$ for the material over-strength coefficient $\gamma_{ov}$, given by the current national and European regulations as equal to 1.10 for S355 grade steels. The entire procedure is applied by assuming that nodes are conceived and designed as full-strength restoring joints.

**Validation of the Proposed Procedure**

The proposed methodology for design of BB.CC. beams is validated through global and local (hereafter indicated as Levels 1 and 2, respectively) FEM analyses, as well as by means of experimental investigations performed on both material and full-scale beam prototypes (Level 3). In particular, through a large numerical Level 1 analysis campaign, by varying the span of each BB.CC.Hxt beam in the respective field of use (L∈ [$L_{\min}$,$L_{\max}$]), the following indicators are monitored:

- the collapse mechanism governing the Ultimate Limit State (ULS) for vertical loads of each BB.CC.*Hxt* prototype for a given span;
- the normalised difference between the demand-to-capacity ratios (s) measured between the upper and bottom chords and between the chords (the greatest between the two indices) and the end diagonals for the maximum load conventionally bearable by the BB.CC.Hxt beam with the considered span.

The analysis of results shows how the design criterion is particularly robust for low ($H$ = 600mm) and high ($H$ = 1200mm) BB.CC. beams. For the latter, the activation of brittle mechanisms involves only 15% of the analysed cases, related particularly to stocky beams, that is members with shape ratios ($r_{HL}$) greater than 1/10, which have not a great relevance from the application point of view.

The analysis of the scatters among exploitation indices shows in all cases an optimal utilization of chords with slight over-strength, oscillating from 5% to 15%, of the top chord with respect to the bottom one, validating the effectiveness of the used coefficient $\Omega_0$.

Except for very stocky lattice beams, the diagonals, that is the lattice web of beams, also appear to have sufficient over-strength in comparison to the bottom chord, with significant differences between the exploitation indices varying with the span, whose average value is about 40% in the proper field of use.

**Conclusions**

In this paper, the use of 3D lattice beams made of S355J2W high strength steel with enhanced resistance to atmospheric corrosion, in combination with structural glass, was investigated in order to create roofing structures for both the valorisation of monumental constructions and the protection of archaeological sites.

The high exposure of cultural assets, which often are placed in medium-high seismic areas, and the use of structural glass as a roofing material required the definition of an adequate design methodology, based on the capacity control criterion, aimed at avoiding the occurrence of premature brittle collapse mechanisms, instead of ductile failures, of the beams under vertical actions with exceptional nature.

Once permanent and variable loads were defined, the design criterion of examined trusses was articulated into six phases based on the design of the bottom chord, the top alveolar chord and the end diagonals.

Finally, the proposed methodology for the design of BB.CC. beams was validated through global and local FEM analyses first and, subsequently, by experimental investigations carried out on material and full-scale beam prototypes. The analysis of results showed that the proposed design criterion was particularly robust and able to control the collapse mechanism of each prototype in a very wide range of used spans, compatible with the actual application fields.

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