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Nonlinear instability analysis of long-span roofing structures: The case-study of Porta Susa railway-station

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ABSTRACT

Instability problems are going to be more and more important for long-span structures, especially for those where structural shape and loading capacity are strictly correlated. In view of this, a complete buckling analysis seems to be essential for the correct prediction of structural behaviour. At the same time, recent disasters, occurred in the last few years, such as the collapse of the new pavilion of Charles de Gaulle Airport in Paris (2006), have brought the instability of shallow long-span roofs at the cutting edge of structural engineering research. In this paper, different studies devoted to the stability of the large span roof of the new railway station of Porta Susa in Torino (Italy) are proposed. In particular, 2D models were realized in order to evaluate in-plane linear and nonlinear instabilities for different loading and restraining conditions of the steel arches constituting the bearing framework of the roof. These arches have been subdivided into different groups according to the geometric characteristics. It has been found that nonlinear analyses are able to give not only an interpretation of the post-buckling behaviour, but also a more correct evaluation of the safety factor for this kind of structures. Moreover, a parametric evaluation, taking into account different cross sections, is presented. The results reveal that much slender arches would offer the same safety factor as the existing ones due to the activation of a different resisting mechanism, although with an evident reduction in the employed material. Finally, the outcomes of this case-study could be generalized in order to investigate the behaviour of other structural typologies and to suggest alternative design approaches.

1. Introduction

The structural behaviour of long-span arches and roofs is a topic treated for a long time in structural engineering research [1–3]. For steel arches, several studies are related to their limit state design and to the consequent collapse mechanism: plastic collapse or buckling. Both the problems are usually studied through numerical approaches [4]. Plastic deformations occur rarely in slender structures where generally the buckling instability takes place when the material is still elastic. On the other hand, the influence of plasticity cannot be neglected in several cases, especially for arches subjected to large bending moments. By the limit analysis, closed form solutions have been obtained during the last sixty years [5]. Recently, the same problem has been analyzed for steel arches characterized by different sizes and shapes [6,7], and also, in very recent papers, for structures subjected to different restraining conditions and loading configurations [8,9]. It has long been recognized that a structure will in general lose its stability by either "snapping" or "buckling". From a theoretical point of view, the structure is said to snap when the equilibrium path emerging from the unloaded state loses its stability on yielding the first locally maximum value of the loading parameter; the structure is said to buckle when the path loses its stability at a point of bifurcation [10]. Physically, this means that when a snap-through instability occurs, the structure reach a new stable configuration when the maximum load is reached. On the opposite, this cannot happen after the attaining of the critical load of the classical buckling theory. As a consequence, not all the structural systems can have a snap-through instability, in fact it is possible only with some loading and geometric configurations. Generally, this problem regards shallow and slender compressed structures [10]. The in-plane elastic stability of arches has been analyzed for a long time since the usage of slender steel and pre-stressed concrete members has been intensified and the problem of buckling has become crucial [11]. In the last few years, different loading configurations and restraining conditions have been studied and many closed-form solutions have been obtained [12], included the case of shallow arches [13]. For elastically restrained arches, considering both geometric

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and material nonlinearities, a closed-form solution has been recently proposed [14]. Generally, this problem is solved obtaining the critical load multipliers by the extraction of the eigenvalues of the arithmetic solving system associated with the linearized expressions of the total elastic potential of the structure; this means that the virtual work of each degree of freedom has to be stated. The so-called snap-trough instability phenomenon was analyzed in a series of works based on the analysis of simple structures [15,16]. The introduction of this different kind of instability has opened the way for the study of the post-buckling behaviour of structures by the analysis of the equilibrium paths [17,18]. Such problem is still actual and a more refined theoretical approach has been presented during the last twenty years [19]. The solution of this problem is completely different from the former one because the intrinsic geometrical non linearity makes impossible the expression of a linear arithmetic system. A numerical step-by-step approach is usually the most used and stable method, but many alternative ways has been proposed during the years, such as the power series expansions for the elastic potential used by Thompson [10]. For some structures, the instability problem could become of primary importance and then the post-buckling behaviour could be used as a leading design condition for structural optimization [20]. In addition, during the last 30 years, some experimental studies on the instability conditions of arches have been conducted [21].

However, the results of these general studies cannot entirely satisfy the safety needs in practical engineering. In fact, history teaches us that the most severe collapses of the past have generally occurred in extreme conditions [22] for which structural weakness emerged only after the failure, or, as in the case of terminal 2E of Charles de Gaulle airport, where the definitive failure cause has not been found yet.

Therefore, a complete and correct understanding of the structural stability response is necessary for the evaluation of the loading capacity of the entire building even in uncommon situations, i.e., for particular loading configurations, even when they are not prescribed by the codes.

The analysis, described in the present paper regards the structural behaviour of a series of arches that constitute the main steel skeleton of a large roofing structure, the actual core of the Porta Susa railway-station, built in Turin (Italy) in 2011 (see Fig. 1).

The numerical simulations that have been conducted in this investigation concerned: the load configurations, their effect and their possible mutual interaction in a geometrically nonlinear regime. This analysis appears to be able to identify the most vulnerable portion of the entire vaulted structure. Also the interaction between plastic collapse and buckling is analyzed and it is shown how the two mechanisms are strictly related and neither of them can be neglected in a correct evaluation of the safety factor.

The results outline the crucial role of the computational nonlinear analysis for the design of this kind of structures. In addition, a correct evaluation of the safety factor is obtained regarding the efficiency of the structural shape. Therefore, thanks to the study of the post-buckling, it has been possible to provide some preliminary remarks about the optimization process in the structural design. All the analyses that have been carried out showed that a true identification and comprehension of the collapse modalities leads to a more performing structural optimization regarding the restraining condition, the geometric configuration, and the mechanical parameters.

2. Analysis of the arch-type members

The structural members of the new Porta Susa station in Turin have been analyzed. These 108 unique (the geometry of each one is different to the other) arch-type members are composed of two equal straight columns and three upper circular arch portions (Fig. 2a and b). The employed section is the same for each element: an alveolar steel HEB 600 beam. Each frame is restricted by a pressed pivot-system at the basis of the columns. The procured rotational stiffness is in general not null, then the established constraint cannot be assumed only as a perfect hinge (see Fig. 2c). From this point of view, it must be considered the role played by the global rigidity to the structural instability, and especially how the collapse can be influenced by a certain restraining degree. For this reasons, the analyses conducted in the present paper will be carried out taking into account the two extreme restraining conditions: (i) perfect hinges and (ii) fully fixed supports. Operating this way, those two conditions will envelope all the possible intermediate circumstances. Obviously, these considerations are necessary in a second-order analysis, because in the presence of a geometric nonlinearity an higher restraining degree not always guarantees a greater maximum critical load or a softer post-peak branch. The two modelling condition has been adopted in all the analyses in order to make comparison between them.

Eventually, the structure of each arch frame member is longitudinally connected by a set of straight beams laying at the top of each frame and by a complex bracing system (see Fig. 2d). Furthermore, some additional transversal elements offer a support to pedestrian platforms, stairways, elevators and anchorages of the tunnel. All these elements, due to their reduced contribution to the roof in-plane stability, were neglected in the numerical simulations proposed in the next sections. The lack of a rigid connection with the vertical elements and the reduced flexural inertia of the longitudinal elements, in comparison to the one of the arch frames, makes this as a right assumption. In the present paper, a classification has been proposed according to the following geometrical parameters: $OR = l_s/l_i$ and $SR = l_s/f$, that define the Opening Ratio and the Shallowness Ratio of each arch frame as shown in Fig. 2b. OR remains approximately constant around the unitary value, whereas SR shows a wide variation from a minimum value equal to 2.62 for the least shallow arch frame to a maximum value of 11.03 for the most shallow one. By means of an unitary step variation of SR, ten sets of arch frames have been identified and for each set the arch frame with minimum SR value has been selected as the representative one (Fig. 3). Operating this way, each typical arch frame differs from the nearest two by a variation in SR at least equal to 1. The most shallow one has been chosen because it represents the most severe geometric configuration towards the snap-through instability. All the numerical analyses have been implemented in the finite element solver and modeller LUSAS 14.3. For the nonlinear analyses, a co-rotational formulation together with an arc-length based solver algorithm has been used. In this way, the post-peak paths can be followed, and if an incremental displacement parameter is performed, even the unstable branch can be carried out by the solver.

3. First-order analysis

As a first stage, for each representative arch frame the Eulerian buckling load, by the critical multiplier λ, has been evaluated according to the following structural constraint conditions: double hinged and fully restrained arches. At the same time, two different loading configurations (1 and 2), assumed according to the Eurocode recommendations [23–25], and two additional ones (locally distributed, 3 and 4) were imposed to evaluate the buckling effects as shown, for the double hinged arch, in Fig. 4. Every load configuration has the same linear density and its position on the span is representative for a particular external loading configuration. Together with the classical load applications (1 and 2), Configura-