Thin-Walled Structures 115 (2017) 100–109

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Full length article

# Strength design of pin-ended circular steel arches with welded hollow section accounting for web local buckling

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ARTICLE INFO

*Keywords:*
Steel arches
Welded hollow section
Global buckling
Web local buckling
Strength
Design method

ABSTRACT

An effective way for improving the global load-carrying capacity of a steel arch with a welded hollow section is to increase the height of its webs. However, the high and thin web may buckle locally, which will affect the global load-carrying capacity and should be taken into account in predicting the global strength of the arch. This paper presents a numerical investigation for the in-plane strength and design for pin-ended circular steel arches with a welded hollow section, focusing on the effects of web local buckling by the large deformation inelastic analysis in association with a finite shell element model. The circular steel arches subjected to a radial load uniformly distributed along the arch length, a load uniformly distributed over the full span of the arch, or/and over the half span of the arch and their various combinations are considered in the investigation of their failure modes and strengths. The initial global and local geometric imperfections as well as residual stresses are included in the finite element model. It is found that web local buckling influences the global strength of the steel arches significantly and needs to be accounted in formulate their strength design formulas. The parameters related to the web local buckling and global buckling of the arch are thoroughly explored. It is found that the stability coefficients of arches under the nominal uniform axial compression are related to the equivalent normalized web height-to-thickness ratio of the cross-section and the normalized slenderness of the arch, and accordingly the design formula of the stability coefficients is proposed by including their effects. In addition, a design formula for predicting the strengths of the arches under a load uniformly distributed over the full span is developed by introducing a corrective coefficient into the stability coefficient, and it is found that the corrective coefficient is related to the arch rise-to-span ratio and slenderness of the arch. Furthermore, a general interaction design formula is developed for predicting in-plane strength of the arches under general in-plane loading such as a load uniformly distributed over the half span of the arch and various combinations of a full span uniform load and a half span uniform load. Comparisons with the finite element results demonstrate the proposed design formulas can provide good predictions and/or lower bound predictions for the strengths of the steel arches with a welded hollow section.

## 1. Introduction

This paper focuses on the in-plane strength and corresponding design method of pin-ended circular steel arches with a welded hollow section accounting for the effects of web local buckling. The elastic buckling load of pin-ended steel circular arches in nominal uniform axial compression produced by a uniform radial load has been studies extensively [1–5], and early studies of the classical in-plane elastic buckling of arches are summarized in *Guide to stability design criteria for metal structures* by Ziemian [6]. The investigation of the in-plane inelastic buckling and strength of arches have been concentrated on their global in-plane strength without considering the effects of the local buckling of the cross-section [7–11]. It is known that the plate local buckling of the cross-section affects the global load-carrying capacities of straight steel members including beams and columns as shown in Little [12] and Usami and Fukumoto [13]. Arches resist the in-plane loading by combined axial compression and bending, they are expected to behave in some extents similarly to the thin-walled steel beams and columns. In order to improve the global in-plane strength, the effective way is to increase the height-to-thickness ratio of plate components of the cross-section. However, this may lead to local buckling of the plate components and so the influence of the local buckling on the global stability of the arches should be taken into account as pointed out by Bradford [14]. ANSI/AISC [15] also categorizes the cross-sections of members into 3 classes according to the width-to-thickness ratio of plate components, such as compact, non-

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http://dx.doi.org/10.1016/j.tws.2017.02.010
Received 19 December 2016; Accepted 11 February 2017

compact and slender-element sections. The members with the slender-element section certainly allow the local buckling of the section. In addition, many national standards [15–17] provide the strength design provisions for steel beams and columns considering local buckling of plate components. However, these strength design provisions are unsuitable for steel arches because the mechanical behavior and failure mechanisms of the arches are much more complicated than that of the straight members. Therefore, investigation of the failure mechanisms and design methods of steel arches considering local buckling of plate components are desirable.

The efforts to improve the load-carrying capacity of steel arches have been reported in the literature. Guo et al. [18,19] proposed the circular steel arches with a sinusoidal corrugated web to achieve the object of saving material and improving the flexural stiffness of the cross-section. The failure modes of arches having a sinusoidal corrugated web have been studied numerically and experimentally [18,19]. It was found that the flanges resist the axial and bending actions while the sinusoidal corrugated web resists the uniformly distributed shear stress across the web height, and that steel arches with a sinusoidal corrugated web may fail in a global instability failure mode or/and the web shear failure mode. The corresponding design method has also been proposed in [18,19]. Gou et al. [20] also studied the failure modes, strengths and design methods for steel box section arches with both webs and flanges having large height-to-thickness ratio by considering local buckling of webs and flanges. It has been found [20] that a box-section arch may reach its ultimate strength in a local failure, global failure or coupled local and global failure mode. A design method for predicting the in-plane strengths of the arches has been developed and expressed in terms of the normalized height-to-thickness ratio of the plate components of the cross-section and the normalized global slenderness of the arch, which were found to have significant influence on the strengths of the arches. It is known that arches are generally subjected to a combined axial, bending and shear actions and the axial and bending actions are mainly resisted by the flanges located far away from the neutral axis of the cross-section. It is preferred that the flanges of a cross-section could resist the normal stresses without local buckling, while the webs resist the shear stresses. This can be achieved by welding flanges having sufficient width-to-thickness ratio with webs to form a welded hollow section. However, when the webs are high and thin, the webs may buckle locally. Although the local buckling would reduce the bending resistance of the webs, the bending stiffness and resistance of the welded hollow section increase a lot because of the increase of the height of the cross-section. Hence, the welded hollow section can improve the load carrying capacity of an arch significantly compared with the steel box-section having a uniform flange and web thickness [20]. To prevent the flanges from local buckling, the width-to-thickness ratio of the flanges can be properly limited in a conservative range. However, the web local buckling may be inevitable because of the large height-thickness ratio and this may influence the global load carrying capacity of the arches. Little research has been reported about the strength and design of steel arches having a welded hollow section. It is not known how the web local buckling influences the in-plane failure mechanism and strength of steel arches having a welded hollow section. There are no design rules in the current steel structure design codes and specifications for the strength design of such steel arches.

This paper, therefore, carries out numerical investigations for the in-plane failure mechanism, the load-carrying capacity and the design method for pin-ended circular steel arches with a welded hollow section accounting for web local bucking (Fig. 1).

The steel arches under uniformly distributed radial load, a load uniformly distributed over the full span or over the half span, and their various combinations are considered. Two finite element (FE) models are formulated using the commercial FE package ANSYS [22] for the numerical investigations: a shell element model associated with the large deformation thin plate theory for simulating the elastic-plastic behavior of steel arches including web local buckling, and a beam element model for simulating the elastic-plastic behavior of steel arches without web local buckling. Initial geometric imperfections of the arch axis and plate components, as well as residual stresses of the welded hollow section are formulated in the FE models. To investigation effects of web local buckling to the strength of an arch with a welded hollow section, the flange local buckling should not occur until the steel arch reaches its global ultimate load. For this, the flange width-to-thickness ratio of the welded hollow section in the FE models is assumed to satisfy the provisions of the compact section in AISC [15] and GB50017 [17].

## 2. Finite element models of steel arches

The shell element SHELL181 and the beam element BEAM188 of ANSYS [22] are used respectively to establish the FE models for elastic-plastic analyses of the steel arches with a welded hollow section. The shell element model is used for investigating elastic-plastic behavior and load carrying capacity of arches with web local buckling, while the beam element model for elastic-plastic behavior and load carrying capacity of arches without web local buckling. In these FE models, the global out-of-plane deformations of the arches are fully restrained. For the beam element model, the out-of-plane displacements of all nodes are restrained, while for the shell element model, the out-of-plane displacements of nodes at the intersection of flanges and webs are fully restrained. Assignments of boundary conditions and the method applying the load are different for two FE models. For the shell element model (Fig. 2), flanges and webs at the ends of the arch rigidly connect to an infinitely rigid end-plate respectively, and the centroid of the two end-plates is pinned to the abutment. For the beam element model, the translational degrees of freedom at both ends of the arch are fully restrained.

Additionally, the external load $q$ is directly applied along the centroidal axis of the arch for the beam element model, while the external load $q$ is applied evenly at the four corners of the hollow section for the shell element model as shown in Fig. 1(b), *i.e.* $0.25q$ at each corner to avoid detrimental bending in the flange so that their resultant will act at the centroid of the hollow section.

In the FE analyses, a bilinear elastic-plastic stress-strain curve having a Young's modulus of $E = 206$ GPa , Poisson's ratio of 0.3, and yield stress $f_y = 235$ MPa is adopted for the steel.

To investigate the influence of web local buckling on the load-carrying capacity, the flange local buckling should be excluded by limiting the width-to-thickness ratio $b/t_f$ of the flanges. Because of compatibility of deformations at intersections of flanges and webs, the constraint intensity of the web to the flange is weakened due to web local buckling [23], therefore the maximum width-to-thickness ratio of the flanges is less than the provisions of AISC [15], Eurocode 3 [16] and GB50017 [17]. The web height-to-thickness ratio ($h/t_w$) used in the investigation is assumed to vary in the range of 20–250, the flange-to-web thickness ratio $t_f/t_w$ in the range of 1–3.5, the rise-to-span ratio in the range of 0.15–0.45 [9,24–26], and the arch slenderness $\lambda_g$ in the range of 20–200 [16] defined by

$$\lambda_g = \frac{S}{2r_x}, \quad \text{with} \quad r_x = \sqrt{\frac{I_x}{A}}, \tag{1}$$

where $S$ is the length of the arch axis, $r_x$ the radius of gyration of the cross-section, $A$ the area of the cross-section, and $I_x$ the second moment of area of the cross-section.

The in-plane global initial geometric imperfections of the arch axis are included in the beam element model and shell element model. It is assumed that the distribution of the initial geometric imperfections of the arch axis is the same as the first order buckling mode of the arch under a uniform radial load, as shown in Fig. 3(a), and the corresponding maximum amplitudes of the imperfection are specified as $L/1000$ [9], which is consistent with the GB50017 [17]. In addition, the web