Engineering Structures 114 (2016) 35–47

Contents lists available at ScienceDirect

Engineering Structures

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

Experimental and numerical study on compressive behavior of convex steel box section for arch rib

Kangming Chen [a], Qingxiong Wu [a,*], Shozo Nakamura [b], Baochun Chen [a]

[a] College of Civil Eng., Fuzhou University, 2, Xueyuan-road, Minhou, Fuzhou 350108, China
[b] Dept. of Civil and Environmental Eng., Nagasaki University, 1-14, Bunkyo-machi, Nagasaki 852-8521, Japan

ARTICLE INFO

Article history:
Received 1 April 2015
Revised 13 January 2016
Accepted 19 January 2016
Available online 18 February 2016

Keywords:
Convex steel box section
Critical stress
Normalized stress–strain relation
Width ratio
Width–thickness ratio
Steel arch bridge

ABSTRACT

The mechanical behavior of a convex steel box section under axial compression is studied using loading test on reduced scale-model and nonlinear FE analysis, taking into consideration initial geometric deflection and residual stress. The actual bearing capacity of the convex steel section used for the arch ribs of the Yongjiang Bridge is obtained. The difference in critical stress among the stiffened plates is revealed. The influence of width ratio η between upper and lower boxes, stiffened plate width–thickness ratio R_R and ratio of stiffener's relative flexural rigidity to its optimum value γ/γ^* on the normalized stress–strain relation of the section are examined. It is found that the bottom plate has the lowest critical stress among all the stiffened plates in the convex section. A smaller width ratio increases the critical stress of the middle plate and hardens the post-buckling behavior. The effect of width ratio on the normalized stress–strain relation of convex section is so small as to be negligible. R_R and γ/γ^* values influence the critical stress of the convex section and significantly affect the post-buckling behavior. Larger R_R and smaller γ/γ^* values reduce the critical stress of the cross section. Equations representing the normalized stress–strain relation of the convex section under axial compressive loading are proposed as a function of these influential parameters and their validity is demonstrated through numerical analysis.

1. Introduction

Steel arch bridges are widely deployed among the world's highway networks. In these applications, arch ribs for the bridges consist of various rectangular box sections, I-sections and pipe sections. Structures using these sections have been extensively investigated, both numerically and experimentally, aiming at understanding their strength under various loads.

Kroll proposed variations of the local buckling coefficient for a box section, a wide-flange I-section and a Z- or channel section with respect to the geometrical properties of the member [1]. Interaction between the elements of a plate assembly is inescapable because of the equilibrium and compatibility conditions that must be satisfied at the junction. In the case of local buckling, it is possible to simplify these conditions considerably, as shown by Benthem [2]. The insignificance of in-plane displacements in comparison to out-of-plane displacements makes it possible to assume that normal displacements are zero for each plate element meeting at a corner. Also, because bending rigidity is so low in comparison with extensional rigidity, it is possible to assume that the in-plane membrane meets another plate element at an angle [3,4]. In the plastic design of steel structures, it must be ensured that the moment capacity of a member is not impaired by local buckling until the required rotation is achieved. This can be achieved by limiting the width–thickness ratios of elements that are vulnerable to local buckling in the inelastic range. Such limited width–thickness ratios were proposed for flanges of I-sections by Lay in 1965 [5]. Minimum stiffness ratio stiffeners for compressed stiffened plates were studied by Yukio [6]. Stiffness requirements for transverse stiffeners of compression panels and longitudinally stiffened panels were proposed by Choi [7,8]. Related provisions are also found in design specifications [9,10]. Further studies on ultimate strength assessment of damage steel stiffened plate due to corrosion, wind dented et al. can be found in studies conducted by James [11], Silva [12] and Xu [13].

The structural sections employed in practice are composed of plate elements arranged in a variety of configurations. It is clear that the behavior of an assembly of plates would be governed by the interactions between the plate components, which might be quite different from those of a single plate. For this reason, many studies on local–overall interactive buckling and ultimate strength of steel box columns were conducted, and many strength curves and hysteretic models of steel box columns have been proposed

* Corresponding author. Tel.: +86 591 83358433.
E-mail address: wuqingx@fzu.edu.cn (Q. Wu).

http://dx.doi.org/10.1016/j.engstruct.2016.01.037
0141-0296/

under certain specified conditions, such as those by Usami, Sakimoto, Liu, Adachi, Ge, Aoki, Kim, Wang and Yuan [14–22].

As the theory of steel arch bridges has developed and with improvements in steel quality, it has become possible to build steel arch bridges with longer spans. In order to satisfy architectural aesthetics as well as the mechanical requirements of steel box arch bridges with long spans, irregular-shaped box sections are frequently adopted for arch ribs. For example, a reversed trapezoid section was applied to the arch rib of the Lupu Bridge [23]. The Yongjiang Bridge in Ningbo City was built with arch ribs of convex box section [24–26], giving the structure a novel appearance. This type of convex section has rarely been employed for a steel box arch bridge anywhere in the world, so there is no established engineering experience for reference. In fact, there are no provisions for the design of such an irregular steel box section anywhere in the world. The arch ribs of the Yongjiang Bridge were designed according to the *Specifications for Highway Bridges* published by the Japan Road Association [10], using the provisions for the design of stiffened rectangular steel boxes to design of the convex sections so as to guarantee local stability.

This paper focuses on the mechanical behavior of convex sections when local buckling, residual stress and initial deflections of the stiffened plates in the section are taken into account. A loading experiment in which axial compression was applied to a 1/4 scale-model of an arch rib segment was conducted, to study the mechanical characteristics and actual bearing capacity of the convex steel section used for the arch ribs of the Yongjiang Bridge. Extensive parametric analyses were carried out to examine the differences in critical stress among the stiffened plates as well as the effects of width ratio η, stiffened plate width–thickness ratio R_R and γ/γ^* value on the normalized stress–strain relation of the section. The width ratio is the ratio of top plate width to middle plate width and is used to investigate the influence of the convex box section. The normalized stress–strain relations of a convex section and a rectangular section are compared. Then a formula for obtaining the normalized stress–strain relation of a convex section is developed. These normalized stress–strain relations can be employed as constitutive material relationships in an overall FE model of a large-scale structure with a convex box section using beam-column elements in order to take into account local buckling and initial imperfections.

2. Outline of bridge studied

The bridge studied in this paper is the Yongjiang Bridge, a half-through X-style steel arch bridge with spans of 100 + 450 + 100 m (Fig. 1). The deck width is 45.8 m. Each arch rib has upper and lower limbs, and the sections of the arch ribs located between $l/4$ and $3l/4$ are convex in shape due to the joint of the upper and lower boxes (Fig. 2), l is the length of main span. In the side spans, the axes of the lower limbs of the arch follow a quadratic parabola and the clear rise is 4.5 m, while the lower limbs have a catenary shape in the main span, where the rise-to-span ratio is 1/5. The axes of the upper limbs follow circular curves at the two ends and quadratic parabolas in the center with a rise-to-span ratio of 1/11.5. In the side spans, the width of the lower limb is 3.5 m, the height varies from 6 m at the abutment to 5 m at the end of rib. The thicknesses of the top and bottom plates and the webs are in the range of 16–20 mm and 20–30 mm, respectively. In the main span, the width of the lower limb is 3.5 m and the height varies from 6.5 m at the springing to 4.8 m at the arch crown. The thicknesses of the top and bottom plates are in the range of 30–55 mm and those of the webs are in the range of 25–30 mm. The width and height of the upper limb of the main span is 2.8 m and 3.0 m, respectively, and the thicknesses of top plate, bottom plate and webs are in the range of 16–20 mm [24–26].

3. Loading test of reduced scale model

3.1. Design and fabrication of the model

The local buckling of the section is most likely to occur at approximate $l/4$ due to the relative larger height of the section. Therefore, a 1/4 scale model of the section at $l/4$ point was made by considering the fabrication, experimental requirements and limitations. In order to reproduce the mechanical behaviors of existing convex section accurately, all the dimensions of the components of reduced scale-model were controlled strictly in accordance with the actual size, except some minor adjustments, such as curvature of arch rib was so small as to neglect. The same steel type of Q345 was adopted. The general layout and completed model is illustrated in Figs. 3 and 4, respectively.

The completed reduced scale-model was placed on two foundations with rollers, and the interfaces between model and roller were painted with machine oil to make sure the model can move effortlessly along the longitudinal direction. The both ends of the model were strengthened by the steel–concrete composite structures for easy and uniform loading. The composite structure was composed of two steel plates, as shown in Fig. 5, with the interval of 0.5 m. The I-steels and steel tubes were employed to connect the two steel plates. In addition, the concrete was poured in the gaps between the two plates. The composite structures were integrated with the main part of the model by some stiffeners. Besides, in order to avoid local buckling in the section close to both ends due to stress concentration during loading, the intervals of diaphragms within 1.5 m from either end were shorten to the half of that in the middle portion, as shown in Fig. 3.

3.2. Arrangement of the measurement points

In order to study the critical load at which the local buckling occurs or stress reaches yield limit, the in-plane strains and out-of-plane deflections were mainly measured during the process of loading. The measuring points were arranged in the regions where the buckling might happen most possibly according to the theoretical analysis.

For monitoring the local deformation and integral slippage, 3 vertical and 1 longitudinal displacement meters were placed at the top and bottom plates, 3 lateral and 1 longitudinal displacement meters were placed at the web plates. 16 displacement meters in total were employed.

For monitoring the location of local buckling, the strain gauges were placed in lines and along the longitudinal direction of the plates. There were 3 longitudinal and 1 lateral strain-measurement lines for top, middle and bottom plates. In addition, 2 vertical and 2 lateral strain-measurement lines were placed in the web plates. Strain gauges were also placed on the stiffeners and diaphragms to measure their strains. 337 strain gauges were used in total.

3.3. Loading

According to the *specifications for highway bridges* [10], the allowable axial compressive stress of the convex section of reduced scale-model can be calculated without considering local buckling, the allowable axial compressive load of the reduced scale model is 23,024 kN. Therefore, the value of 23,500 kN was preliminary determined as the maximum trial load. In this experiment, the