Journal of Constructional Steel Research 65 (2009) 479–488

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

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

Improving strength and ductility of continuous composite plate girder bridges

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ARTICLE INFO

Article history:
Received 6 August 2007
Accepted 24 May 2008

Keywords:
Bridge
Plate girder
Steel
Concrete
Composite
Plastic mechanism

ABSTRACT

The ultimate load carrying capacity of continuous composite plate girder bridges is usually limited by the local buckling failure of steel girders at interior supports. This paper presents a simple reinforcement method which changes the failure mechanism of the continuous girder from local buckling to formation of plastic hinges at the interior supports and mid-span. Such a change in failure mechanism greatly improves the strength and ductility of the superstructure. In this method the compressive portion of the web near the interior support is braced against local buckling by bolting pairs of stiff bracing elements on opposite sides of the web. The bracing elements prevent local buckling failure of the support section and create a section which can rotate inelastically at plastic moment allowing the second hinge to form at mid-span. The bracing elements may be plates or longitudinal stiffeners which should be designed to remain elastic while the section undergoes plastic deformation. The behavior of plate girders which are reinforced by such bracing elements is studied using nonlinear finite element analyses.

1. Introduction

Continuous composite plate girder bridges are the most common type of steel bridges in the span range of 30–90 m. The concrete deck is usually 200–300 mm thick and spans 3–8 m between adjacent girders. To reduce the weight of the girders, grade 355 MPa steel and relatively slender webs ($100 < D/t < 200$) are often used throughout the length of the girders. The neutral axis of the composite plate girder in the positive moment region is very close to compression flange and the ultimate bending moment capacity in this region is usually the plastic moment capacity of the composite section. However, the ultimate load carrying capacity of the continuous girder is governed by local buckling failure of the steel section near the interior support. Due to lack of ductility associated with such a failure, the potential ability of the composite section at mid-span to reach its plastic moment capacity can not be fully utilized. If the local buckling failure is prevented, the support section can rotate inelastically at plastic moment allowing the second hinge to form at mid-span. This would significantly increase the ductility and load carrying capacity of the bridge.

This paper presents a reinforcement method which would delay local buckling in the support region in order to allow the formation of plastic hinges at support and mid-span. In this method, the local buckling resistance of the support section is increased by providing elastic supports on the web. The elastic supports which restrain the out-of-plane deformation of the web are provided by bolting stiff bracing elements on opposite side of the web. General design guidelines are developed and the behavior of sections designed in accordance with the guidelines is studied using nonlinear finite element analyses.

2. Flexural behavior of plate girders

Flexural behavior of plate girders which are adequately braced against lateral torsional buckling is largely dependent on local stability of the section. If the web and compression flange are stocky, then the section is stable at large inelastic strains and it can develop a plastic hinge with significant rotational capacity at plastic moment M_p. If these elements are slender, local buckling of the web and compression flange limits the ductility of the section and flexural strength will be less than the plastic moment. In U.S. design specifications [1–3] sections are classified as compact, noncompact, or slender depending on the slenderness ratio of the web and compression flange. A compact section is one that can develop a plastic hinge with significant rotational capacity at plastic moment. A noncompact section is one that can develop a moment equal to or greater than first yield moment M_y. A slender section is one that fails due to local buckling before the yield moment is reached. Fig. 1 shows the moment–rotation behavior of girders with the three classes of cross section.

For a laterally braced girder, the compactness requirements according to AASHTO LRFD specification are the followings.

$$\frac{b_f}{2t_f} \le 0.382\sqrt{\frac{E}{F_{yf}}} \tag{2.1}$$

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doi:10.1016/j.jcsr.2008.05.010

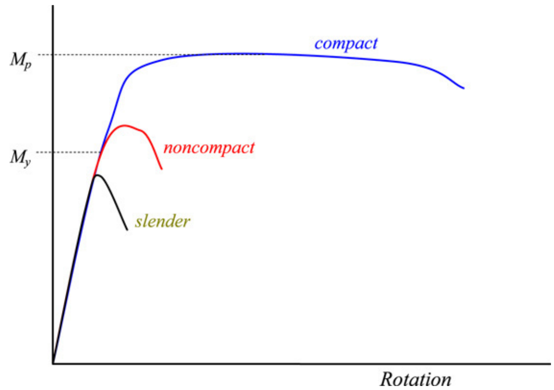

Fig. 1. Moment rotation behavior of girders.

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yf}}} \tag{2.2}$$

where:
b_f = width of compression flange, t_f = thickness of compression flange
D_{cp} = depth of web in compression, t_w = thickness of web
F_{yf} = flange yield stress, E = modulus of elasticity.

Experimental data [4–6] indicates that sections may not reach their plastic moment capacities when the web and compression flange slenderness ratios both exceed 75% of the limits given in (2.1) and (2.2). If the slenderness ratios of the web and the flange are greater than 75% of their respective limit, the AASHTO specification requires the following interaction equation to be satisfied.

$$\frac{2D_{cp}}{t_w} + 9.35\left(\frac{b_f}{2t_f}\right) \leq 6.25\sqrt{\frac{E}{F_{yf}}}. \tag{2.3}$$

2.1. Composite plate girder

Composite girders subjected to positive bending moment are not susceptible to local buckling failure. Due to the large compression area provided by the concrete slab, the neutral axis of the composite section is close to the compression flange and only a small portion of the web is subjected to relatively low compressive stresses which is not likely to cause any instability problem. Furthermore, in such sections, yielding first takes place at the tension flange and then progresses into the web while the compressive stresses are largely resisted by the concrete slab. Experimental studies [7] have indicated that composite bridge girders with slender webs can reach their plastic moment capacity with adequate ductility.

2.2. Longitudinally stiffened plate girder

For long span plate girder bridges where very slender webs are used, one or more longitudinal stiffeners are usually welded to the compression portion of the web in order to reduce its out-of-plane deflection. The optimum location of a single longitudinal stiffener to effectively control web deflection is at $0.2D$ from the compression flange [8]. This location has been adapted by the AASHTO specification as the standard location for a single longitudinal stiffener. The specification limits the moment capacity of such girders to the first yield moment but it permits a 100% increase in maximum allowable web slenderness ratio.

According to the AASHTO specification, longitudinal stiffener should be sufficiently stiff to maintain a nodal line along its length in the buckled web and should have adequate local and overall buckling resistance. The AASHTO requirements for moment of inertia, I, and radius of gyration, r, of a longitudinal stiffener to maintain a nodal line in the buckled web is as follows:

$$I \geq Dt_w^3\left[2.4\left(\frac{d_o}{D}\right)^2 - 0.13\right] \tag{2.4}$$

$$r \geq 0.234d_o\sqrt{\frac{F_y}{E}}. \tag{2.5}$$

In computing I and r values, a centrally located web strip with $18t_w$ in width is considered as part of the longitudinal stiffener.

Studies on longitudinal stiffener effectiveness have mainly dealt with plate girders with welded stiffeners [9–13]. Such studies indicate that strength and ductility of plate girders improve by welding longitudinal stiffeners to the web. Behavior of plate girders with bolted longitudinal stiffeners has not been studied. Unlike welded stiffeners, bolted longitudinal stiffener may be designed to remain elastic while the girder undergoes plastic deformation. Under such conditions the longitudinal stiffener maintains its stiffness and provides bracing for the web even after the section completely yields at M_p. The inelastic behavior of such girders is expected to be better than that of a plate girder with welded longitudinal stiffeners.

3. Proposed method

In this section a simple reinforcement method for improving strength and ductility of continuous composite bridges is introduced. The aim of this method is to change the failure mechanism from local buckling failure at interior supports to the formation of plastic hinges at the supports and mid-span. This way the local buckling resistance of the support section is increased by providing elastic supports on the web. The elastic supports are provided by bolting stiff bracing plates or longitudinal stiffeners on opposite side of the web as illustrated in Fig. 2. The main objective is to create a section which has high resistance to local buckling and is able to reach its plastic moment capacity with good ductility. The plates or stiffeners should brace compressive portion of the web and should terminate near transverse stiffeners. They should not be subjected to significant axial stresses. Friction at interface of the web and bracing elements would be the main source of such axial stresses. This stress can be reduced by providing oversized holes for the bolts and treating the interface surfaces. With such connection, the bracing elements could be designed to remain elastic while the web undergoes plastic deformation. The elastic bracing elements can prevent premature local buckling of the section if the slenderness ratio of the compression flange is within the compactness limit. Under such circumstances the support section can rotate inelastically at plastic moment allowing the second hinge to form at mid-span.

The longitudinal extent of reinforcement is limited to plastic hinge zone at the interior support. This is usually less than the depth of the girder on either side of the support. The bracing elements may be of any engineering material.

3.1. Bolted bracing plates

The bracing plates should be adequately rigid to prevent buckling of the web and shall remain elastic while the web undergoes plastic deformation. The required stiffness of the bracing plates can be estimated using an inelastic approach. In this approach it is assumed that at ultimate load, the compressive stresses transmitted to the unbraced web is carried by two strips adjacent to the compression flange and the neutral axis with a total width which would satisfy the compactness requirement.