Engineering Structures 151 (2017) 503–517

Contents lists available at ScienceDirect

Engineering Structures

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

Experimental and analytical investigation of end zone cracking in BT-78 girders

Vidya Sagar Ronanki, David I. Burkhalter [1], Sriram Aaleti *, Wei Song, James A. Richardson

Department of Civil, Construction and Environmental Engineering, The University of Alabama, Box 870205, Tuscaloosa, AL 35487-0205, United States

ARTICLE INFO

Article history:
Received 12 April 2017
Revised 28 July 2017
Accepted 7 August 2017

Keywords:
Prestressed concrete girders
End zone cracking
Transfer length
Bulb-tee
Finite element modelling

ABSTRACT

In the past decade, with a growing interest in long span bridge girders, designers are using an increasing number of prestressing strands. This scaling up has led to cracking issues in the end zone of narrow stemmed bulb-tees and I-girders, impacting the service life and possibly design capacities in case of excessive cracking. In a project funded by the Alabama department of transportation (DOT), a 180 ft long girder design was developed by modifying a standard Bulb-Tee girder and using a 10 ksi self-consolidating concrete mix. The new girder is 78 in. deep and has 66–0.6 in. strands. The impact of the draping angle and debonding on the end zone cracking was first evaluated using a 3 D finite element model (FEM) developed in ABAQUS. Subsequently, the critical stresses were monitored during the detensioning process of four 54 ft long full-scale girders with different end zone details. It was found that the combination of limiting the draping angle and debonding the strands resulted in minimizing the end zone cracking. The detailed FEA model which was developed is also verified using experimental field data. The results of this experimental and analytical investigation will be presented in this paper.

1. Introduction

Precast prestressed concrete girders offer several advantages including, high quality, low life-cycle cost, and modularity. Consequently, they provide a high performance, economical and an overall efficient solution for highway bridge super structures. Over the years however, there was a natural progression towards increasing the girder sizes and lengths to meet the design and economic constraints. This resulted in an increase in the number of strands in girder cross-sections, leading to problems related to end zone cracking. It should be noted that end zone cracking in itself is not an unknown issue. In the initial developmental stages of prestressed concrete girder designs, this problem was observed due to a complete absence of vertical reinforcement [1]. Whereas, the problems currently associated with end zone design are more to do with the scaling up of the girder size and an increase in the amount of prestressing strands that are being used. The consequences of these cracks are concerning. These cracks which naturally form near the location of prestressing strands (see Fig. 1), open up a direct path for moisture and corrosives to reach the strands and reinforcing bars. This results in strand corrosion, concrete spalling, and eventually may lead to reduction in the flexural and shear load capacity of the member by interrupting the bond between the strands and the concrete. Excessive cracking in the end zones requires the fabricators to epoxy inject the cracks or, in some severe cases, even reject the girder.

Alabama Department of Transportation (ALDOT) had previously implemented a girder spanning 165 ft (50.29 m) using a modified bulb-tee (BT) shape girder. In-spite of being designed to be in full compliance to the current AASHTO LRFD standards [2], these girders would often develop cracks in the end zone regions as shown in Fig. 1. A prestressed concrete girder spanning 180 ft (54.86 m) was specifically developed for ALDOT with minimal variations to standard BT girder formwork. This girder design, designated as BT-78, is a 78 in. (1.98 m) deep girder designed with 66–0.6 in. (15.24 mm) prestressing strands and 10 ksi (68.95 MPa) self-consolidating concrete mix. With the increase in the number of strands in the girder, there was a need to evaluate the effectiveness of end zone reinforcement details used by ALDOT to minimize end zone cracking in this girder. Experimental field testing of instrumented full-scale girder specimens with different end zone reinforcement was performed. A non-linear finite element analysis (FEA) model for the end zone region was developed and used to explore the influence of design parameters such as strand

* Corresponding author.
E-mail addresses: ronankisagar@gmail.com (V.S. Ronanki), dburkhalter@mbasei.com (D.I. Burkhalter), saaleti@eng.ua.edu (S. Aaleti), wsong@eng.ua.edu (W. Song), jrichardson@eng.ua.edu (J.A. Richardson).

[1] Current address: Structural Project Engineer, MBA Engineers, Inc., 300-20th Street North, Suite 100, Birmingham, AL 35303, United States.

http://dx.doi.org/10.1016/j.engstruct.2017.08.014
0141-0296/

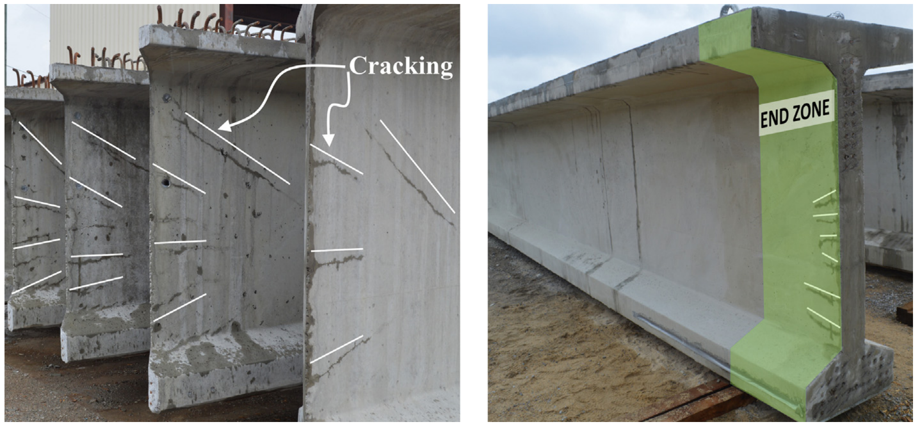

Fig. 1. Cracking observed in end zone region of bulb-tee (BT) girders stored at a precast yard.

debonding and draping angle of harped strands on the end zone behavior. This FE model was also validated using the field data. This paper presents the details of these experimental and analytical investigations and the resulting recommended end zone reinforcement details to minimize end zone cracking in deep girders.

2. Historical background and literature review

A summary of the basis for end zone reinforcement design and a brief review of studies available in the current literature regarding end zone cracking and mitigation is presented in this section. The basis for anchorage zone reinforcement in current design codes comes from the experimental and analytical studies carried out by Marshall and Mattock [3] and Gergely and Sozen [4] in early 1960s. Marshall and Mattock [3] established an empirical relationship between the stirrup forces and the extent of cracking in the endzone, based on experimental studies performed on prestressed girders with depths around 25 in. (0.63 m). While, this study provided an estimate of the reinforcement required, the analytical and experimental work carried out by Gergely and Sozen [4] provided a relationship to quantify the stresses in the vertical stirrups for a given crack width. This relationship would allow for the computation of limiting stresses on rebar for a permissible crack width. Tuan et al. [5] proposed a major modification to the distribution of the end zone reinforcement based on the experimental investigation of NU-I and NU inverted Tee girders. The authors recommended placing a pair of very large bars, designed at a stress level of 20 ksi (137.9 MPa), to resist two percent of the total prestress at very near the end of the girder to control the splitting cracks. The reinforcement to resist the remaining two percent of the prestressing force is recommended to be distributed across the critical shear section. The authors also proposed an alternative design of distributing the reinforcement to resist the splitting force of four percent along a distance of $h/2$ from the girder end with 50 percent of that reinforcement placed within a distance of $h/8$ of the girder end [6]. Following this, in a study supported by Virginia DOT (VDOT), Crispino et al. [6] investigated the issue of end zone cracking using an experimentally verified strut and tie models on PCBT-63 girders. It was concluded from this study that for the cracks to be within VDOT acceptable limits, the stress in anchorage zone reinforcement should be limited to 12 ksi (82.7 MPa) instead of 20 ksi (138 MPa). Considerable tensile stresses were observed in the region between h/4 and 3 h/4 and let to a recommendation that this region should also be considered in the end zone reinforcement design. Okumus et al. [7] performed a non-linear finite element analysis of the end zone of a Wisconsin BT-54 (54 W girder) to understand the end-zone behavior during the detensioning sequence. The results from the finite element models were verified using experimental data from two 54 W girders. After establishing parity with field results further models were prepared to examine the effect of release sequence, the effect of increased vertical reinforcement in the end zone and the effect of debonding. Based on these finite element models, it was observed that using large vertical bars at the girder end limits the strains in the web area. Debonding the strands at the girder end directly lowers the stresses transferred to the concrete and controls cracking. It was also observed that changing the cutting sequence does not particularly influence end zone cracking. Hamilton et al. [8] performed a multi-component test program for the Florida DOT on FIB girders to evaluate the effects of vertical post tensioning, debonding of strands and end zone stirrup distribution to mitigate endzone cracking. This study concluded that partial debonding was effective in controlling the length and width of the web splitting cracks. Further, it was also seen that increasing vertical end zone reinforcement compared to AASHTO [2] requirements decreased the length and widths of the web splitting cracks. In a recent study performed by Arab et al. [9], using a 205 ft (62.48 m) long, Washington State DOT's WF100G super girder with 80–0.6 in. diameter (15.24 mm) strands, it was found that longitudinal reinforcement placed in the end zone does not aid in the reduction of end zone cracks. This investigation carried out on the full span girder also reaffirmed the recommendations made by Tuan et al. [5] regarding the distrubution of end zone reinforcement. A summary of the parameters involved in these studies is presented in Table 1. All of the above mentioned studies suggested increasing the end zone reinforcement and debonding of strands to control cracking. The effectiveness of these recommendations along with a few further modifications such as inclination of draped strands were evaluated while arriving at the BT-78 girder design.

3. Design of Bulb-Tee 78 (BT-78) girder

The design and detailing of the BT-78 girder was carried out as per the current ALDOT practices [11] and the requirements of AASHTO LRFD code specifications [2]. The girder section was designed to span a distance of 180 ft (54.86 m) at 6 ft (1.83 m) girder spacing under HL-93 loading. The maximum moment demand on the BT-78 girder at mid-span under the service and ultimate load combinations were calculated as 10,646 kip-ft (14435.98 kN-m) and 14,673 kip-ft (19896.59 kN-m) respectively. As per ALDOT requirements, the girders were designed with simply supported boundary conditions and zero-tensile flexural stresses in concrete during detensioning and under service loading conditions. This required the BT-78 section, to be designed with 10 ksi.