Engineering Structures 33 (2011) 3597–3609

Contents lists available at SciVerse ScienceDirect

Engineering Structures

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

Shear failure analysis on ultra-high performance concrete beams reinforced with high strength steel

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ARTICLE INFO

Article history:
Received 18 March 2010
Received in revised form
7 March 2011
Accepted 27 June 2011
Available online 20 August 2011

Keywords:
Bond strength
Passive reinforcement
Shear reinforcement
Dowel action
Moment–shear interaction

ABSTRACT

A new deck system for moveable bridges was developed that makes use of ultra-high performance concrete (UHPC) reinforced with high strength steel (HSS) rebar to achieve the light weight and high strength requirements in moveable bridge applications. However, the typical deck strips of this deck system failed predominantly due to shear cracks in simply supported beam proof tests. This paper investigates the mechanism of the deck strip shear failure experimentally and analytically. Experimental studies were performed at several scales, including material characterization, bond strength tests, small-scale prism tests, and full-scale beam tests. Specimens with traditional shear strengthening techniques were also tested. Several existing formulas were utilized to predict the shear strength, and the results were compared to the experimental results. The accuracy and limitations of these formulas are discussed. The shear failure of UHPC–HSS beams is not characterized by brittle response or catastrophic load reduction as with normal reinforced concrete. Therefore, this particular shear failure mode is regarded as acceptable. However, the additional shear resistance caused by the localized deformation of the longitudinal reinforcement is not recommended to be considered for design capacity formulas.

1. Introduction

Ultra-high performance concrete (UHPC) is a fiber-reinforced concrete (FRC) with high compressive strength cement matrix and high tensile strength fibers. It exhibits the strain hardening effect in uniaxial tension tests due to the existence of the fibers with volume fraction at about 2%. According to the classification method introduced by Naaman and Reinhardt [1], UHPC can be regarded as one kind of high performance fiber-reinforced cement composite (HPFRCC). Several UHPC products are commercially available worldwide, and Ductal® is one widely available in the US. The material properties of Ductal® were fully investigated by Graybeal [2]. The high strength and good durability were confirmed based on the experimental results. Compressive strengths as high as 221 MPa are ensured by applying a heat treatment process, which is recommended by Lafarge, the manufacturer of the UHPC material used in this paper; otherwise, lower strengths will result. The benefits of this recommended heat treatment process were also confirmed by Graybeal [2]. The tensile strength was experimentally investigated by Chanvillard and Rigaud [3] using both uniaxial and four-point bending tests. The ultimate tensile strength from the direct tension test was found to be around 10.8 MPa, while the equivalent strength obtain from the flexural test was usually higher due to the scale effect. The fibers blended in the UHPC matrix provide a bridging effect across the micro cracks and thus increase the tensile strength and ductility. The punching shear resistance of the UHPC slab was investigated by Harris and Roberts-Wollmann [4], and the minimum deck thickness to prevent the punching shear failure under the factored wheel load (165 kN) for a 200 mm by 500 mm patch is predicted to be 25 mm. In Europe, Ductal® has been investigated by Toutlemonde [5] to build a full depth waffle shape bridge deck system. In US, the Pi-girder and two-way waffle shape full depth deck system are currently being investigated in Iowa state [6–9] under the research projects sponsored by the Federal Highway Administration (FHWA).

The term high strength steel (HSS) used in this paper refers to structural steel material that has a minimum yielding stress over 517 MPa (75 ksi). Several types of Grade 75 stainless steel rebar are commercially available [10] that meet the requirement of ASTM A955 [11]. Although the superior corrosion resistance of stainless steel rebars makes them the best choice for deck applications, the material cost is usually several times higher than that of normal Grade 60 carbon steel rebar. High strength microcomposite steel rebar (MMFX2) is an uncoated, high strength rebar made from a low carbon, chromium alloy steel. It meets the requirement of ASTM A1035 [12] for Grade 100 rebar with yielding stress of 690 MPa and ultimate strength as high as 1200 MPa. Although

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0141-0296/$ – see front matter
doi:10.1016/j.engstruct.2011.06.023

the corrosion resistance of MMFX2 rebar is not as good as that of stainless steel rebar according to Clemeña [13], it is much better than that of carbon steel rebar. MMFX2 rebars were used in this research for their superior strength and ductility.

The structural behavior of UHPC beams reinforced with passive HSS rebar is affected by the bond strength between the two materials. Although no exact information was found in the literature on this topic, several related works are mentioned here to give a lower bound estimation of the bond strength between the two. Holschemacher [14] investigated the bond strength between a pure UHPC matrix and ribbed Grade 60 carbon steel rebar by using pull-out specimens. The local bond stress was around 40–70 MPa for US #3 rebars with 45 mm concrete cover. Some of the specimens with 25 mm cover failed in concrete splitting. The UHPC used in the test had no fibers in the mix design, and none of the specimens were subjected to heat treatment. Lubbers performed anchorage tests on UHPC (Ductal®) [15]. Low relaxation, 12 mm diameter, 1862 MPa prestressing strands were embedded in UHPC with a minimum bond length of 305 mm and no prestress force applied. All strands that fractured during the pullout test showed the existence of the high bond strength. Normal strength concrete beam splice tests performed by Ansley [16] on US #6 and US #8 MMFX2 rebars show that the bond length required to yield the rebar is 45 times the rebar diameter. After yielding, the nonlinear ductile response of the rebar material reduced the bond strength and changed the commonly brittle splice failure to a gradual and more ductile failure.

Advanced construction materials, such as UHPC and HSS, provide an opportunity to help rehabilitate infrastructure systems that are either structurally deficient or obsolete. In particular, a deck system utilizing ultra-high performance concrete (UHPC) with only passive HSS longitudinal reinforcement was proposed for use on moveable bridges [17] to replace the existing open grid steel deck system. The original grid deck system has several shortcomings: the riding surface is less skid resistant when wet; traffic-induced vibration causes noise and sensations of poor ridership [18]; and the steel deck is corrosion and damage prone, and costly to maintain. The replacement deck system has stringent acceptance criteria on the overall depth, self weight, and capability to meet the AASHTO LRFD code [19] requirements, which prevent the use of conventional reinforced concrete. Several light-weight deck systems made of fiber-reinforced plastic (FRP) [20] or aluminum [21] were proposed, while their application in the field are still under investigation. The combination of UHPC and HSS rebar provides a solution with a light-weight, high strength deck system that was experimentally proven to meet the crucial requirements [17]. However, one of the remaining concerns about the new UHPC–HSS deck system is its shear type failure mode. Although the ultimate load of all the simply supported T-section deck strips, which were originally designed based on the flexural strength criterion [17], exceeded the expected load demand, the specimens all failed with widened shear cracks similar to normal strength reinforced concrete. However, the shear failure of the UHPC–HSS beam was not abrupt or catastrophic. It was observed from the experiment that along with the widening of the shear cracks, the longitudinal rebar at the crack location bent locally while the concrete in the compression zone either stayed intact or gradually crushed. The high ductility of this failure mode is attributed to the high strength, high ductility, and self-compacting property of the UHPC material, which causes the considerable bond strength between the HSS rebar and UHPC. More investigation of this type of shear failure is necessary to determine if it should be avoided or be accepted in design practice.

The objective of this paper is to further investigate the mechanism of the shear failure mode of UHPC beams with passive HSS rebars via both experimental and analytical investigations. Experimentally, mechanical tests at different scale levels were performed. The material level tests were performed to confirm the material properties. A series of bond tests based on a compression mechanism were performed to obtain the estimate of the bond strength between HSS rebar and UHPC blocks with concrete cover approximately equal to the rebar diameter. Small-scale beams with different end anchorage types were tested. The full-scale T-section beam tests had similar specimen geometry and loading configurations as in the previous research [17]. For the purpose of this study, additional specimens were tested with transverse reinforcement either by adding single-leg stirrups or by bending up the longitudinal rebar. Another two specimens were tested with different longitudinal reinforcement in an attempt to achieve flexural failure. Analytically, although there are quite a few shear strength design formulas [22–27] in the literature, for most of them, UHPC is out of the applicable range due to the ultra-high compressive strength. The only off-the-shelf equation for shear design with UHPC is from the "Ultra-High Performance Fibre-Reinforced Concretes—Interim Recommendations" published by Association Franccaise de Génie Civil (AFGC) [27]. It has been used to predict the shear strength of prestressed bridge girders [28]. Dowel action contribution is not included in the French code formula but is believed by Reineck [25] and He and Kwan [26] to be an important shear transfer mechanism in beams without shear reinforcement. Their 'tooth model' is used to check the shear capacity controlled by the interfacial bond strength. A deformable strut and tie model was also proposed based on the observed deformation shape of the specimens. The predictions based on the three analytical methods were compared with the experimental results and their accuracy and limitations were discussed. In addition, moment and shear interaction is discussed in this study and the difference between UHPC and normal strength concrete were presented. The analytical approaches used by Choi et al. [29] and Choi and Park [30] were adopted to investigate the maximum shear capacity under the influence of external moment.

Shear-induced deformation and failure is very important in the design of reinforced concrete flexural members without transverse reinforcement. Modified compression field theory (MCFT) was used by Vecchio to predict the shear responses of reinforced concrete [31]. The moment–shear interaction was considered at the material level under the principal stress domain. It was found that although the shear resistance generally decreases with the increase of the moment, for T-sections, the shear resistance may increase with slightly increased external moment. Application of MCFT to UHPC members is dependent on the further investigation on the material responses under multi-dimensional stress states, and this limits its application in this paper. Equation 11-5 in the ACI 318-08 code [22] reflects the fact that shear strength decreases when the moment capacity is fully utilized. The same conclusion was drawn by Muttoni and Ruiz [23] for rectangular sections based on the assumptions that the shear failure was directly related to the critical shear crack width. Choi et al. [29] and Choi and Park [30] claimed that only the top concrete portion in compression provides the shear resistance of the section without shear reinforcement. In addition, the allowable shear contribution is dependent on the normal stress caused by the external axial load and the moment. By checking the principal tensile and compressive stresses, the maximum allowable shear stress of all section layers can be obtained. Therefore, the maximum allowable shear force of the section can be estimated with respect to the external moment level and the resisting stresses distributed along the section height. Although shear–moment interaction always exists at the section level, the impact may not be significant at the component level. By comparing the load capacity of the flexural member to its plastic load capacity calculated based on the ideal flexural failure, Imam et al. [32] found that the influence of shear