Journal of Constructional Steel Research 83 (2013) 37–50

Contents lists available at SciVerse ScienceDirect

Journal of Constructional Steel Research

Behaviour of uni-axially loaded CFST columns confined by tie bars

J.C.M. Ho *, M.H. Lai

Department of Civil Engineering, The University of Hong Kong, Hong Kong

ARTICLE INFO

Article history:
Received 7 August 2012
Accepted 28 December 2012
Available online 1 February 2013

Keywords:
Analytical model
Columns
Concrete-filled-steel-tube
Tie bars
High-strength concrete
Normal-strength concrete

ABSTRACT

Concrete-filled-steel-tube (CFST) columns have been widely adopted for column construction of tall buildings due to its superior strength and ductility performance contributed by the composite action. However, this beneficial composite action cannot be fully developed at early elastic stage as steel dilates more than concrete and thereby causing imperfect interface bonding, which reduces the elastic strength and stiffness. To resolve the problem, confinement in the form of tie bars is proposed in this study to restrict the elastic lateral dilation of concrete. For verification, 30 CFST columns of various dimensions cast with normal- (NSC) or high-strength concrete (HSC) and installed with tie bars were tested under uni-axial compression in this study. From the results, it was evident that: (1) tie-confined CFST columns had uni-axial strength larger than those of unconfined CFST columns. (2) Tie bars can increase slightly the elastic stiffness of the CFST columns. (3) Tie bars can restrict effectively the lateral dilation of CFST columns. (4) The failure modes of tie-confined NSCFST and HSCFST columns were the local buckling at the ends and formation of longitudinal steel cracks initiated at bolt holes. Lastly, based on the test results, analytical model taking into account the confining effects of steel tube have been developed to predict the uni-axial strength of tie-confined CFST columns.

1. Introduction

Thanks to the advancement of concrete technology, the production of 150 MPa (cylinder strength) ordinary ultra-high-strength concrete (UHSC) (i.e. concrete without steel/special fibres) is easily achievable. This type of UHSC is a very ideal construction material to be adopted for column construction of tall buildings because it increases the strength-to-weight ratio of the structures, reduces the embodied carbon of the structures and saves extra floor space. However, the application of such UHSC on nowadays practical column construction is very rare. One of the major reasons that inhibit the adoption of UHSC is owing to its brittleness. UHSC columns will fail abruptly without ample warning when it is not confined laterally [1–3]. Although installing traditional transverse reinforcement within column potential plastic hinge [4–6] can provide some confining effect to improve the ductility, its effectiveness reduces significantly as concrete strength increases. When concrete strength increases beyond $f_c' = 100$ MPa, the transverse steel required to restore reasonable level of ductility is so large that will adversely affect the concrete placing quality [7–9]. Therefore, the conventional type of transverse rebars is not appropriate for confining UHSC columns.

To possibly push up the limit of concrete strength that can be used in practical column construction, special type of confinement that enables the provision of larger, more uniform and continuous confining pressure is required. For this reason, concrete-filled-steel-tube (CFST) column that consists of a hollow steel tube with concrete filled inside was advocated [10–13]. From structural point of view, the steel tube provides both axial, flexural and shear strength to the column and confining pressure to the in-filled concrete that enhances the overall strength-to-weight ratio. This can be shown by the improved resistance under uni-axial compression [14–26], uni-axial tension [27], torsion [28], pure bending [29–31], combined axial compression and bending [32–34] and in extreme situation, such as fire [35–38] and earthquake [39]. From materials' point of view, CFST column is particularly suitable for high-strength concrete (HSC) and UHSC columns. As HSC and UHSC are significantly more brittle than normal-strength concrete (NSC) [40], HSC and UHSC columns need to be provided with more confinement in order to avoid brittle failure under combined axial load and bending [1,9,41,42] during earthquake attack. By confining the in-filled concrete through the use of steel tube, brittle failure mode of HSC and UHSC columns can be prevented. From practical construction point of view, the steel tube acts as formwork such that no external formwork for concreting is required. It saves the construction materials and shortens the construction cycle time. Considering the above advantages, CFST columns are becoming increasingly popular.

Despite the above advantages, a major limitation of using CFST columns is the imperfect steel–concrete interface bonding that occurs during initial elastic phase. There are mainly 3 different conditions of debonding in CFST structures: (1) due to poor concrete placing (e.g. segregation and honeycombing); (2) due to shrinkage and temperature variation

* Corresponding author.
E-mail addresses: johnny.ho@hku.hk (J.C.M. Ho), lmh58743@hku.hk (M.H. Lai).

0143-974X/$ – see front matter © 2013 Elsevier Ltd. All rights reserved.
http://dx.doi.org/10.1016/j.jcsr.2012.12.014

in the core concrete; (3) due to larger dilation of steel than concrete at the initial stage under compression because of different Poisson's ratios (0.2 for concrete and 0.3 for steel) [43–45]. The imperfect interface bonding will decrease the confining pressure provided to the concrete, and reduces the elastic strength and stiffness. One of the consequences is the increase in the inter-storey and/or overall drift ratio of tall buildings, which may violate the serviceability limit state requirement. Also, the reduced interface bonding would increase the chance of having tube buckling at elastic stage if the wall is thin compared with its diameter [46]. This problem has drawn the attention of more and more researchers in recent years. For example, it is reported that the difference in uni-axial strength between bonded and unbonded HSCFST columns could be up to 17% [18]. Liao et al. [47] concluded that with circumferential gap ratio 2.2%, ultimate strength of CFST columns could decrease by 29% and the ultimate moment of CFST beams by 34%. Moreover, Xue et al. [48] indicated that the debonding would take adverse effects on the ultimate load and ductility of CFST stub columns and the ultimate load could reduce by 16.9%.

Mainly two types of methods have been proposed in the past to improve the condition of imperfect steel–concrete interface bonding of CFST columns during compression. The first one is to adopt expansive concrete [49–52], which compensates the shrinkage and temperature variation effect. However, when in compression, steel tube still dilates more than concrete, which will reduce the confining pressure in the expansive concrete. The second type is to change the cross-section shape of the steel tube or adding additional restraint, which included stiffeners [15,19,53] and binding bars [54]. Nonetheless, it is convinced that the installation and fabrication of internal tab stiffeners was quite difficult, especially when the column size is relatively small. The installing of binding bars required welding on the stiffening plate, the quality of which is difficult to control, which was shown in the Specimen C1 [54].

In this paper, the authors proposed to provide external tie bars threaded into the steel tube and tightened by nuts against the external face of CFST columns. The improved behaviour was investigated by a series of uni-axial compression test on 30 CFST columns with and without tie bars based on axial load-carrying capacity, elastic stiffness and Poisson's ratio. From the test results, it is clear that (1) tie bars can improve slightly the axial load-carrying capacity. (2) Tie bars are ineffective in increasing the initial stiffness. (3) Tie bars are effective to limit the lateral deformation of the core concrete and the steel tube at the locations of the tie caps, but the effectiveness reduced in both the horizontal and vertical planes between caps. (4) The installation of tie bars needs to drill holes on the steel tube which destroy the column profile. It initiated steel cracks in longitudinal directions under large axial strain and limited the strength and stiffness enhancement. Lastly, based on the test results, an analytical model taking into account the confining effects provided by the steel tube and ties is developed to estimate the axial load-carrying capacity of unconfined and tie-confined CFST columns. The validity of the analytical model is justified by comparing the predicted theoretical strength with the measured strength in the tests.

2. Experimental test programme

2.1. Specimens

A total of 30 CFST columns with different cross-sections and material properties have been fabricated and tested under uni-axial compression. The CFST column specimens are divided into 5 groups depending on the concrete grades and steel tube thickness: (1) 6 CFST with concrete strength of 30 MPa and steel tube thickness of 5 mm.; (2) 6 CFST columns with concrete strength of 30 MPa and steel tube thickness 8 mm; (3) 6 CFST columns with concrete strength of 80 MPa and steel tube thickness of 5 mm; (4) 6 CFST columns with concrete strength of 80 MPa and steel tube thickness of 8 mm. (5) 6 CFST columns with concrete strength of 120 MPa and steel tube thickness of 5 mm. There was one unconfined specimens (i.e. without tie bars) in each group, which was to serve as control for comparison purpose. The rest of the specimens were installed with tie bars provided at different spacing (i.e. 5*t*, 10*t*, 12.5*t*, 15*t* and 20*t*, where *t* is the thickness of the steel tube and is 5 or 8 mm in this study). The grade of steel tube is S355 produced as per BS EN 10210-2:2006. The actual strength of the steel tube is 365 MPa. The diameter of the external steel tube and height of the CFST columns are exactly the same for specimens in Groups 1 to 4, which are 168.3 mm and 330 mm respectively, and for specimens in Group 5, they are 114.3 mm and 248 mm. Fig. 1 shows the photos of typical CFST columns (Group 3). The diameters of all tie bars are 8 mm and the nominal yield strength is 250 MPa. The ties adopted in this test were all threaded. Also, they were fabricated to be slightly longer than the outer diameter of the steel tube so that nuts could be installed at both of its ends to tighten them against the external face of the steel tube. The nuts were installed just tight and therefore no initial pre-stressing force was applied to the steel tube. At each level, a pair of steel ties (with a level difference of a tie diameter) was installed perpendicular to each other. The pair of ties was then rotated by 45° at the next level and the arrangement continued for the subsequent layers of ties. There was 1 mm tolerance for the hole drilled on the hollow steel tube for accommodating the ties. Details of the CFST specimens with tie bars are shown in Fig. 2.

A naming system was established to identify each of the specimens. It consisted of two alphabets and four numbers. For example, CT5-5-168-30 represents a CFST column specimen (indicated by the first letter “C”) confined by tie bars (indicated by the second letter “T”). The spacing of the tie bars is 5 times the thickness of the steel tube, i.e. 5*t*, (indicated by the first number “5”). The thickness of the steel tube is 5 mm (indicated by the second number “5”). The outer diameter of the steel tube is about 168 mm (indicated by the third number “168”) and lastly, the concrete cylinder strength is 30 MPa on the testing day (indicated by the last number “30”). The CFST column without the provision of tie bars is represented by CN0-5-168-30, where N0 stands for “no confinement”. Table 1 summaries the properties of the specimens.

2.2. Instrumentations

In this experiment, the compression machine SATEC Series RD Model with maximum load and travelling displacement of 5000 kN and 100 mm respectively was adopted for the uni-axial compression test. The details of the test set-up and instrumentation are shown in Fig. 3a to c. Three linear variable differential transducers (LVDTs) with 100 mm stroke were installed to record the full length axial displacement of the CFST columns by measuring the relative displacement between the top and bottom loading platens. Three numbers of two-directional strain gauges (Tokyo Sokki Kenkyujo Co., Ltd) were installed at around the mid-height of the external face of the tested specimens, which were 120° separated from each other at the centre of the section to measure the longitudinal and transverse strains. A circumferential extensometer with maximum measuring range of 6 mm was installed between two adjacent rows of the tie bars, to be exact, just below the tie bars to measure the tube dilation near the tie bars during the initial elastic stage. The extensometer would be removed when the lateral dilation was about to reach 6 mm.

2.3. Testing procedure

All the hollow steel tubes and CFST columns were tested under displacement control. For CFST columns, the top surface was packed with a layer of gypsum before initial compression was applied to