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Numerical modelling and performance assessment of FRP strengthened full-scale circular hollow section steel columns subjected to vehicle collisions

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ABSTRACT

Axial load-bearing structural members often experience significant damage or failure when subjected to moving vehicle or vessel collisions. Hollow steel tubular columns are highly vulnerable under transverse impact loading. Thus, strengthening/retrofitting of existing steel tubular columns may be required if these members are not designed to withstand expected transverse impact from transport accidents. This paper investigates the performance of FRP strengthened full-scale circular hollow section (CHS) tubular columns subjected to vehicular impact. Initially, finite element (FE) models of bare and fibre-reinforced polymer (FRP) strengthened CHS medium-scale specimens were developed to conduct transverse impact analysis for the model validation purpose. The impact simulation results were compared with the drop mass impact test results and good agreements were found between the FE and experimental tests. The validated FE models were extended to full-scale bare and FRP wrapped CHS columns. The full column-vehicle collisions were simulated using a realistic vehicle model by considering varying axial static forces and vehicle impact velocities. The results showed that CFRP strengthening improved the impact resistance capacity of a bare CHS column by preventing plastic hinge formation due to excessive local buckling subjected to accidental vehicular impact. Three-layer CFRP strengthening proved to be an effective strengthening system compared to two-layer CFRP strengthening system. The effect of load eccentricity was assessed further and found that CFRP strengthening significantly contributed to prevent the failure of CHS columns with varying eccentricities subjected to credible vehicular impact events.

KEYWORDS

Circular hollow section (CHS) steel members, FRP, CFRP, transverse impact, full-scale column, vehicular impact.

Introduction

Columns and bridge piers are critical elements of a structure against dynamic imposed loads, as failure of these members may cause catastrophic failure of the whole, or part, of the structures. Hollow steel tubes have been used widely as axial load-bearing members in bridge piers, building columns and offshore structures due to their lightweight and fast installation advantages. The transverse impact from moving vehicles or ships could cause significant damage or complete failure of these tubular columns. A traditional strengthening method, such as welding additional steel plates, will increase the chance of corrosion degradation of the existing columns. Other options, such as pouring concrete into tubular columns to avoid local buckling failure of columns under transverse impact from moving vessels/vehicles, may not be suitable in the case of offshore structures in the deep ocean and structures in service condition.

Thus, there is a growing need within the structural engineering research community to identify the most suitable technique of strengthening hollow steel columns subjected to extreme loading such as vehicular collisions.

Strengthening reinforced concrete (RC) structures using fibre-reinforced polymer (FRP) is a common practice in the construction industry due to the several unique advantages of FRP materials. In recent years, extensive research works have been carried out to investigate the performance enhancement of FRP strengthening steel plates and members subjected to static loads (Zhao and Zhang 2007; Okeil et al. 2009; Dawood and Rizkalla 2010; Fawzia et al. 2010; Teng et al. 2012; Fawzia 2013; Kabir et al. 2016a, 2016b, 2016c, 2016d, 2016e; Zheng and Dawood 2016; Batuwitage et al. 2017; Ulger and Okeil 2017; Liu and Dawood 2019). The improvement of the axial static load-carrying capacity of carbon fibre-reinforced polymer (CFRP) strengthened hollow tubular columns has been evaluated in several past works (Shaat and Fam 2006; 2007; Haedir and Zhao 2011). However, research is very limited on the performance of FRP strengthened steel members subjected to dynamic loadings such as transverse impact, compared to strengthened steel members subjected to static loading. The joint behaviours of CFRP-steel plate joints under tensile impact have been studied both experimentally and numerically (Al-Zubaidy 2012a, 2012b, Al-Mosawe 2016). Very recently, the dynamic behaviour and impact resistance of FRP strengthened hollow and concrete-filled steel tubular members were investigated extensively through experimental impact tests and FE analysis (Alam et al. 2014, 2015a, 2015b; Alam and Fawzia 2015; Alam et al. 2016; Al-Husainy et al. 2016; Shakir et al. 2016; Alam et al. 2017a, 2017b, 2017c; Kadhim et al. 2018a; Kadhim et al. 2018b; Saini and Shafei 2019).

In most of the cases, it is not possible to conduct experimental tests of full-scale specimens due to the high costs associated with the laboratory test setup. In addition to that, CFRP is expensive when strengthening a large surface area. Thus, FE numerical simulation can be an alternative option to assess the performance of FRP strengthened full-scale columns due to vehicle collisions. Full-scale FE modelling and impact analysis of RC and steel columns have been investigated in a number of research works (El-Tawil et al. 2005; Ferrer et al. 2010; Thilakarathna et al. 2010; Sharma et al. 2012; Makarem and Abed 2013; Al-Thairy and Wang 2014, Abdelkarim and ElGawady 2017; Do et al. 2018a; Do et al. 2018b). Abdelkarim and ElGawady (2016) conducted vehicular impact simulation of full-scale hollow-core FRP–concrete–steel bridge columns. Several early works have shown that hollow tubular members are vulnerable under transverse impact loading ((Bambach et al. 2008; Remennikov et al. 2011; Yousuf et al. 2013, 2014). Thus, strengthening of such load-bearing structural members may be required if they are not designed to withstand transverse impact loads from moving vehicles or vessels. The recent experimental work of Alam et al. (2017b) clearly indicated that FRP strengthening can improve the performance of scaled CHS tubular members subjected to drop mass impact loading. However, to understand the true contribution of the FRP strengthening of hollow tubular members, it is important to study their full-scale structural behaviour when subjected to credible impact events. The above literature reveals that no work has previously studied the performance of FRP strengthened full-scale circular hollow section (CHS) columns subjected to vehicular impact.

The objective of this research is to evaluate the performance enhancement of FRP strengthened columns subjected to realistic vehicle impact. Firstly, numerical models of a total of twelve bare and FRP strengthened medium-scale CHS members (one bare member and eleven strengthened members) were developed using ABAQUS (SIMULIA 2011). Lateral impact simulation was conducted at the mid-span of the specimens to validate the FE models using drop hammer impact test results presented in a recent work by the authors (Alam et al. 2017b). The validated FE models were further used to develop full-scale ground floor level columns of a typical low-rise building. A simplified vehicle model was also modelled to simulate realistic vehicular impact by considering both column and vehicle deformations during impact events. The performance improvements of wrapped columns were assessed by comparing the axial static load-bearing capacities of bare and strengthened columns subjected to vehicular impact with different vehicle speeds. Furthermore, the effect of eccentric axial loading was also investigated by considering a wide variation of axial load eccentricity.

Test specimens and experimental program

The test specimens of a circular hollow section (CHS) steel tube were prepared by strengthening with externally bonded FRP sheets. Detail of the laboratory test procedure and results can be found elsewhere (Alam et al. 2017b). All the CHS steel test specimens were supplied by OneSteel Limited, Australia and were of identical dimensions, with 1600 mm of length, 114.5 mm of outer diameter and 4.5 mm wall thickness. Two different FRP types, i.e. carbon fibre-reinforced polymer (CFRP) and glass fibre-reinforced polymer (GFRP) sheets, were selected to strengthen CHS steel specimens using structural epoxy adhesive. The CFRP sheet and epoxy adhesive were provided by BASF Construction Chemicals Australia Pty Ltd, while the GFRP sheets were obtained from CG Composites, Australia. Variations of wrapping direction of fibre, wrapping layers and wrapping length were considered in the test program. Details of the test specimens are listed in Table 1. The longitudinal FRP layers are effective in controlling global deformation, whereas hoop layers are better to control local deformation subjected to lateral impact loading (Alam et al. 2017b). The first part of specimens' label indicates material types (S = steel, C = CFRP, G = GFRP, and GC = both GFRP and CFRP). The second part represents strengthening configuration (B = bare specimens, L = longitudinal FRP direction, and H = hoop FRP direction). The number of FRP layers can be counted from the number of letters. The final part denotes impact velocity (*V1*= 3.3 m/s and *V2* = 3.6 m/s).

All test specimens were one-third scale (approximately) members for validation of FRP strengthened CHS FE models subjected to lateral impact loading.

Lateral impact tests were performed at the mid-span of the specimens using a drop hammer impact testing machine. The instrumented drop hammer facility was equipped with 592 kg of falling mass and dynamic load cell. The height of the drop hammer was adjusted to 3.3 m/s and 3.6 m/s impact velocities, respectively. Clear span of the specimens was kept as 1300 mm with simply supported ends condition. Fig. 1 shows the schematic view of experimental setup. The impact velocity was changed for one specimen to validate the experimental test results in different impact velocities.

Numerical modelling

General

The FE models of bare and FRP strengthened CHS columns were developed and dynamic impact analyses were performed using ABAQUS/Explicit (SIMULIA 2011). The full length 3-D numerical models of the test specimens listed in Table 1 were built for the model validation purpose. The length, outer diameter and wall thickness of CHS steel tube in all the models were kept the same, as described in the previous section. It was observed from the specimen preparation and specimen testing stages that for one-layer, two-layer and three-layer FRP wrapped specimens, the epoxy-cured FRP layers formed a single composite plate after curing. The failure modes from the test results also revealed that for two- and three-layer wrapped specimens, FRP laminates debonded as one composite plate rather than two or three different laminate layers. Thus, interlayer delamination failure was not observed from the experimental tests. To incorporate this behaviour of FRP composites, an equivalent single FRP layer was modelled for both CFRP and GFRP laminates as shown in Fig. 2. It was also noticed that a very thin film of adhesive layer was formed for all the test specimens between the composite

FRP plate and the outer steel surface. Thus, the FE model of a FRP wrapped specimen consisted of three parts: steel tube, adhesive layer, equivalent single FRP layer. The average thicknesses of one, two and three layers of FRP wrapping were measured, as listed in Table 2. The thickness of epoxy saturated one-layer CFRP sheet was found as 0.5 mm when it was cured on flat metal sheet for material testing. However, the average thickness increased to 1.4 mm when it was wrapped and cured on the outer surface of CHS members. This is because in CHS, impregnated adhesive material tries to flow downward due to gravity until it reaches an initial set position. Although, all the steps were taken to rib-roll the saturate CFRP sheet uniformly and remove any excess adhesive, it was not possible to achieve uniform thickness on the curved outer surface of CHS members. Thus, the thickness of cured wrapped FRP plate was higher than the cured FRP plate on plane sheet. To avoid complexity, uniform FRP and adhesive layers were considered in FE modelling. To maintain consistency, the thicknesses of one FRP laminate and one adhesive layer were considered as 0.7 mm and 0.1 mm, respectively. Thus, for two FRP laminates, equivalent single layer thickness was found as 1.5 mm (thicknesses of two FRP laminates plus one adhesive layer between the laminates). In a similar manner, equivalent single FRP layer thickness for three FRP laminates was estimated as 2.3 mm (thicknesses of three FRP laminates plus two adhesive layers between the laminates). Selection of single-layer FRP thickness as 0.7 mm provided good agreement between the FE and the test results, despite the variation of measured and FE thickness in the case of one-layer strengthened member. Earlier studies have shown that epoxy adhesive thickness varies between 0.1 mm to 0.3 mm for FRP strengthening of metal plates and tubes (Fawzia et al. 2005, 2006; Bambach et al. 2009; Bambach 2010).

The appropriate element selection is essential in FE modelling to capture the actual behaviour of the test specimens. The dynamic impact simulation of FRP strengthened specimens are challenging due to the highly brittle behaviour of FRP laminates and adhesive layers under direct dynamic compressive loading. In this study, an equivalent FRP layer was modelled using 8-node quadrilateral continuum shell element (SC8R) in ABAQUS/Explicit. This type of element was used in early works to predict the behaviour of FRP composites (Al-Zubaidy et al. 2012b; Alam et al. 2015a; Faggiani and Falzon 2010; Smojver and Ivančević 2011). The debonding between the steel surface and FRP layer was simulated by introducing an adhesive layer between FRP and steel surfaces. The 8-node 3-D cohesive element (COH3D8) was adopted to model the adhesive layer. The CHS steel tube specimens were modelled with 8-node linear brick element with reduced integration and hourglass control (C3D8R). The falling mass (weight block) and the impactor were modelled as a rigid mass using 8-node solid elements. The dimension of the weight block was $1000\ mm\ \times 750\ mm\ \times 100\ mm$ whereas the cylindrical impactor was $100\ mm$ in diameter and $50\ mm$ in height (Yousuf et al. 2012). The densities of the weight block and impactor were adjusted to get a total mass of 592 kg of the drop hammer facility. Fig. 3 shows the detail of various parts of FE modelling of the FRP wrapped specimen subjected to transverse impact loading. The simply supported boundary conditions were assigned at the bottom side of the FE models to keep consistency with the experimental setup (Fig. 3).

FE meshing and contact modelling

For high strain rate dynamic loading, the mesh sizes of FE models should be selected properly to accurately capture the structural behaviour. In this model validation work, relatively dense meshing (smaller mesh size) was adopted at impact location compared to the away-from-impact location. The impact zone was modelled by using 5 mm size elements and away-from-impact zone was modelled by using elements of 10 mm size. A mesh sensitivity analysis was performed before selecting the above-mentioned mesh sizes. To avoid the excessive distortion of FRP and adhesive parts, the damage degradation parameter was set to 0.99 and element deletion was activated.

The cohesive element with finite thickness (0.1 mm) was placed between the steel tube outer surface and FRP layer inner surface. The tie constrain was used to establish contact between the steel tube, adhesive and FRP layer. The outer surface of the steel tube was attached with the inner surface of the adhesive layer. The inner surface of the CFRP layer was tied together with the outer surface of the adhesive layer. The general contact method was used to simulate the contact interactions between the outer surface of the impactor and FRP layer, adhesive layer and outer surface of steel tube. The normal behaviour of interaction properties was defined by “Hard” contact, and “Penalty” friction formulation was adopted to define tangential behaviour. A friction coefficient value of 0.5 was selected for this study and closely matched with the previous studies (Alam and Fawzia 2015; Alam et al. 2015a).

Material models

FRP

The damage model of fibre-reinforced composites from the ABAQUS library was adopted to simulate the failure and damage of FRP materials. Due to the constitutive material behaviour of FRP laminates (elastic-brittle), the failure of these materials can initiate without any significant plastic strain. The “Lamina” type elastic properties definition was selected and the FRP failure behaviour was modelled using “Hashin” failure criteria (Hashin and Rotem 1973; Hashin 1980). The detail of the FRP layer modelling technique can be obtained from an earlier study (Alam et al. 2015a). The tensile mechanical properties of unidirectional CFRP and GFRP laminates were obtained through static tensile coupon tests. The tensile coupon specimens were fabricated and static tensile loading was applied as per ASTM D3039 (ASTM 2008). The standard deviation and coefficient of variation (COV) of material properties obtained from a set of three identical coupon specimens (three CFRP coupons for CFRP material properties and three GFRP coupons for GFRP material properties) are presented in Table 3. The manufacturer tested values were obtained from dry CFRP and GFRP (no epoxy saturated) tensile coupon tests, whereas coupon tests in this study were conducted with the impregnated CFRP and GFRP laminates. Table 3 shows that the thickness of impregnated CFRP was significantly higher than the dry CFRP. Thus, the tensile strength and modulus values of tested FRP composites were much lower than the manufacturer tested values. Fig. 4 depicts the typical stress-strain relationships of CFRP and GFRP laminates from material testing. FRP laminates and adhesive layers were combined to represent two and three FRP layers in FE models. Thus, equivalent tensile strength and tensile modulus of the FRP layer were calculated using the following equations, as proposed in the literature (Al-Zubaidy 2012b; Fawzia et al. 2006):

$$\sigma_{equ\,(FRP\,layer)} = \frac{\sigma_{FRP\,laminates} \times t_{FRP\,laminates} + \sigma_{Adhesive\,layer} \times t_{Adhesive\,layer}}{t_{equ\,(FRP\,layer)}} \quad (1)$$

$$E_{equ\,(FRP\,layer)} = \frac{E_{FRP\,laminates} \times t_{FRP\,laminates} + E_{Adhesive\,layer} \times t_{Adhesive\,layer}}{t_{equ\,(FRP\,layer)}} \quad (2)$$

Here $\sigma_{equ\,(FRP\,layer)}$, $E_{equ\,(FRP\,layer)}$, $t_{equ\,(FRP\,layer)}$ are tensile strength and tensile modulus, and thickness of equivalent FRP layer, $\sigma_{FRP\,laminates}$ and $\sigma_{Adhesive\,layer}$ are tensile strengths of FRP laminates and adhesive layers obtained from the material testing, $E_{FRP\,laminates}$ and $E_{Adhesive\,layer}$ are tensile moduli extracted from stress-strain relationships of material testing, $t_{FRP\,laminates}$, and $t_{Adhesive\,layer}$ are thicknesses of FRP laminates and adhesive layer. The material properties of the FRP layer were also calibrated to consider the effect of combined longitudinal and hoop layers. The bidirectional properties of FRP materials were initially approximated for FE modelling. Sensitivity analysis was performed for every model containing bidirectional FRP, to obtain material properties. The bidirectional tensile strengths of two-layer CFRP and two-layer GFRP materials used in FE modelling were 600 MPa and 390 MPa, respectively. No attempts were made to consider the dynamic material properties of the equivalent FRP layer. Although previous studies showed enhancement of strength and elastic modulus under high strain rate tensile coupon tests of FRP materials (Al-Zubaidy 2012b; Zhang

2016), the strain rate effect was negligible in the case of CFRP strengthened concrete-filled steel tubular (CFST) columns as reported in Alam et al. (2015a).

Adhesive

A thin epoxy adhesive layer was modelled for all the specimens to simulate FRP debonding from the steel surface (Figs. 2 and 3). The cohesive elements were deployed to model the adhesive zone with traction-separation law. The details of cohesive zone modelling can be found elsewhere (Alam et al. 2015a). The material properties of epoxy adhesive were tested by Kabir et al. (2016d) as listed in Table 3. The strength, mode I and II elastic stiffness and fracture energy properties provided in Kabir et al. (2016d) were used for adhesive modelling in this work (Table 3).

Steel

The elastic-plastic behaviour of steel material was modelled by an isotropic classic metal plasticity model. The standard deviation and COV of mechanical properties of steel material were obtained through testing of five identical coupon specimens according to AS 1391 (AS 2007). Table 3 lists the yield stress, modulus of elasticity and tensile strength results obtained from steel coupon tests. The strain rate of steel material was considered by adopting the Cowper-Symonds power law relation with multiplier factor of 40.4^{-1} and exponent of 5 (Jones 1997).

Model validation

The transverse impact simulation was performed to validate the FE models by comparing the structural responses and failure modes of bare and strengthened FE models with test results. Good agreements between lateral displacement-time curves were noticed for all the test specimens, as shown in Fig. 5. Both peak and residual displacements from FE analyses were well matched with the corresponding test curves. A comparison of peak lateral displacements of tests and FE results is presented in Table 4. The maximum percentage of error was only 6% with mean and COV being 1.02 and 0.03, respectively. Good agreements between tests and FE results confirmed that FE models were capable of capturing dynamic responses of bare and FRP wrapped CHS tubular members under transverse impact. The comparison of failure modes of bare and two-layer longitudinally wrapped specimens are displayed in Fig. 6. Good matching of local and global deformation patterns was noticed between the test specimens and the FE models.

Full-scale column and vehicle model

Detail of FE model

The validated FE models of bare, two-layer (first layer in the hoop direction and second layer in the longitudinal direction) and three-layer (first layer in the longitudinal direction, second layer in the hoop direction and third layer in longitudinal direction) CFRP strengthened specimens were extended to full-scale CHS column models, as shown in Fig. 7. The outer diameter, wall thickness and length of validated CHS members were increased to 355.6 mm, 12.7 mm and 4000 mm, respectively, to represent a typical ground floor level column of low-rise buildings or car parks. The outer diameter-to-wall thickness ratio and slender ratio (kL/r) of full-scale column was calculated as 28. Here k is the effective length factor and assumed as 0.85 (AS 1998); in the case of bottom end fixed condition and the top end is left free to move vertically, L is the overall length of the column and r is the radius of gyration of the column section. The design axial capacity of the column was found lower than the squashing load ($P_{Squash} = A_s \times f_y$). Here, A_s is steel area and f_y is yield stress of steel material. Thus, possible failure mode of full-scale column was expected as global buckling failure subjected to uniaxial compression. Bottom and top end plates were modelled as rigid elements to apply the boundary conditions and axial loading of the columns. Reference points were introduced to both rigid end plates to apply boundary conditions. In ABAQUS, a reference point controls the motions or rotations of rigid body and the relative positions of associated nodes and elements remain constant during the analysis (SIMULIA 2011). The boundary conditions were applied at bottom and top ends of column models to represent typical CHS columns in low-rise buildings or car parks. Fixed support was assigned at the bottom of the columns by restraining translations and rotations in all three directions. The top end reference point was only allowed to translate in Y axis and rotate around Z axis to allow axial shortening of column due to lateral impact loading (Fig. 7(a)). A simplified vehicle model of a Chevrolet C2500 pick-up was modelled to simulate a realistic vehicle model, as shown in Fig. 7. The weight of the vehicle was considered as 1840 kg, which is similar to the actual weight of an empty Chevrolet C2500 pick-up (Al-Thairy and Wang 2014). The vehicle mass was kept the same (1840 kg) for all the analyses as the effect of vehicle mass is not a focus of this study. According to Eurocode 1 Part 1.7 (CEN 2006), the impact force is approximately proportional to the square root of the vehicle mass. The model validation process involved validation of vehicle frontal deformation and impact force characteristics compared to actual vehicle frontal crush test results and previous FE results (Alam et al. 2016). Thus, the mass-spring model is capable of capturing frontal deformation and impact force properties of full-scale vehicles. The bilinear impact force-vehicle crush distance behaviour of the simplified vehicle model was simulated by using initial and final stiffnesses of vehicle front k_1 and k_2, respectively. The value of k_1 was calculated from the equation proposed by Al-Thairy and Wang (2014) whereas k_2 value was kept the same as Alam et al. (2016). The detail of the modelling and model validation process of this mass-spring vehicle can be found in a recent study by the authors (Alam et al. 2016). It should be noted that the current bilinear vehicle model is only valid for the column diameter used in this study. For different vehicle types and column diameters, the bilinear model needs to be calibrated using the relevant vehicle crush test results and column geometric information. The design axial capacity of a full-scale column was calculated according to AS 4100 (AS 1998). Different mesh sizes were adopted in different zones of column models to predict the behaviour of members subjected to transverse vehicular impact. The mesh sensitivity study was performed to select the suitable mesh sizes in impact location and away from impact location. Fine mesh with mesh size 10 mm along the length of the columns was adopted up to 1000 mm length of the columns from the bottom end, as shown in Fig. 7(b). The rest of the column length was modelled using 50 mm mesh size along the length up to the top end. The vehicle front bumper was kept at a height of 800 mm, which is nearly similar to the front bumper of a real vehicle.

Application of axial static load

The axial preloading was applied to the rigid body top end plate of the columns as concentrated loading in a separate quasi-static analysis step in ABAQUS/Explicit. The axial static preloading was applied to simulate the realistic service static loading of a column. The smooth amplitude function was used to apply the static load as shown in Fig. 8. Table 5 presents the axial loads with percentages of design axial capacity applied as service static loads to the columns. The applied axial load was varied between 25-50% of the axial capacity of the bare CHS column. Generally, columns are designed to resist typical service load of 40-50% of their axial capacities. Thus, the maximum applied axial preloading was 50% of the design capacity of the CHS column. The numerical simulation of axially preloaded columns under vehicular impact loading was conducted in two steps. During the first step of analysis, axial loading was applied, using the smooth amplitude function available in ABAQUS, during the lower natural period (0.025 s) of the bare column as shown in Fig. 8 to achieve required axial preloading. At time t=0.025 s, impact loading step was started and the simplified vehicle impacted axially preloaded columns. For more details about the axial load application process, readers are referred to the early studies of the authors (Alam and Fawzia 2015; Alam et al. 2015a).

Performance assessment of strengthened columns

Three columns: S-B (bare), C-HL (two-layer) and C-LHL (three-layer) were considered in an impact collision simulation of this study. A hoop layer was introduced in both two- and three-layer strengthened columns due to the advantages of improved structural performance observed in the experimental tests. In this performance assessment study, it was assumed that the columns were external ground-floor columns of a low-rise building in a typical suburb or central business district (CBD) area in Australia, with expected vehicle speed between 40 km/h to 60 km/h. Figs. 9-11 show the axial load-time responses of bare and strengthened columns subjected to different design axial capacities and vehicular speeds. With 25% design axial loading all columns exhibited stability by successfully carrying the applied axial load subjected to 40 km/h, 50 km/h, 55km/h and 60 km/h, vehicular speeds, respectively. Fig. 10(d) shows that FRP strengthening was effective to prevent failure of three-layer FRP strengthened columns subjected to 60 km/h impact velocity at 40% of design axial load. Bare and two-layer FRP strengthened columns were found failed with drop of axial loading under same loading configuration. Bare and three-layer strengthened columns showed stability with 50% design axial loading and subjected to 50 km/h impact velocity but the two-layer strengthened column failed with gradual dropping of axial loads, as shown in Fig. 11(b). Thus, the two-layer CFRP strengthening column performed worse than the bare specimen (Fig. 11(b)) when axial loading increased to 50% design axial load and impact velocity increased to 50 km/h. This could be due to the excessive debonding failure of two-layer CFRP wrapping, which occurred within a very short time after impact with an increased axial loading and impact velocity. Experimental tests of CFRP strengthened CHS and CFST members under lateral impact showed that with the increase of impact velocity, effectiveness of the two-layer CFRP strengthened member reduced due to CFRP debonding at impact location (Alam et al. 2017(b), (c)). In order to avoid such premature failure, it is recommended that at least three-layer CFRP be applied in strengthening CHS members subjected to lateral impact loading. At 50% design axial loading all three columns failed when impact velocity increased to 55 km/h and 60 km/h, respectively (Fig. 11 (c) and (d)).

It has been observed that during an impact event, sudden change in axial loading time histories was noticed during the early stage of impact loading, as shown in Figs. 9-11. Axial force and axial shortening time histories of three-layer FRP strengthened column with 40% design axial load and 50 km/h impact velocity are plotted in Fig. 12 to understand such rapid change. During the first 0.025 s of analysis, axial load and axial displacement (shortening) increased almost linearly as initial preloading was applied, using the smooth amplitude function in ABAQUS. From 0.025 s to 0.075 s of analysis time, axial displacement was nearly constant. This is because, during the early stage of vehicle-column collision, the vehicle front bumper deformed rapidly to allow crushing of the front bumper before reaching the stiffer part (engine box) (Alam et al. 2016). At time t=0.075 s, the rate of axial displacement increased sharply as the vehicle engine box contacted with the column. It can be noted that high fluctuation in axial load time history was recorded during this time (Fig. 12). Such rapid variation in axial load response was stabilised after time, t=0.175 s as the vehicle separated from the column at this time. The column axial deformation was in downward direction until the separation of the vehicle from the column. Thus, there was no vertical upward movement of the column to cause a sudden drop in axial load level. The deformation behaviours of bare and FRP strengthened columns are shown in Fig. 13 at time t=0.085 s during large axial load variation. The contours in all the failure mode figures are von Mises stress distribution and the stress unit is in N/m^2. All three columns were found stable with no large lateral displacement despite local inward buckling due to vehicular impact. The large variation in axial load response of columns within a very short period of time may be contributed by the impact force from the moving vehicle and excitation of the system in a vertical direction. Similar phenomena in axial load time responses during the early stage of impact loading were recorded in experimental tests of axially loaded concrete filled double skin tube specimens subjected to lateral impact loading (Aghdamy et al. 2016).

The deformation behaviours of bare and CFRP wrapped columns subjected to 60 km/h vehicular velocity are shown in Fig. 14. Both bare and two-layer CFRP strengthened columns have shown large local deformation with formation of plastic hinges at impact location and bottom support at time t=0.16 s with static loading of 40% design axial capacity. The three-layer CFRP wrapped column only showed inward local deformation at impact location with no significant deformation at bottom support at time t= 0.16 s (Fig. 14). At time t=0.175 s, the member failure of bare and two-layer CFRP strengthened columns was more prominent with further lateral displacement and axial shortening. On the other hand, the three-layer CFRP strengthened column exhibited no further deformation and carried axial service load (40% design capacity) successfully at t=0.175 s. The deformations of bare and two-layer strengthened columns continued with the increase of analysis time. But no further lateral deformation was noticed for the three-layer strengthened column. Fig. 10(d) also confirms this observation, as axial load carrying capacities of bare and two-layer strengthened columns dropped at impact time t=0.175 s while the three-layer strengthened column continued to carry applied axial load after the end of the vehicle impact collisions. Thus, CFRP strengthening improved the vehicular impact resistance capacity of the full-scale CHS column with service static loading. CFRP cracks, debonding, and matrix fracture failure were noticed at impact location and bottom support of strengthened columns.

Effect of load eccentricity

In a practical situation, full-scale columns experience axial load eccentricity. To investigate the effect of axial load eccentricity of bare and CFRP strengthened CHS columns, the position of concentrated load was varied, as shown in Fig. 15. The eccentricities *E1*, *E2* and *E3* were 22 mm, 44 mm and 66 mm, respectively, from the centre at the impact side of the plate. Similarly, *E4*, *E5* and *E6* were located at the opposite side of impact with the distance of 22 mm, 44 mm and 66 mm, respectively, from the centre of the plate. The axial load eccentricity was considered on both sides of the impact, to observe the effect of both positive and negative moments on the behaviour of impacted columns due to axial load eccentricity. Eccentricity for tubular columns is normally described as the ratio of eccentricity (*e*) to the outer diameter of the tube (*D*), i.e. (*e/D*). The *e/D* ratio in this study varies from 0.06 to 0.19. The level of eccentricity selected in this study belongs to low-to-medium level according to the definition in previous studies by Han et al. (2003) where *e/D* varied from 0 to 0.3 and Moliner et al. (2015) where *e/D* varied from 0.13 to 0.31. Fig. 16 shows the effects of load eccentricity on the axial load responses of a bare column subjected to 50 km/h and 60 km/h impact velocities, respectively. The applied axial loads were changed from 40% to 50% of the design axial capacity of the bare column. It can be seen in Fig. 16(a) that the bare column showed stability by successfully carrying the axial service loading without a sudden drop of axial loading at *E0*, *E1*, *E2*, *E3*, *E4*, *E5* and *E6* eccentricity when the axial load was 40% of design axial capacity. With the increase of design axial capacity to 50%, the bare column failed at *E1*, *E2* and *E3* eccentricity but showed stability at *E0*, *E4*, *E5* and *E6* eccentricity (Fig. 16(a)). Thus, bare CHS columns were shown vulnerable when axial load eccentricities were at the same side of impact loading. This is because the moments produced due to impact force and axial load eccentricity were both in the same direction, which contributed to the failure of the CHS column. However, moment produced due to eccentricities *E4*, *E5* and *E6* was opposite of the moment produced due to vehicular impact loading. Thus, axial load eccentricity on the opposite side of impact loading aided in controlling the failure of the columns due to lateral impact. With the increase of vehicle velocity to 60 km/h, the bare CHS column failed with catastrophic collapse at both

40% and 50% design axial loading as shown in Fig. 16(b). The effect of axial load eccentricity on three-layer CFRP strengthened columns, subjected to 50 km/h and 60 km/h impact velocities, are shown in Fig. 17. The CFRP strengthened column has shown stability in all eccentric loading conditions subjected to 50 km/h vehicular impact, as depicted in Fig. 17(a). This indicates the significant improvement of impact resistance capacity of CFRP strengthened CHS columns by preventing failure at *E1*, *E2* and *E3* axial load eccentricities (Fig. 17(a)). CFRP strengthening was also effective with the increased impact velocity to 60 km/h as the 40% design axially loaded column showed stability at axial load eccentricities of 22 mm and 44 mm, opposite to the impact side. No effect of CFRP strengthening was noticed with axial preloading of 50% of design capacity and impact velocity of 60 km/h (Fig. 17(b)).

Table 6 lists residual axial capacities and average peak lateral displacements at impact location of bare and strengthened columns, with 50% design axial loading and 50 km/h vehicle speed. The residual axial capacities and peak lateral displacements of stable columns were of average value between 0.15 s to 0.3 s analysis time. Figs. 16 and 17, show that axial loads of stable columns were almost constant between 0.15-0.3 s analysis time. The residual axial capacities and peak lateral displacements of collapsed columns were obtained from 0.18 s analysis time. This is because it was found that the axial loads dropped suddenly during this time, indicating failure of the member (Fig. 16(a)). High lateral displacements and lower axial capacities of bare columns with eccentricities *E1*, *E2* and *E3* were due to the failure of the members subjected to vehicular impact. FRP strengthening significantly reduced the maximum lateral displacement to 21 mm for FRP strengthened columns with eccentricity *E3*. It was found that the lateral displacement of stable strengthened columns with eccentricities *E4, E5* and *E6* were higher than the bare counterparts. This could be due to the change of section properties of FRP strengthened members at impact location. From the experimental tests, it was noticed that the local deformation of FRP strengthened members were different than the bare members (Alam et al. 2017c).

The deformation behaviour of eccentrically loaded bare and strengthened columns have been displayed in Figs. 18 and 19. The failure of a bare CHS column was evident at *E1*, *E2* and *E3* eccentricities at 50% design axial capacity and 50 km/h impact loading as presented in Fig. 18. However, CFRP strengthening of such a column under the same axial static and impact loading configurations helped to avoid column failure (Fig. 19). However, additional considerations (e.g. concrete filling, safety barrier) are required to prevent failure of strengthened CHS columns (Fig. 17(b)) with 50% of design capacity and vehicle speed 60 km/h or higher. The suitability of such systems should be investigated extensively prior to recommendations.

Conclusion

In this study, FE numerical models of bare and FRP wrapped CHS columns were developed and validated with the experimental test results for reduced-scale models. The validated FE models were extended to full-scale CHS building columns to investigate the effects of CFRP strengthening subjected to vehicular impacts. The key findings and observations of this work are summarised below:

1. Three-dimensional FE models of bare and FRP wrapped CHS steel members were developed to perform lateral drop hammer impact simulation.
2. A good match for lateral displacement-time curves between FE and tests results were found as maximum peak lateral displacement variation was only 6% of test result. The failure modes of FE models also agreed well with the test specimens.
3. Bare and two-layer CFRP strengthening with hoop-longitudinal wrapping configuration showed almost similar impact resistance capacities subjected to Chevrolet C2500 pick-up impact loading. However, three-layer CFRP strengthening

with longitudinal-hoop-longitudinal wrapping configuration exhibited improved impact resistance capacity compared to the other two types of column.

4. The failure behaviour of bare and two-layer CFRP strengthened columns showed that the CHS column failed with large local deformation at impact point and bottom support. CFRP debonding failure, matrix crack and fracture and CFRP damage at impact location were observed in CFRP strengthened columns. Three-layer CFRP strengthening reduced local deformation at both impact point and bottom support thus improved the section capacity under impact loading.
5. CFRP strengthening was effective for the columns with axial load eccentricity subjected to 50 km/h and 60 km/h vehicular impact loading. Three-layer CFRP strengthening successfully prevented failure of the bare column at 40% and 50% of design axial capacities of the bare CHS column, considering different load eccentricities.

The numerical simulations presented in this study show that FE analysis can be applied successfully to investigate or design FRP strengthening system for tubular columns to prevent failure due to vehicular impact. The outcomes of this work can be used as a reference for selecting FRP wrapping scheme and orientation to maximise utilisation of such a system in field practice. The findings presented in this paper are based on the FE analysis results obtained from low-rise building columns and medium size vehicle impact. More work with different vehicle and column (e.g. bridge piers, high-rise building columns) types and sizes are required to develop design guidelines or specifications for practical application of such a strengthening system to tubular columns.

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Table 1 Details of impact test

Specimen ID	Outer diameter, D (mm)	FRP layers	FRP type	Impact velocity (m/s)	FRP bond length (mm)
S-B-V1	114.5	-	-	3.3	-
C-L-V1	117.5	1	CFRP	3.3	1.3
C-LL-V1	119	2	CFRP	3.6	1.3
C-LL-V2	117.5	2	CFRP	3.3	1.3
C-HL-V1	117.5	2	CFRP	3.3	1.3
C-LLL-V1	120.5	3	CFRP	3.3	1.3
C-LHL-V1	120.5	3	CFRP	3.3	1.3
C-HLH-V1	121	3	CFRP	3.3	1.3
G-LL-V1	117.5	2	GFRP	3.3	1.3
G-HL-V1	117	2	GFRP	3.3	1.3
GC-LL-V1	117.5	2	GFRP+CFRP	3.3	1.3
C-LL975-V1	119.0	2	CFRP	3.3	1
C-LL650-V1	119.5	2	CFRP	3.3	0.7

Table 2 Thicknesses of FRP plates and adhesive layer used in FE modelling

Number of layers	D_o(mm)	$t_{wrapping}$ (mm)	t_{FRP} (mm)	$t_{Adhesive}$ (mm)
1 layer	117	1.4	0.7	0.1
2 layers	118	1.8	1.5	0.1
3 layers	121	3.1	2.3	0.1

$D_o =$ measured average outer diameter of wrapped specimens; $t_{wrapping} =$ measured impregnated FRP thickness; t_{FRP} = equivalent FRP sheet thickness for FE models; $t_{Adhesive} =$ net composite material thickness for FE models.

Table 3 Material properties of steel, CFRP, GFRP and adhesive.

Properties	Steel tube	$CFRP_{Manu}$	$GFRP_{Manu}$	$CFRP_{Test}$	$GFRP_{Test}$	Adhesive (Kabir et al. 2016d)
Elastic Modulus (GPa)	211	230	72	75	23	3
Standard Deviation	23.9	--	--	2.1	1.7	--
COV (%)	11.3	--	--	2.8	7.2	--
Tensile Strength (MPa)	366	4900	3400	987	508	46
Standard Deviation	3	--	--	55.8	20.2	--
COV (%)	0.8	--	--	5.7	4	--
Yield Stress (MPa)	317	--	--	--	--	--
Standard Deviation	4.6	--	--	--	--	--
COV (%)	1.4	--	--	--	--	--
Thickness of FRP (mm)	--	0.2	--	0.5	0.5	--
Standard Deviation	--	--	--	0.03	0.01	--
COV (%)	--	--	--	6.3	2.5	--
Mode I Fracture Energy (N/m)	--	--	--	--	--	1000
Mode II Fracture Energy (N/m)	--	--	--	--	--	1250
Mode I Elastic Stiffness (N/mm^3)	--	--	--	--	--	2.8×10^{13}
Mode II Elastic Stiffness (N/mm^3)	--	--	--	--	--	1.4×10^{13}

Note: $CFRP_{Manu}$= Dry CFRP properties obtained from manufacturer; $GFRP_{Manu}$= Dry GFRP properties obtained from manufacturer; $CFRP_{Test}$= Epoxy cured CFRP laminate properties obtained from test; $GFRP_{Test}$= Epoxy cured GFRP laminate properties obtained from test.

Table 4 Comparison of peak lateral displacement of tests and FE results

Specimen ID	$\delta_{p(test)}$(mm)	$\delta_{p(FE)}$ (mm)	$\delta_{p(test)}/\delta_{p(FE)}$
S-B-V1	50.5	48	1.05
C-L-V1	46.5	46	1.01
C-LL-V1	39	39.5	0.99
C-LL-V2	49	48	1.02
C-HL-V1	39.5	37	1.07
C-LLL-V1	41	41.5	0.99
C-LHL-V1	38.5	38	1.01
C-HLH-V1	41	40	1.03
G-LL-V1	41.5	41	1.01
G-HL-V1	44	44	1.00
GC-LL-V1	45	43.5	1.03
C-LL975-V1	40.5	42	0.96
C-LL650-V1	43.5	41.5	1.05
Mean			1.02
COV			0.03

Table 5 Summary of axial loading application

Applied axial load (kN)	Axial capacity (kN)	Percentage of axial capacity (%)
940	3760	25
1500	3760	40
1880	3760	50

Table 6 Residual axial capacities and maximum lateral displacements of columns with various ecentricities

Column ID	Residual Axial Capacity (kN)	Lateral Displacement (mm)
B-E1	1579	281
B-E2	1136	429
B-E3	158	536
B-E4	1902	5
B-E5	1902	2.5
B-E6	1902	2.5
C-E1	1901	17.5
C-E2	1901	19.5
C-E3	1903	21.0
C-E4	1902	12.0
C-E5	1903	9
C-E6	1901	6