Axial load resistance of a novel UHPFRC grouted SHS tube sleeve connection: experimental, numerical and theoretical approaches

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Abstract

This study conducts experimental, numerical and theoretical analyses on the axial load resistance of a novel ultra-high performance fiber reinforced concrete (UHPFRC) grouted square hollow section (SHS) tube sleeve connection. The experimental study tests ten full-scale specimens with varying shear key spacings, grout thicknesses, grout lengths and volume proportions of steel fiber in the UHPFRC. Two types of failure modes are observed: (1) for the connection with high strength of the grouted part, the failure mode is fracture of the inner tube; (2) for the connection with lower strength of the grouted part, the failure mode is grout shear crushing with significant bond-slip between grout and steel tube. To further understand the load transfer mechanism of the connection, an advanced 3D nonlinear FE model is built to simulate the axial load-displacement behavior, state of stress and strain, as well as crack development of the grout. Based on the test and FE results, a new theoretical model is derived to predict the axial load resistance of the connection. The proposed model has considered the effect of section shape and material parameters, and is applicable to UHPFRC grouted SHS tube sleeve connection with different corner radii. The validations against the test results show that the new model can provide reasonably effective and accurate predictions to the axial load resistance of the novel grouted sleeve connection subjected to tension.

Keywords: sleeve connection, UHPFRC, axial load resistance, shear key, friction and adhesion

1. Introduction

Modular construction, by which modules are prefabricated off-site and assembled on-site, has become a popular option in construction industry due to its higher efficiency and productivity [1,2], better quality and safety [3,4], as well as lesser labor intensive and pollution [5]. Depending on the degree of off-site manufacturing, modular unit may vary from simple stick frame systems, such as pre-cast concrete or prefabricated bathroom pods, to fully prefabricated prefinished volumetric constructed (PPVC) module [6]. A PPVC module is completed with internal finishes, fixtures and fittings in an off-site fabrication facility, before it is delivered and installed on-site, thus offers the highest prefabrication rate [7]. One of the most critical issues affecting the integrity and safety of modular buildings is the connections between the PPVC modules [8].

PPVC modules are normally connected externally for minimization of interior decoration on-site [9]. According to joint locations, connections are classified as corner, perimeter and interior connections, respectively. Bolted connections are the most-commonly used connections, including beam-beam connections, column-column connections and beam-column connections. Extensive research gas been conducted in this field. Liu et al. [10] investigated the ultimate load resistance of a bolted-flange column-column connection under combined compression, bending and shearing. Load-transfer mechanism was studied and load resistance equations were proposed following the yield line theory and T-stub analogy. Chen et al. [11] designed a novel beam-beam bolted connection that provides easy access interior module connections, and experimentally investigated its static and hysteretic behaviors. The ultimate strength and energy dissipation ability of the interior connection were found to be sensitive to the bending stiffness of each unit joints and their relative stiffness. Torbaghan et al. [12] investigated the performance of a simple and efficient moment connection for pre-fabricated steel structures subjected to cyclic loading. Both experimental tests and FE simulations were performed on connections of beams, columns, plates and stiffeners of varying thicknesses. The connections exhibit excellent performance under cyclic loading. Although bolted connections are widely used, there are still some critical issues requiring attention. Firstly, the accumulation of geometric and positioning deviations may easily cause alignment issues especially for high-rise modular buildings [13-15]. Secondly, corrosion is a critical problem for bolted connections exposed to humid weather environment [16]. Thirdly, extensive usage of bolted connections may reduce productivity of modular construction and cause collision during modular assembling [17, 18].

To overcome these issues, a shear key-grouted column connection of square hollow section (SHS) was developed, which connects the upper and lower columns by filling the gap in the overlapped zone of the connection with grout [19]. The idea of the SHS column connection originates from the CHS column connection. The difference is that the CHS column connection consists of circular hollow section tubes, and is widely used in offshore structures due to the excellent streamline, such as offshore pile foundations and the transition parts of wind turbine towers, etc. In contrast, the SHS column connection can be applied in modular construction industry as it is easy for installation, arrangement and standardization. Extensive studies have been conducted to investigate failure modes and load resistances of CHS pile-to-sleeve connections [20-22]. Axial load resistance of a grouted connection is attributed to the bond strength due to friction and adhesion between grout and steel tube, and the mechanical interlock provided by shear keys [23-25]. Many full-scale [26-27] or large-scale [28-30] experiments have been conducted to evaluate load resistance of pile-to-sleeve connections. Krahl and Karsan [31] were the first to analyze load transfer mechanism of shear key-grouted pile-to-sleeve connections, and proposed an analytical equation for axial load resistance based on the compression strut model. Lee et al. [32] experimentally and numerically investigated the axial load resistance of high-strength grouted connections and studied the influence of loading eccentricity. The comparisons show that the load resistance under concentric loading is very close to that under eccentric loading. Chen et al. [33] conducted a series of parametric analyses based on an established FE model and concluded that increasing radial stiffness and shear key height-to-spacing ratio could effectively increase axial load resistance of pile-to-sleeve connections. Lotsberg [34] studied the structural mechanics of grouted connections in monopile wind turbine structures when they were subjected to not only axial force but also severe dynamic moment. A design methodology was proposed for ultimate limit state and fatigue limit state designs.

Existing design codes, including DNV 2014 [35], NORSORK 2012 [36], API 2007 [37], ISO 2007 [38], have recommended various equations for calculating axial load resistance of pile-to-sleeve connections. However, these equations cannot be directly applied to SHS column connections, as the confinement effect on the grout is quite different. Sui et al. [19] investigated the load transfer mechanisms of a grouted prefabricated SHS column connection under axial compression and tension. The investigation concludes that for a connection under axial compression, load is transferred from the upper outer tube to the lower outer tube. The load resistance is dominated by the geometric sizes and material properties of the outer tubes. For a connections under axial tension, load is transferred from the upper outer tube to the inner tube through the grout. The failure mechanism is more complex and the load resistance is difficult to predict. Dai et al. [39] conducted push-out tests and numerical simulations of SHS sleeve connections for modular construction, but did not propose any analytical design equations to predict grout failure. Moreover, the push-out tests may not accurately replicate real loading scenarios where a connection is subjected to “pull out” and bending.

To address the important issues raised above, the current study designs a novel UHPFRC grouted SHS tube sleeve connection, which is an improvement of the sleeve connection reported by Sui et al. [19]. As the space for grouting between the outer and the inner tube is limited, using UHPFRC with higher strength, higher ductility and better workability is more desirable than using normal concrete with coarse aggregates. In the research reported in this paper, ten full-scale specimens under axial tension are tested first to examine their failure modes and axial load resistances. The main reasons to conduct axial tension tests are that not only axial tension is a potential loading scenario of the column under accidental action, but also the tests will provide useful information to study flexural performance of the column. Advanced 3D nonlinear FE simulation is performed then to study the stress and crack development of the grouting material. Finally, a theoretical model based on the load transfer mechanism observed from the tests and the FE simulations is developed to predict axial tensile resistance of the UHPFRC grouted SHS tube sleeve connections.

2. Full-Scale Experiment

Figures 1a and 1b show the fabrication procedure of the UHPFRC grouted SHS tube sleeve connection. Each column connection consists of four main parts: the upper outer tube, the lower outer tube, the inner tube and the grout in the annulus. The inner tube is welded to a steel plate that is then welded to the top of the lower outer tube. The connection at this region is strengthened with additional stiffeners to ensure that punching shear fracture would not occur at the intersection between the inner tube and the steel plate. The upper part and the lower part are then assembled on site and the annulus between the inner tube and the outer tube is grouted with UHPFRC.

2.1 Test specimens

Figure 2 shows the configuration of the UHPFRC grouted SHS tube sleeve connection. Table 1 lists the geometric parameters of the ten specimens. The main geometric parameters include the outer diameter, thickness and radius of the round corner of the outer tube (B_o, t_o and r_o), the outer diameter, thickness and radius of the round corner of the inner tube (B_i, t_i and r_i), the length and thickness of the grout (L_g and t_g), the width and height of the shear key (w and h) and the shear key spacing (s). The shear keys, in the form of steel bar of 6 mm height and 12 mm width, are welded to both the inner surface of the upper outer tube and the outer surface of the inner tube, as shown in Figure 2. These specimens have different shear key spacings (s = 60 mm, 80 mm, 120 mm), grout thicknesses (t_g = 27 mm, 32 mm, 37 mm), grout lengths (L_g = 300 mm, 360mm, 420 mm), and steel fiber volume ratio in the UHPFRC (V_s = 0%, 1%, 2%).

2.2 Material properties

Three types of UHPFRC with 0%, 1% and 2% steel fiber in volume are prepared for the tests. Table 2 shows the mix proportions of the UHPFRC that consists of type II 52.5 R Portland cement, ultrafine silica fumes (SF), fine sands (grain size less than 4 mm), ground granulated blast furnace slags (GGBFS), polycarboxylate superplasticizer, shrinkage-reducing admixtures and steel fibers (12 mm in length and 0.6mm in diameter) [19]. With added steel fibers, the compressive strength of UHPFRC-1% and UHPFRC-2% tends to increase. Thus, in order to ensure that the two grout materials have similar compressive strength to the UHPFRC without steel fibers (around 100 MPa), a slightly higher W/B ratio of the UHPFRC-1% and UHPFRC-2% than that of the UHPC are used (Table 2). For each mixture, three Φ100x200 mm concrete cylinders according to ASTM C39/C39M [40] are prepared for compressive strength tests, and five concrete coupons according to JSCE-2008 [41] are prepared for tensile strength tests. The material properties of the concrete from the tests are summarized in Table 3.

It is noted that the compressive strengths of the three types of UHPFRC are around 100 MPa. In this case, the connections are grouted by the UHPFRC with similar compressive strength but different steel fiber contents. The SHS tubes and plates are made of mild steel Q235, and the shear keys is made of HRB 400 rebar. Material coupons from the corner region of the tubes and the flat regions of both the tubes and steel plates are tested, respectively. A universal test machine was used for the tensile tests of the steel coupons and rebars based on ASTM E8/E8M-2016 [42]. Table 4 summarizes the Young's modulus, 0.2% offset yield strength f_y, and ultimate strength f_u of the steel tubes, plates and rebars, respectively.

2.3 Test set-up, loading and measurement

Figure 3 shows the test set-up and measurement scheme. The test adopted a computer-controlled servo hydraulic actuator with a tensile capacity of 5000 kN. By using high-strength bolts, the top of the specimen is connected to the ball joint of the actuator, and the bottom of the specimen is connected to the bearing floor. The actuator applies an axial tensile force on the top of the specimen through displacement control with a loading rate of 0.2 mm/min. The loading rate increases to 1 mm/min when the specimen starts to yield or the loading force starts to drop down. The Linear Variable Displacement Transducers (LVDTs), T1 and T2, measure the vertical displacement at the top of the specimen, and T3 and T4 measure the vertical displacement at the point of connection. It should be noted that, to measure the actual deformation of the specimen and the connection, additional LVDTs have to be installed near the bottom of the specimen to observe the deformation of the bottom plate. Strain gauge pairs, OH1-6 and OV1-6, are evenly distributed along the length of the outer tubes to measure the strains both in the circumferential and the longitudinal directions. IH1-2 and IV1-2 measure the circumferential and longitudinal strains on the inner surface of the inner tube near the connection.

2.4 Failure modes

Figure 4 shows two typical failure modes observed from the ten UHPFRC grouted SHS tube sleeve connections under axial tension. They are (1) outer tube yielding with fractured inner tube, as shown in Fig. 4(a), and (2) grout shear failure, as shown in Fig. 4(b).

For the specimens with high shear resistance UHPFRC grout, e.g., S80T32L420F0, the outer and inner tubes govern the behavior of the specimen. Bond-slip between the upper outer and inner tubes is limited. The grout is almost intact since little concrete fragments drop out from the annulus of the tubes. The upper outer tube above the grouted region and the lower outer tube are all yielded. The inner tube near the overlapped region also yields significantly, which is finally fractured at the peak load. The specimen S80T32L420F1 is supposed to exhibit a similar failure mode to specimen S80T32L420F0, since the shear resistance of S80T32L420F1 is greater than that of 80T32L420F0 due to the effect of the added steel fibers. However, due to operational reasons during the assembly of specimen S80T32L420F1, one of the bolts connecting the end plate to the actuator was not properly installed, resulting in excessive deformation of the end plate and termination of the loading process. Thus, S80T32L420F1 failed due to excessive deformation of the end plate, rather than what was expected.

For the specimens with UHPFRC grout that has lower shear resistance than that of the outer or inner tube, e.g., S120T32L300F0 and S80T32L300F0, the grout predominately governs the behavior of the specimen. No obvious yielding is observed on the outer tubes. Instead, bond-slip between the upper outer tube and the inner tube becomes significant. The grout starts to crack at the peak load. The final status of the grout is shown in Fig. 4(b), from which serious shear cracks and crush are observed at both the flat and the corner regions of the steel tube. For other specimens, e.g., S60T32L300F0, S80T32L300F1, S80T32L300F2, S80T37L300F2, S80T27L300F2 and S80T32L360F1, the failure modes show a mixture of the characteristics observed from the aforementioned two typical failure modes, i.e., yielding is observed on the upper outer tube above the grouted region and the lower outer tube, and bond-slip exists between the upper outer tube and the inner tube. Grout crushing caused by shearing occurs after reaching the peak load.

Figure 5 shows section view of the ten specimens obtained by waterjet cutting after being tested and Figure 6 displays the detailed crack patterns in the grout of each specimen. It is noted that both) S80T32L420F1 and S80T32L420F0 have failed by fracture of the inner tube with limited diagonal cracks in the grout. The cracks initiate from the adjacent diagonal shear keys. If the shear resistance of the grout is higher than that of the outer or inner tube, the load can be effectively transferred from the upper tube to the lower one through the grout. The grout may stay relatively intact before the upper tube or the lower tube reaches its ultimate strength. The final failure thus would initiate from the inner tube rather than outer tube, e.g., fracture of the welded part, because the inner tube has smaller sectional size compared to the outer tube. However, if the ultimate strength of the outer or inner tube is higher than the shear resistance of the grout, initial cracking may occur in the grout, followed by slipping between the steel tube and the grout. The force from the upper tube may induce shear damage of the grout associated with bond-slip between the upper outer tube and the inner tube, while the tubes may not yield, depending on the effective load that is transferred to them. Ideally, from the viewpoint of design, shear resistance of the UHPFRC grout can be equal to the tensile resistance of the inner tube. The crack patterns of the grout are classified into two types. The first type is diagonal line crack linking two staggered shear keys on the inner surface of the outer tube and the outer surface of the inner tube, respectively. The two adjacent parallel diagonal line cracks form a compression strut, through which the load is transferred from the outer tube to the inner tube. Based on the geometric relationship with grout thickness t_g and shear key spacing s, the angle of the compression strut α is calculated as $\tan^{-1}(0.5s/t_g)$. The second type is large-area crushing of the grout along the outer surface of the inner tube. The development of the cracks will be discussed in Section 3.

2.5 Load-displacement curves

Figure 7 displays the load-displacement curves of all the ten SHS-UHPFRC grout sleeve connections, where P is the external load from the actuator, and δ is the vertical displacement measured at the top end-plate. All the curves exhibit a similar trend before reaching the peak load resistance. The load-displacement relationship is initially linear, and then nonlinear due to grout cracking. After reaching the peak load resistance, the curve drops down either due to fracture of the inner tube (e.g., S80T32L420F1 and S80T32L420F0) or due to shear crushing of the grout. The peak load resistance of the specimen is positively related to the shear resistance of the grout. The specimen with the longest grout length, S80T32L420F0, has the largest shear resistance of the grouted part and the peak load resistance reaches 3005.2 kN. The specimen S80T32L420F1 is expected to have the same level of load resistance to S80T32L420F0. However, due to the operational reasons discussed above, which causes prematurely excessive deformation of the end plate, thus lower than expected load resistance. The specimen with the shortest grout length and largest shear key spacing, S120T32L300F0, has the lowest shear resistance of the grouted part with a peak load resistance of only 1404.6 kN. Evidently, grout resistance design is of great significance in the design of the connection.

2.6 Load-strain curves

Figure 8 displays the load-strain curves at the measuring points. For all the specimens except S120T32L300F0 and S80T32L420F0, the circumferential and longitudinal strains measured at the lower outer tube (OH-3, OH-6, OV-3 and OV-6) are far beyond the yield strain of the steel, indicating that the lower outer tube has yielded significantly. It is found that the circumferential strain is much larger than the longitudinal one for OV3. It may be because: (1) OH3 and OV3 are located right below the welded stiffener where stress concentration occurs and (2) necking or buckling near the welded point has occurred, resulting in a higher strain in the circumferential in the longitudinal direction. The other regions on the outer tubes also yield but with smaller yield strain (e.g., location OH-1, OH-4). For S120T32L300F0, the largest strain measured at the outer tubes is close to the yield strain of the steel, which indicates that the design strength of the grout part is equal to that of the steel tube. This will cause simultaneous shear failure of the grout and yield of the steel tube. In this case, the strength of the grout connection can be properly designed. For S80T32L420F0, the strains measured at the upper outer tube above the grouted part (OH-1, OH-4, OV-1 and OV-4) and at the lower outer tube (OH-3, OH-6, OV-3 and OV-6) are far greater than the yield strain of the steel, indicating both the upper outer tube and the lower outer tube have yielded significantly. However, the measured strains at the outer tube within the grouted part are very small. In view of the failure mode, it can be concluded that the UHPFRC grouted part of S80T32L420F0 has sufficient strength to transfer fully the load from the upper tube to the lower tube.

2.7 Effect of shear key spacing

Figures 9(a) compares the load-displacement curve and the load-longitudinal strain distribution respectively, of the SHS-UHPFRC sleeve grout connections with shear key spacing of 60 mm, 80 mm and 120 mm. The shear key spacing significantly affects the load resistance of the connection. The peak load resistance decreases with the increase of shear key spacing since fewer number of shear keys result in fewer number of compression struts formed in the grout to carry the axial force. At the peak load, the longitudinal strains at the lower outer tube of the three specimens are higher than the yield strain.

The measured strains at the outer tube within the grouted part are lower than the yield strain, indicating that the outer tube within this region is effectively strengthened by the grout and with reduced deformation.

2.8 Effect of grout length

Figure 9(b) compares the load-displacement curve and load-longitudinal strain distribution of the grouted connections with varying grout length of 300 mm, 360 mm and 420 mm, respectively. The specimen S80T32L420F0 and S80T32L420F1 are designed with high strength grouted part (420 mm), which are expected to fail due to yielding of the steel tube, although S80T32L420F1 fails earlier due to the excessive deformation of the end plate as discussed in Section 2.4 and 2.5. It can be seen from the curves that the shear resistance of the grouted part is positively related to the grout length. The specimen S80T32L420F0 fails by fracture of the inner tube, while the specimen S80T32L300F1 and S80T32L360F1 fail by shear crushing of the grout. The load-displacement curves show that the peak load resistance increases with the increase of grout length. More shear keys exist in a longer grout, which provide larger mechanical interlock strength with more compression struts to contribute to the strength of the grout to sustain the axial force. Moreover, a longer grout has increased contact area between concrete and steel tube, which increases the interfacial bond resistance of the grouted connection.

2.9 Effect of grout thickness

Figure 9(c) compares the load-displacement curve and load-longitudinal strain distribution of the grouted connections with varying grout thickness of 27 mm, 32 mm and 37 mm, respectively. The load-displacement behavior and the peak load resistance of the three specimens are very close to each other, indicating that the grout thickness has marginal effect on the load resistance of the grouted connections.

This is because an increase of grout thickness only changes the compression strut angle and does not increase the number of shear keys and the contact area between the concrete and steel tubes. Since the allowable change of grout thickness is very limited, the resulted difference in shear resistance of the grout is small. For the sake of practicability, a minimum grout thickness of two fiber length, i.e., 24mm in the present study, is recommended. For each specimen, the part of the outer tube outside the grouted zone has all yielded. However, the grouted part is still within elastic stage. This indicates that the high strength grouted connection is effective in transferring the load from the upper to the lower tube.

2.10 Effect of volume fraction of steel fiber in UHPFRC

Figure 9(d) compares, respectively, the load-displacement curve and load-longitudinal strain distribution of the grouted connections with steel fiber volume fraction of 0%, 1% and 2%. The small addition of steel fibers increases both compressive and tensile strength of the grout to some extend. The design compressive strength of the three UHPFRC are 96.6 MPa, 105.8 MPa and 108.9MPa, respectively, as shown in Table 3. However, the shear strength of the three mixes are significantly different. The comparisons in Fig.9(d) show that the peak load resistances of S80T32L300F1 and S80T32L300F2 are very close, which are much higher than that of S80T32L300F0. This is attributed to enhanced shear strength of the grouted connections, S80T32L300F1 and S80T32L300F2, that are significantly higher than that of S80T32L300F0. The comparisons also show that the UHPFRC with a fiber volume fraction of 1% is the best shear design of the three, indicating that a higher fiber volume fraction (>1%) may not be beneficial. This will also reduce the cost of UHPFRC. Moreover, as shown in the loading curves in Figure 9d, a higher fiber volume fraction (2%) provides a higher peak displacement. This indicates that the added fibers also improve ductility of the connection. As shown in Table 3, with different fiber volume fractions, the three UHPFRC have similar tensile strength f_t , while their shear strength, f_v , are significantly different. From the above observations, it can be concluded that shear resistance of the grout connections is best characterized by their respective shear strength, as the tensile strength is much less sensitive to fiber fraction. Moreover, the longitudinal strains at the lower outer tube of these three specimens are higher than the yield strain, leading to a full utilization of material strength.

3. Numerical Modelling

This study also performs numerical simulation to further understand the load transfer mechanism of the grouted connection and the state of stresses of the grouted connection during loading. The numerical simulation is executed using the standard static solver in the advanced finite element program ABAQUS.

3.1 Material model of UHPFRC and steel

The material model adopts the concrete damage plasticity (CDP) model to represent the behavior of the UHPC and UHPFRC. The CDP model specifies the inelastic behavior of concrete as elasticity-based isotropic damage in combination with isotropic tensile and compressive plasticity. The definition of the CDP model requires compressive constitutive relationship (including compressive stress-strain curve and damage variables), tensile constitutive relationship (including tensile stress-strain curve and damage variables), yield surface and flow potential parameters. According to the material test results, the 28-day cylinder compressive strength and tensile strength of UHPC, UHPFRC (1%) and UHPFRC (2%) are used to calibrate the FE model. Sui et al. [19] and Zhang et al. [43] have discussed the compressive and tensile constitutive relationship for UHPFRC and UHPC, respectively, and proposed the nondimensional stresses-inelastic strain curves and nondimensional damage variables-inelastic strain curves, which are directly used in the current study. The yield surface and flow potential parameters, including the dilation angle, second stress invariant ratio (K), ratio of biaxial to uniaxial compressive strength (f_{b0}/f_{c0}), eccentricity and viscosity factor are specified as 45°, 0.667, 1.07, 0.1 and 0.0001, respectively. The steel section adopts the elastic-plastic model which transfers the engineering stress-strain relationship to the true stress-strain relationship. Since the round corner of the SHS tube hardens in the cold-forming process during fabrication, the material properties of this part are separately defined.

3.2 Element type, boundary condition and contact definition

Figure 10 shows the FE model of the grouted connection, and only one quarter of the model is built since the engineering stresses and strains of the structure are symmetric in the XZ and the YZ planes. Both the steel and the concrete are meshed using the eight-node solid element with reduced integration (C3D8R), which is adequate for the nonlinear analysis especially in contact simulation. Convergence analysis has been conducted to investigate the sensitivity of the mesh size. The global element size for the steel tubes is 8 mm and for the concrete is 4 mm. The elements around the shear keys and the corner of the tubes and concrete are further refined. To simulate the real boundary conditions at the top and bottom of the specimens, the through-hole bolts are modelled between the end plate and the rigid end plate. In this case, the rigid bottom plate is fixed, while the top end plate is coupled with a reference point, where a vertical displacement load acts. The contacts between concrete and steel, between bolts and plates, and between plates adopt the standard general contact. The two contact surfaces are defined by the balanced master-slave relationship to guarantee the accuracy of the contact analysis. The contact properties include both normal and tangential behavior. The former is specified as hard contact, and the latter is specified by a friction coefficient of 0.7 for concrete-steel contact and 0.5 for steel-steel contact.

3.3 Validation of FE model

Figure 11 compares the load-displacement curves of the ten grouted connections obtained from the tests and the FE simulations. The FE models present good predictions of the load-displacement curves before the peak load is reached for the first eight specimens. The predicted load resistance is very close to the test values. When approaching the peak load, significant amount of cracks are generated in the grout. The load resistance of the specimen is reduced due to shear crushing of the grout and the load-displacement curves show dramatical load drops. The FE simulation terminates due to numerical convergence problem caused by serious damage of concrete. For S80T32L420F0 and S80T32F420F1, the load-displacement curves by FE simulation increase monotonically and this continues after the failure point from the tests. This is because the grout experiences only minor damage during loading. The failure of these two specimens is governed by the steel inner tube whose damage is not considered in the constitutive model of the steel material. Thus, the fracture process of the inner tube is ignored in the simulation. However, the predicted load-displacement curves of these specimens match very well with those from the tests before the peak load resistance.

3.4 Development of concrete cracks

Compared to the experimental results, the validated FE model provides a convenient and useful tool to extract detailed information on the development of crack in the grout. Figure 12 plots stiffness degradation of the grout at two critical stages, i.e., when the load is half of the peak load and at the peak load. For the first eight specimens failed by grout shear crushing, the crack pattern in the grout is initially diagonal between two staggered shear keys on the inner surface of the outer tube and the outer surface of the inner tube. At the peak load, the diagonal line cracks develop rapidly, leading to severe crushing along the longitudinal direction and large bond-slip between grout and steel tubes. For the last two specimens with larger grout length, only minor diagonal line cracks are formed and the grout is not crushed. The damage of the grout at the end of test is also presented for comparisons. As can be seen in Figure 12, generally, the proposed FE model is able to satisfactorily reproduce the damage in the grout observed from the tests.

4. Theoretical model of axial load resistance

According to the test and numerical results, there are two types of failure modes when an UHPFRC grouted SHS tube sleeve connection is subjected to axial tension, They are (1) inner tube fracture, and (2) shear crushing of the grout associated with bond-slip between the grout and steel tube. Therefore, the axial load resistance associated with the first failure mode can be obtained by multiplying the ultimate strength of steel by the cross-section area of the inner tube. The second failure mode is, however, more complex due to shear crushing of grout. Hence, the axial load resistance associated with the second failure mode needs to be evaluated appropriately. To provide guidance for the design of UHPFRC grouted SHS tube sleeve connections, this section derives a theoretical model based on the load transfer mechanism of the sleeve connection to predict the axial load resistance.

4.1 Existing analytical models for pile-to-sleeve connection

Currently, there is not any design guideline for axial load resistance of UHPFRC grouted SHS tube sleeve connections. The only relevant design guideline is for axial load resistance of pile-to-sleeve connections normally used for offshore structures, such as offshore pile foundations and transition pieces of wind turbine towers, etc. The main difference is that the cross sections of piles and sleeves are of circular hollow section (CHS). For a pile-to-sleeve connection subjected to axial load, the ultimate shear stress τ_u is defined as the ratio of ultimate axial load resistance P_u to the outer surface area of the pile within the grouted region, as given in Eq.(3),

$$\tau_u = \frac{P_u}{\pi D_p L_g} \tag{3}$$

where D_p is the outer diameter of the inserted pile, and L_g is the length of the grouted region.

The axial load resistance of pile-to-sleeve connections is developed from, firstly, the bond strength due to friction and adhesion between grout and pile and, secondly, the mechanical interlock strength from shear keys. Existing design codes have given different equations to predict shear stress in a pile-to-sleeve connection. These equations include two terms related, respectively, to the two aspects mentioned above, each of which needs to be calculated independently. Based on a large number of experimental data, DNV 2014 [35] proposed the following equation for shear resistance,

$$\tau_u = \left[\frac{800}{D_p} + 140\left(\frac{h}{s}\right)^{0.8}\right] k^{0.6} f_{cu}^{0.3} \tag{4}$$

where the first term is attributed to friction and adhesion, and the second term is to mechanical interlocking. h is shear key height, s is shear key spacing, f_{cu} is concrete compressive strength, and k is radial stiffness parameter which is defined as:

$$k = \left[\frac{D_p}{t_p} + \frac{D_s}{t_s}\right]^{-1} + \frac{E_g}{E_s}\left[\frac{D_s - 2t_s}{t_g}\right]^{-1} \tag{5}$$

In which D_s is outer diameter of the sleeve, t_p, t_s, t_g are thickness of the pile, sleeve and grout, respectively. E_s and E_g are elastic modulus of the steel and concrete, respectively.

Based on the load transfer mechanism of pile sleeve connection, Krahl and Karsan [31] have established a set of force equilibrium equations for a cracked compression strut in grout, and derived shear stress due to mechanical interlock in relation to shear key height, shear key spacing and concrete compressive strength. The bond strength due to friction and adhesion is determined as a constant based on a regression analysis of experimental data. The ultimate shear resistance is shown as below.

$$\tau_u = 1.15 + 1.72 f_{cu}\left(\frac{h}{s}\right) \tag{6}$$

To conservatively estimate axial load resistance and extend this equation to pile-to-sleeve connections without shear keys, API 2007 [37] recommended a smaller value for the coefficient of the second term, as given in Eq. (7).

$$\tau_{\mathrm{u}} = 0.138 + 0.5 f_{\mathrm{cu}} \left(\frac{h}{s} \right) \tag{7}$$

As the cross-section shapes of the UHPFRC grouted SHS tube sleeve connection and the CHS pile-to-sleeve connection are different, the interfacial bond strength between the concrete and steel tubes as well as the confinement to the grout are very different. Thus, the axial load resistance equations for pile-to-sleeve connection cannot be directly applied to UHPFRC grouted SHS tube sleeve connections.

4.2 Friction and adhesion for UHPFRC grouted SHS tube sleeve connection

The axial load resistance due to friction and adhesion for UHPFRC grouted SHS tube sleeve connection is calculated as:

$$P_{\mathrm{b}} = \tau_{\mathrm{b}} \cdot 4 B_{\mathrm{i}} L_{\mathrm{g}} \tag{8}$$

where τ_{b} is bond strength; B_{i} is width of the inner tube, L_{g} is length of the grouted region.

Roeder et al. [44] have investigated the factors affecting bond strength of concrete-filled steel tubes and concluded that SHS tubes possessed lower bond strength than CHS tubes. In addition, bond strength is not sensitive to concrete strength, but negatively related to tube diameter and diameter-to-thickness ratio. Based on a regression analysis of experimental data, Lyu and Han [45] derived the bond strength equations shown in Eqs.(9) and (10) for CHS and SHS concrete-filled steel tubes, respectively.

$$\tau_{\mathrm{b\_CHS}} = 0.071 + 4900 \left(\frac{t}{D^2} \right) \tag{9}$$

$$\tau_{\mathrm{b\_SHS}} = 0.043 + 1100 \left(\frac{t}{B^2} \right) \tag{10}$$

Eq. (9) and Eq. (10) have the similar form but with different coefficients. To unify the above two equations and make the bond strength equation applicable to different shapes of cross section, Eq. (11) below is proposed for grouted SHS tube connections by introducing corner radius to width ratio $2r_o/B_o$, where r_o and B_o are corner radius and width of outer tube, respectively. If $2r_o/B_o$=0, the cross section is square and Eq. (11) is reduced to Eq. (10); if $2r_o/B_o$=1, the cross section is circular and Eq. (11) becomes Eq. (9); if $0<2r_o/B_o<1$, the cross section is square with rounded corners

$$\tau_b = \left(0.043 + 0.028\frac{2r_o}{B_o}\right) + \left(1100 + 3800\frac{2r_o}{B_o}\right)\left(\frac{t_o}{B_o^2}\right) \tag{11}$$

4.3 Shear key interlock for UHPFRC grouted SHS tube sleeve connection

The load resistance due to mechanical interlock of shear keys is calculated as below:

$$P_s = nf_{cu}^* \cdot 4\left(B_i + h\right)h = 4f_{cu}^*\left(B_i + h\right)\frac{hL_g}{s} \tag{12}$$

where $n=L_g/s$ is number of shear keys; f_{cu}^* is confined concrete strength; B_i, h, L_g and s are width of inner tube, shear key height, shear key spacing, and grout length, respectively.

The shear resistance contributed by shear keys is calculated by dividing the load resistance by the outer surface area of the inner tube within the grouted part. Let $\xi = f_{cu}^*/f_{cu}$, then,

$$\tau_s = \frac{P_s}{4B_iL_g} = \xi f_{cu}(1 + \frac{h}{B_i})\frac{h}{s} \tag{13}$$

For a cracked compression strut, the free body diagram is shown in Figure 13(a). The triangular part with side length of ah represents the critical region of the concrete contributing to the load resistance. Because this critical region cannot be seen during loading, the section has been cut apart after the test. Thus, the size of the triangular part can be directly extracted from the FE results or test results [31], as shown in Fig. 13(b). Table 5 lists the values of a for all the ten specimens. As these values are very close to each other, the average value of them, 3.8, is used for a in the following study.

Based on force and bending moment equilibrium for the free body shown in Figure 13(a), the following equations (14)-(16) can be established.

$$4B_{\mathrm{i}}F_5 + 4(B_{\mathrm{i}} + h)F_1 = 4(B_{\mathrm{o}} - 2t_{\mathrm{o}})F_6 + 4(B_{\mathrm{o}} - 2t_{\mathrm{o}} - h)F_2 \tag{14}$$

$$4B_{\mathrm{i}}F_3 = 4(B_{\mathrm{o}} - 2t_{\mathrm{o}})F_4 \tag{15}$$

$$4(B_{\mathrm{o}} - 2t_{\mathrm{o}} - h)F_2 \cdot (t_{\mathrm{g}} - 0.5h) + 4(B_{\mathrm{o}} - 2t_{\mathrm{o}})F_6 \cdot t_{\mathrm{g}} = 4(B_{\mathrm{o}} - 2t_{\mathrm{o}})F_4 \cdot (0.5s - ah) + 4(B_{\mathrm{i}} + h)F_1 \cdot 0.5h \tag{16}$$

In addition, the following relationships exist between the forces.

$$F_1 = hf_{\mathrm{cu}}^{*} \tag{17}$$

$$F_5 = \mu F_3 \tag{18}$$

$$F_6 = \mu F_4 \tag{19}$$

where μ is the friction coefficient between the steel tube and the concrete. Six unknown forces exist in the above six independent equations. In order to evaluate the confining effect, the ratio of F_3 to F_1 is solved as:

$$R = \frac{F_3}{F_1} = \frac{(B_{\mathrm{i}} + h)(t_{\mathrm{g}} - h)}{B_{\mathrm{i}}(1.5s - ah - \mu t_{\mathrm{g}})} \tag{20}$$

F_3 is also the product of the normal confining pressure f_{conf} and the side length of the critical triangular part ah.

$$F_3 = ahf_{\mathrm{conf}} \tag{21}$$

Combining Eqs. (17), (20) and (21) yields,

$$\frac{F_3}{F_1} = \frac{ahf_{\mathrm{conf}}}{hf_{\mathrm{cu}}^{*}} = \frac{af_{\mathrm{conf}}}{f_{\mathrm{cu}}^{*}} = R \tag{22}$$

The lateral confinement of the grout is affected by the corners of the column cross section [46]. Wu and Wang [47] have proposed a unified strength model for square and circular concrete columns confined by fiber reinforced polymer (FRP) materials. The strength model for columns with a corner radius degenerates into a model for circular columns when the corner radius is half of the column width, and degenerates into a model for sharp corner square columns when the corner radius is zero. Faustino et al. [48] has simplified the relationship between f_{cu}^* and f_{conf}, as shown in Eq. (23), by introducing $2r_o/B_o$ to consider the corner effect,

$$f_{cu}^* = f_{cu} + k\frac{2r_o}{B_o}f_{conf} \tag{23}$$

where k is a constant coefficient.

For the UHPFRC grouted SHS tube sleeve connections, this study intends to develop a similar unified strength model considering the corner effect. Fig. 13(c) shows the confinement state of grout in the UHPFRC grouted SHS tube sleeve connection. Only the grout within the shaded area can be effectively confined for SHS. According to Krahl and Karsan [31], the confined strength of grout in the CHS pile-to-sleeve connection is determined as $f_{cu}^* = f_{cu} + 4.1f_{conf}$. Thus, the unified strength model for the grout in the UHPFRC grouted SHS tube sleeve connection with rounded corners is proposed as,

$$f_{cu}^* = f_{cu} + 4.1\frac{2r_o}{B_o}f_{conf} \tag{24}$$

Substituting Eq. (24) into Eq. (22), the ratio of f_{cu}^* to f_{cu} can be obtained,

$$\xi = \frac{f_{cu}^*}{f_{cu}} = \frac{a}{a - 4.1\frac{2r_o}{B_o}R} = \frac{a}{a - \frac{8.2r_o}{B_o}R} \tag{25}$$

Substituting Eq. (25) into Eq. (13), the shear resistance due to mechanical interlock of shear keys can be determined by Eq.(26),

$$\tau_s = \left(1 + \frac{h}{B_i}\right)\frac{h}{s}\frac{a}{a - \frac{8.2r_o}{B_o}R}f_{cu} \tag{26}$$

where a has been determined from the FE model as shown in Table 5 and R is calculated by Eq. (20).

4.4 Prediction of axial load resistance

Similar to CHS pile-to-sleeve connections, axial load resistance of UHPFRC grouted SHS tube sleeve connections consists of the bond strength due to friction and adhesion between grout and steel tubes and the mechanical interlock due to shear keys. Based on the discussions in Sections 4.2 and 4.3, the total shear stress is calculated as,

$$\tau_u = \tau_b + \tau_s = \left[\left(0.043 + 0.028\frac{2r_o}{B_o}\right) + \left(1100 + 3800\frac{2r_o}{B_o}\right)\left(\frac{t_o}{B_o^2}\right)\right] + \left[\left(1 + \frac{h}{B_i}\right)\frac{h}{s}\frac{a}{a - \frac{8.2r_o}{B_o}R} f_{cu}\right] \tag{27}$$

The axial load resistance is then obtained by multiplying the shear stress with the cross-sectional area of the inner tube within the grouted region as follows.

$$P_u = 4B_i L_g \tau_u \tag{28}$$

4.5 Validation of theoretical model

The proposed model for the axial load resistance of an UHPFRC grouted SHS tube sleeve connection is validated against the test results of the ten specimens in this study and the four specimens tested independently by Dai et al. [39]. Table 6 lists the calculated shear stresses and load resistances of these specimens as well as the comparisons between the predicted axial load resistance and the test results. It can be seen that the axial load resistance due to friction and adhesion contributes less than 10% of the total axial load resistance. The main contribution of the load axial resistance is the mechanical interlock of the shear keys which provides over 90% of the total resistance. As can be seen, the calculated total axial load resistances of the UHPFRC grouted SHS tube sleeve connections are reasonably close to the test results. Figure 14 plots the ratios of the test results to the predicted results. The mean value of the ratios is 1.21 with a standard deviation of 0.17. Thus, the proposed theoretical model is reasonably successful in predicting axial load resistance of the UHPFRC grouted SHS tube sleeve connections failed by grout shear crushing associated with bond-slip between the grout and steel tube. The predictions are relatively conservative, which is beneficial for practical design.

5. Conclusion

An novel UHPFRC grouted SHS tube sleeve connection has been developed and its axial load resistance behavior has been investigated experimentally, numerically and theoretically. The experimental program tested ten full-scale sleeve connection specimens with different shear key spacings, grout thicknesses, grout lengths and volume proportions of steel fibers in the UHPFRC. An advanced FE model was built for the UHPFRC grouted SHS tube sleeve connection to examine its load-displacement curve behavior, stress and strain development as well as crack development of the grout. Based on the load transfer mechanism, a theoretical model was developed to predict axial load resistance of the UHPFRC grouted SHS tube sleeve connections subjected to tension. The researched reported in the paper supports the following conclusions:

(1) There are two types of failure modes, namely, (a) inner tube fracture, and (b) grout shear crushing associated with bond-slip between grout and steel tube, when an UHPFRC grouted SHS tube sleeve connection is subjected to axial tension. For a grouted sleeve connection with high strength grouted part, failure of the connection is governed by fracture of the inner tube. While for the grouted sleeve connection of lower shear resistance, failure is governed by the shear in the grout.

(2) For shear crushing failure of the grout, the FE simulation can successfully reproduces cracks development in the grout. During loading, diagonal line cracks initially appear between staggered shear keys on the inner surface of the outer tube and the outer surface of the inner tube. At the peak load, the diagonal line cracks develop rapidly causing severe concrete crushing along the tube surface, resulting in large bond-slip between the concrete and steel tubes.

(3) The axial load resistance of an UHPFRC grouted SHS tube sleeve connection decreases with shear key spacing but increases with grout length. Grout thickness has marginal effect on the load resistance of the grouted connection. The addition of steel fiber up to 1% in volume is effective in

increasing load resistance of the grouted sleeve connection. A higher fiber volume addition (>1% in volume) is not necessarily beneficial in design because it may not further enhance the shear resistance of the grouted sleeve connection.

(4) The newly derived theoretical formula in this paper is effective in predicting tensile resistance of the grouted connection. Corner radii factors are considered in the new model to consider the effect of sectional shape on the bond strength between grout and steel tube, and the confinement of the grout. The load axial resistance equation consists of two terms, namely, the friction and adhesion between grout and steel tubes, and the mechanical interlock contribution by shear keys.

(5) The new formula provides effective predictions to axial load resistance of UHPFRC grouted SHS tube sleeve connections subjected to tension. The validation against the test data in this paper and published literature indicates that the new formula can provide reasonably reliable and accurate prediction to axial load resistance for design purposes.

(6) The present study mainly focuses on the axial load resistance of the novel UHPFRC grouted SHS tube sleeve connection. In the real practice, the connection in a modular construction may be subjected to complex state of stresses, e.g., dead load along with seismic or wind load. Thus, future research to conduct studies on flexural, shear and hysteretic behavior of the grouted SHS tube sleeve connection subjected to combined bending, shearing and seismic loading is required to ensure better and deeper understanding of the new grouted sleeve connection.

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Nomenclature

ah	Side length of the critical triangle in the grout under the shear key
h	Shear key height
f_{conf}	Normal confining pressure
f_{cu}	Compressive strength of concrete
f_{cu}^{*}	Confined concrete strength
f_t	Tensile strength of concrete
r_o, r_i	Radius of the round corner of the outer tube and the inner tube, respectively
s	Shear key spacing
t_o, t_i	Thickness of the outer tube and the inner tube, respectively
w	Shear key width
B_o, B_i	Width of the outer tube and the inner tube, respectively
E_c	Elastic modulus of concrete
E_s	Elastic modulus of steel
L_g	Length of the grouted region
P_b	Load resistance due to friction and adhesion
P_s	Load resistance due to mechanical interlock of shear key
P_u	Ultimate axial load resistance
T_g	Thickness of the infilled grout
V_s	Volume proportion of steel fiber
τ_b	Shear stress due to friction and adhesion
τ_s	Shear stress due to mechanical interlock of shear key
τ_u	Ultimate shear stress
ξ	Ratio of f_{cu}^{*} to f_{cu}
μ	Friction coefficient between concrete and steel

Data Availability Statement

All data, models, and code generated or used during the study appear in the submitted article.

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Table 1: Geometric dimensions of test specimens

Specimen	$B_o \times t_o \times r_o$ (mm×mm×mm)	$B_i \times t_i \times r_i$ (mm×mm×mm)	h (mm)	w (mm)	s (mm)	T_g (mm)	L_g (mm)	V_s (%)	h/s	B_i/B_o
S80T32L300F0	250×8×30	170×12×25	6	12	80	32	300	0	0.075	0.68
S60T32L300F0	250×8×30	170×12×25	6	12	60	32	300	0	0.100	0.68
S120T32L300F0	250×8×30	170×12×25	6	12	120	32	300	0	0.050	0.68
S80T32L300F1	250×8×30	170×12×25	6	12	80	32	300	1	0.075	0.68
S80T32L300F2	250×8×30	170×12×25	6	12	80	32	300	2	0.075	0.68
S80T37L300F2	250×8×30	160×12×25	6	12	80	37	300	2	0.075	0.64
S80T27L300F2	250×8×30	180×12×25	6	12	80	27	300	2	0.075	0.72
S80T32L360F1	250×8×30	170×12×25	6	12	80	32	360	1	0.075	0.68
S80T32L420F1	250×8×30	170×12×25	6	12	80	32	420	1	0.075	0.68
S80T32L420F0	250×8×30	170×12×25	6	12	80	32	420	0	0.075	0.68

Table 2: Mix proportion of UHPFRC (kg/m^3)

Mix	W/B	W	OPC	SF	GGBFS	S	F	HWRA	SRA
UHPC	0.19	209.5	823.3	135.5	170.1	1060.0	0	7.29	6.29
UHPFRC-1%	0.21	213.9	750.0	130.5	165.1	1120.0	78.0	7.15	6.42
UHPFRC -2%	0.24	229.1	705.0	120.5	155.1	1150.0	156.0	5.81	6.87

Notes: W/B=water to binder ratio; W=water; OPC=ordinary Portland cement; SF=silica fume; GGBFS= ground granulated blast furnace slag; S=sand; F=steel fiber; HWRA=high Water reducing agent; SRA=shrinkage reducing agent.

Table 3: Material properties of the concrete

Concrete	f_{cu} (MPa)	E_c (GPa)	f_t (MPa)	f_v(MPa)	Poisson’s ratio
UHPC	96.6	41.4	5.4	7.1	0.185
UHPFRC-1%	105.8	42.6	6.0	14.2	0.182
UHPFRC-2%	108.9	44.2	6.1	16.2	0.192

Table 4: Material properties of the steel components

Component	Material	E_s (GPa)	f_y (MPa)	f_u (MPa)
Inner tube flat	Mild steel	205.3	260.5	405.6
Inner tube corner	Mild steel	202.2	482.5	522.1
Outer tube flat	Mild steel	202.1	323.5	457.7
Outer tube corner	Mild steel	208.4	461.7	541.6
Steel plate	Mild steel	206.2	377.6	546.8
Shear key	HRB 400Φ6	193.6	357.0	485.0

Table 5: Determination of the coefficient *a*

Specimen	*a*
S80T32L300F0	3.8
S60T32L300F0	3.7
S120T32L300F0	3.9
S80T32L300F1	3.7
S80T32L300F2	3.8
S80T37L300F2	3.7
S80T27L300F2	3.8
S80T32L360F1	3.8
S80T32L420F1	3.8
S80T32L420F0	3.8
Average	3.8

Table 6: Validation of the model

Literature	Specimen	R	ξ	f_{cu} (MPa)	f_{cu}^{*} (MPa)	τ_b (MPa)	τ_s (MPa)	P_b (kN)	P_s (kN)	P_u (kN)	P_{test} (kN)	$\frac{P_b}{P_u}$	$\frac{P_s}{P_u}$	$\frac{P_{test}}{P_u}$
This paper	S80T32L300F0	0.36	1.10	96.6	106.5	0.31	8.27	62.7	1687.4	1750.1	1885.3	4%	96%	1.08
	S60T32L300F0	0.60	1.18	96.6	114.4	0.31	11.84	62.7	2416.1	2478.8	2133.4	3%	97%	0.86
	S120T32L300F0	0.19	1.05	96.6	101.9	0.31	5.27	62.7	1075.7	1138.4	1404.6	6%	94%	1.23
	S80T32L300F1	0.35	1.10	105.8	116.7	0.31	9.06	62.7	1848.1	1910.8	2450.5	3%	97%	1.28
	S80T32L300F2	0.35	1.10	108.9	120.1	0.31	9.32	62.7	1902.2	1964.9	2458.5	3%	97%	1.25
	S80T37L300F2	0.46	1.14	108.9	124.0	0.31	9.65	59.0	1852.6	1911.6	2515.7	3%	97%	1.32
	S80T27L300F2	0.27	1.08	108.9	117.3	0.31	9.09	66.4	1963.9	2030.3	2476.4	3%	97%	1.22
	S80T32L360F1	0.35	1.10	105.8	116.7	0.31	9.06	75.2	2217.7	2292.9	2904.3	3%	97%	1.27
	S80T32L420F1	0.35	1.10	105.8	116.7	0.31	9.06	87.8	2587.3	2675.1	2670.6	3%	97%	1.00
	S80T32L420F0	0.35	1.10	96.6	106.5	0.31	8.27	87.8	2362.3	2450.1	3005.2	4%	96%	1.23
Dai et al. [39]	S6G28	0.93	1.34	90.4	121.1	0.67	10.91	85.8	1406.5	1492.2	1645.0	6%	94%	1.10
	S4G28	0.43	1.14	90.4	103.3	0.67	6.33	86.0	818.2	904.2	1287.0	10%	90%	1.42
	S6G18	0.41	1.12	90.4	101.6	0.67	9.16	101.1	1391.9	1493.0	1728.0	7%	93%	1.16
	S4G18	0.21	1.06	90.4	96.0	0.67	5.77	101.4	880.0	981.4	1520.0	10%	90%	1.55
Mean														1.21
Std.dev														0.17