

Article

Compressive Behaviour of Long Steel Tube Columns Filled with Recycled Large Aggregate Self-Compacting Concrete

Jianchao Wang ^{1,*}, Huayu Li ² and Wei Hou ³

¹ School of Science, Shenyang Jianzhu University, No. 25 Hunnan Middle Road, Hunnan District, Shenyang 110168, China

² School of Civil Engineering, Shenyang Jianzhu University, No. 25 Hunnan Middle Road, Hunnan District, Shenyang 110168, China; lihuayu@stu.sjzu.edu.cn

³ School of Marxism, Shenyang Jianzhu University, No. 25 Hunnan Middle Road, Hunnan District, Shenyang 110168, China; houwei@sjzu.edu.cn

* Correspondence: wangjianchao@sjzu.edu.cn

Abstract: One of the important directions for green development in the world today is to expand the application methods of recycled concrete, improve the utilization rate of waste aggregates, and slow down the consumption of natural resources. The column structure with a large length and slenderness ratio is the most widely used compression unit in practical engineering, which conforms to the principle of sustainable development. In this paper, we study the mechanical properties and failure modes of long columns fabricated from steel tubes filled with recycled large aggregate self-compacting concrete (RLASCC-ST-LC) under compression load. Moreover, we examine the influence of steel tube thickness, recycled large-aggregate particle size, the strength of self-compacting concrete, and the length-to-diameter ratio on the performance of the members through finite element modelling. The results indicated that RLASCC-ST-LCs exhibited different degrees of buckling damage, and the damage processes were basically the same as that of steel tube concrete. When the thickness of steel pipe increased from 4 mm to 5 mm, the ultimate bearing capacity of the component increased by 12.1%; when the strength of self-compacting concrete increased from 30 MPa to 40 MPa and 50 MPa, the ultimate loads of the component increased by 6.96% and 12.4%, respectively. However, the increase in the aspect ratio weakened the bearing capacity of the component, and the ultimate bearing capacities reduced by 4.78% and 10.51% when the aspect ratios were 8, 10, and 12. Finally, based on the existing design codes, the theoretical calculation formulas are proposed for the ultimate bearing capacities of RLASCC-ST-LCs. These findings have significant implications for the widespread application of RLASCC-ST-LCs.

Keywords: recycled large aggregate self-compacting concrete; long columns; failure mode; steel pipe thickness; length-to-diameter ratio; theoretical calculation



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1. Introduction

With the rapid development of the global economy, the replacement of old and new buildings has become a norm. In this environment, how to reasonably deal with the relationship between a large amount of waste aggregates and the materials required for new construction projects has become an important issue in current development. It is well known that concrete made from dismantled waste aggregates has a high crushing index and strong water absorption, which affects the overall strength of recycled concrete and hinders the development of such material [1–3]. In order to play an effective role in restraining the recycled concrete, it is combined with other materials to improve the mechanical properties. The excellent performances of steel and recycled concrete (RC) have significant complementary effects [4–7]. In recent years, a series of studies have been conducted on concrete-filled steel tube (CFST) structures with the goal of improving the overall strength and performance of the components. Yang et al. [8–10] filled recycled aggregates into

short steel tube columns of the cross-section types with different substitution rates (0, 30%, and 50%) to investigate their compressive properties. The incorporation of steel tubes showed a better restraining effect on the internal concrete. Compared with ordinary steel tube concrete, recycled-aggregate-concrete-filled steel tubes (RACFSTs) exhibited similar damage patterns, but had weaker load-bearing capacities. Several researchers [11–14] have investigated the damage process, load–deflection curves, and stress distributions by varying the load duration and using different load combinations, reflecting the excellent seismic and deformation performances of RACFSTs. In order to widely employ RACFST structures in practical engineering, it is necessary to take the time cost and environmental impact caused by aggregate particle size into account. A large number of performance studies have been carried out to investigate the compressive properties [15], tensile properties [16], size effect [17], durability [18], and water permeability [19] of coarse-grained aggregates [20,21]. Despite the fact that recycled coarse aggregates are weaker than natural aggregates, their strengths have been enhanced due to their large contact area and roughness, which allows for better contact with the cement matrix. Meanwhile, it has been shown that the utilization of dismantled waste coarse aggregates to fill steel pipes is reasonable and effective [22–24]. Zhao et al. [25–27] chose waste coarse aggregates with a particle size within 30 mm and waste fine aggregates within 5 mm to fill into the steel tube, and the study showed that there was a certain superposition effect between coarse and fine aggregates. Zhao et al. [28–30] employed waste aggregates with a particle size range of 10–60 mm to prepare RACFSTs, and experimentally studied the creep behaviours as well as structural responses of the waste coarse aggregate-filled steel tubes. They attempted to use the substitution rate of waste coarse aggregates, load eccentricity, and thickness of steel tubes as the test variables. From the test results, although the ultimate strength of the components fluctuated, there were no significant changes in the overall structural performances of the components. This approach not only greatly reduces the deficiencies caused by the waste aggregate in the steel tubes, but also compresses the time cost of the actual construction and promotes the protection of the environment [31,32].

In most cases, CFST structures usually exist as compression units, and the problem of slenderness ratio should not be neglected in the actual production process. Because of the large slenderness ratio of the compression member, it will usually be destabilised, so that the material itself cannot play its strength role. Hence, the designer will usually take a variety of structural measures to reduce the slenderness ratio of the member, so as to improve the ultimate load carrying capacity of the member.

In this paper, RLASCC-ST-LC members with different load eccentricities (0, 25 mm, and 50 mm) are designed to study their damage forms and working mechanisms. The influences of steel tube thickness, recycled large aggregate strength, and recycled large aggregate particle size on the ultimate bearing capacities of RLASCC-ST-LCs are analysed. This study is beneficial to help the design and construction of RACFST columns in the actual production process.

2. Experimental Research

2.1. Materials

2.1.1. Recycled Large Aggregate

An abandoned wall at the age of about 1 year is selected as a source of recycled large aggregates (RLAs). Figure 1 shows the design strength of C30, after primary crushing, to get a regenerated block with a diameter of 70 ± 5 mm, before the use of a rebound meter strength of 27.6 MPa. Due to the small cracks appearing inside or on the surface of the recycled large aggregates during the crushing process, the regenerated large aggregates of the absorbent water became strong, which had an impact on the mechanical properties of the members. Therefore, the recycled aggregates were soaked in water for more than 48 h, and the recycled aggregates could absorb water and reach the saturated state.

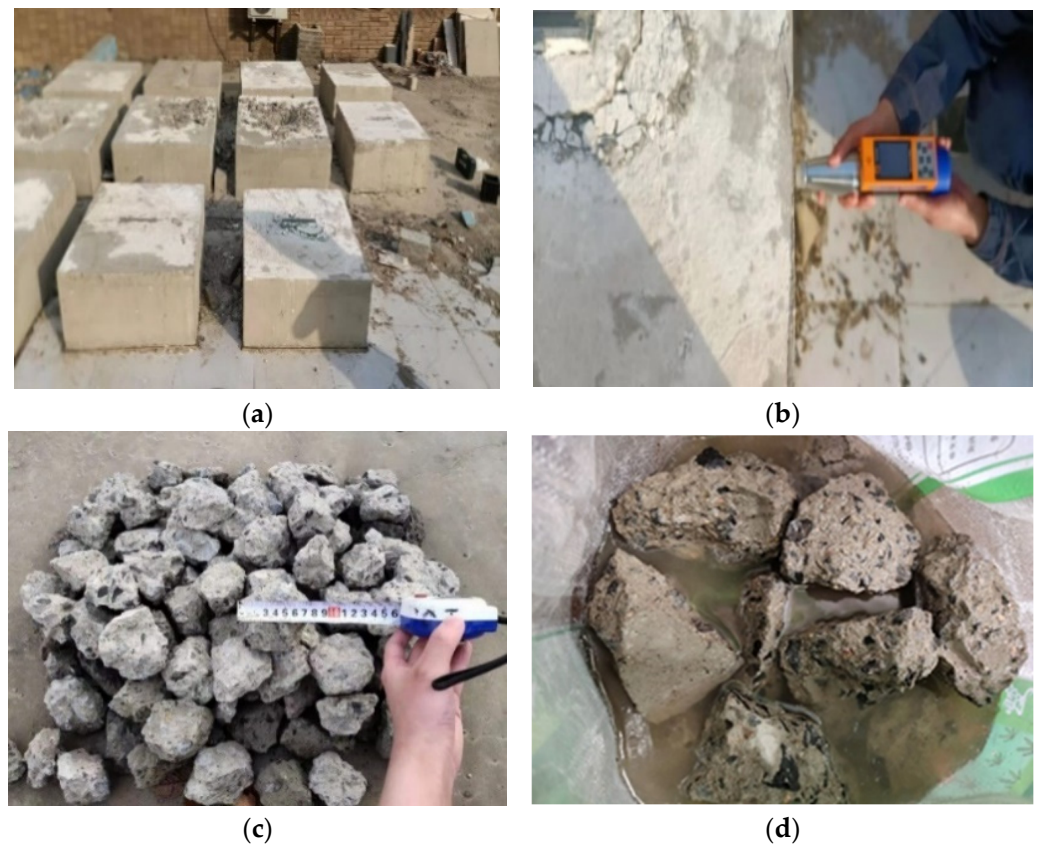


Figure 1. Selection of recycled large aggregates. (a) Abandoned wall; (b) strength measurement using a rebound instrument; (c) recycled aggregates; and (d) recycled aggregate during soaking.

2.1.2. Self-Compacting Concrete

The core concrete selected for the test was self-compacting concrete of a C30 strength class, consisting of silicate cement, primary fly ash, secondary zone medium sand, natural gravel, and high-efficiency water reducing agent. The mix ratio was calculated according to the specification of self-compacting concrete [33], and the mix ratio of self-compacting concrete is shown in Table 1.

Table 1. Mix proportion of self-compacting concrete.

Mix Proportion	Gravel (kg/m ³)	Sand (kg/m ³)	Cement (kg/m ³)	Fly Ash (kg/m ³)	Water (kg/m ³)	Water Reducer (%)
C30	864	762	305	184	224	19.56

2.1.3. Steel

Three tensile samples were selected from the 4 mm thick steel, as shown in Figure 2, in order to reduce the error of the test data measurement. In order to reduce the errors of the test data measurements, on these selected measurement positions, the length and width of the drawing test samples and the thickness of the average of the results were measured many times with vernier callipers, in accordance with the specification requirements [34]. As shown in Figure 3, the steel properties were measured by using the universal testing machine through the tension tests, and the test data are shown in Table 2.

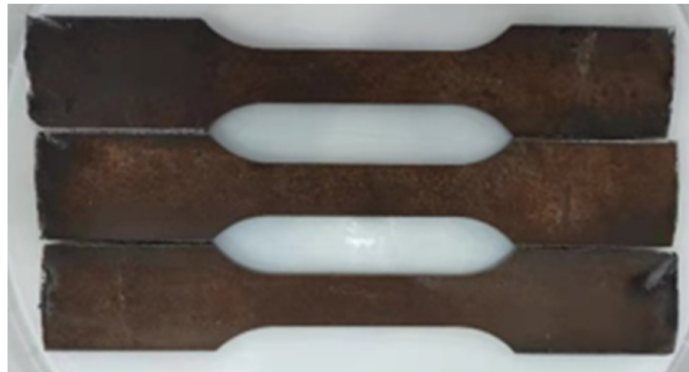


Figure 2. Tensile specimens.

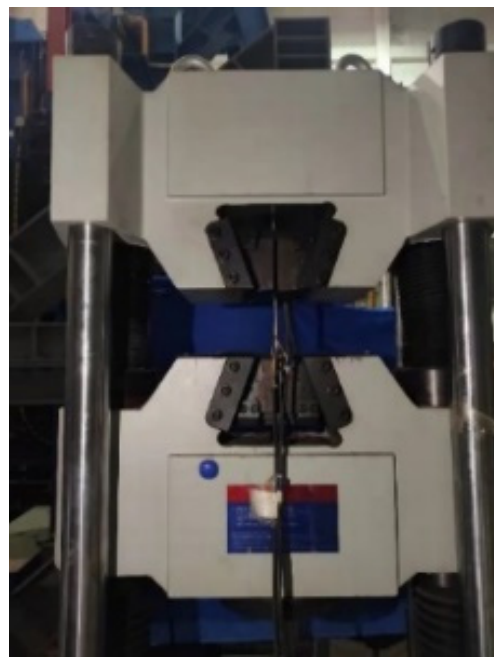


Figure 3. Tensile test.

Table 2. Measured mechanical properties of steel.

f_y (MPa)	f_u (MPa)	E_s (MPa)	μ_s
346	459	20,597	0.26
324	437	19,473	0.25
308	426	18,945	0.26

2.2. Determination of Concrete Cube Compressive Strength

In order to produce two groups of 150 mm cubic specimens, a group of three self-compacting concrete cubic specimens, as shown in Figure 4, made according to the specification [35], were subjected to compressive strength tests to determine the concrete strengths with an average value of 35.85 MPa. The average value of 31.05 MPa was determined on the other group of three recycled large-aggregate self-compacting concrete cubes.

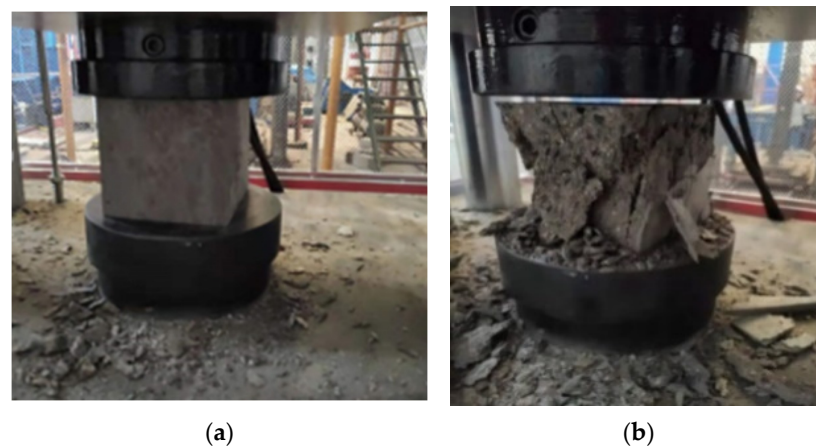


Figure 4. Compressive strength tests on concrete cubes. (a) Cube 1 for compressive strength. (b) Cube 2 for compression strength.

2.3. Specimen Design and Preparation

In this paper, a total of three specimens were made, and the load was applied at the symmetrical position of the column end with eccentricities of 0, 25 mm, and 50 mm, respectively; the specific parameters of the construction are shown in Table 3. As shown in Figure 5, the prepared self-compacting concrete was poured in from the top of the steel pipe with a thickness of about 20 ± 3 mm, and then the recycled aggregates with a particle size of 70 ± 5 mm were thrown in. Layered pouring was adopted, and the self-compacting concrete was placed into the steel pipe with recycled aggregates alternately until the steel pipe was filled, because the self-compacting concrete itself has the characteristic of high fluidity, so the whole process does not need vibration. After a period of time, when the self-compacting concrete had naturally settled, the remaining self-compacting concrete was used to fill and level the inside of the steel pipe, which was then placed indoors for curing. After 28 d of curing, the end plate of $280 \text{ mm} \times 280 \text{ mm} \times 20 \text{ mm}$ was welded, as shown in Figure 6, and the stiffening ribs were distributed uniformly in the circumferential direction.

Table 3. Grouping of specimens.

Numbering	H (mm)	D (mm)	e (mm)	$f_{cu,RLA}$ (MPa)	R_{RLA} (mm)	$f_{cu,SCC}$ (MPa)	t_s (mm)
Z1	1752	219	0	27.6	70 ± 5	35.9	3.6
P1	1752	219	25	27.6	70 ± 5	35.9	3.6
P2	1752	219	50	27.6	70 ± 5	35.9	3.6



Figure 5. Completing the specimen preparation.



Figure 6. Completed welding stiffener.

2.4. Test Setup and Loading

The tests on RLASCC-ST-LCs were carried out using a 5000 kN testing machine. Three displacement transducers with a range of 150 mm were placed at the quadrature of the height of the member, so as to measure the lateral deflection of the member. The displacement transducers were placed on the left and right sides of the bearing plate at the bottom end of the loading platen, to monitor the longitudinal displacements of the member. The mid-span position was polished with sand paper, and four groups of strain gauges were arranged uniformly along the ring direction, to collect longitudinal and transverse strains in the member. The loading device is shown in Figure 7.

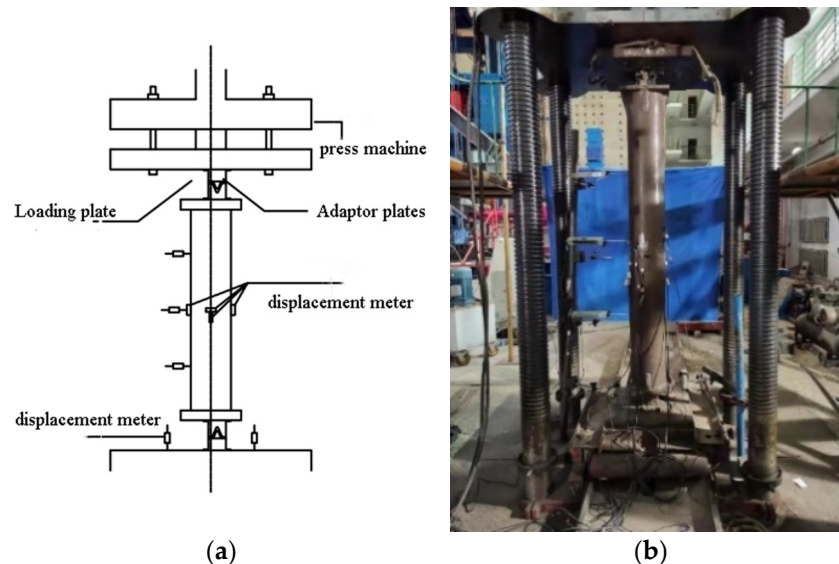


Figure 7. Test loading arrangement. (a) Loading arrangement; (b) test setup.

This test was conducted by displacement loading, with the load not exceeding 20 per cent of the expected peak load during the preloading test. After completion of the preloading test, the formal loading test was conducted. Each load level was 1/10 of the predicted ultimate load, and the load at each level was held for 2 min. The load data and the deformation of the specimen were observed, and the loading stopped when there was a

faster decrease in the load of the member or when the lateral deflection deformation of the member was obvious.

3. Experiment Results and Analysis

3.1. Failure Modes

Figure 8 shows the damage and failure patterns of a typical specimen. In the preloading period, a slight buckling occurred on the compression side, and with the increase in load, the buckling phenomenon gradually expanded to the tension side. It was clearly seen that the member showed an obvious bulging phenomenon in the upper region of the span, and the member was destabilised, which was basically the same as that of ordinary steel pipe concrete. Stripping the external steel tube, the internal concrete on the compression side had obvious compaction, while the internal concrete on the tension side possessed nearly uniformly distributed cracks, which shows that the stress condition of the member was good. By observing the section location of the core concrete, it was difficult to distinguish the distributions of new and old concrete, which proved that the bond between recycled large aggregate and self-compacting concrete was good.

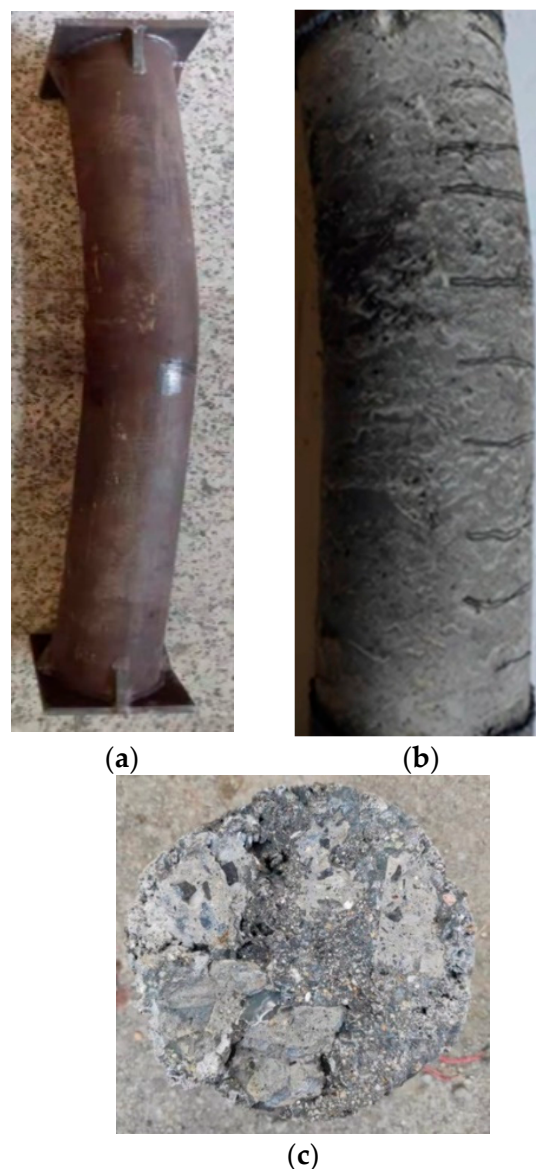


Figure 8. Failure modes of a typical RLASCC-ST-LC. (a) Typical component failure mode; (b) core concrete failure mode; and (c) state of core concrete failure section.

3.2. Load–Longitudinal Displacement Curves

The load–longitudinal displacement relationships of the test specimens are shown in Figure 9. It can be seen that at the beginning of loading, the load–displacement curve shows a linear growth pattern; the specimen was in an elastic state, and at this time, the steel pipe had a better restraining effect on the internal core concrete. When the ultimate load was reached, the displacement of the member rapidly increased, the deformation of the steel took place, the restraining ability of the internal concrete was weakened, and the overall load carrying capacity of the member decreased. When the distance between the loading point and the central point varied from 0 to 25 mm and 50 mm, the ultimate load capacity of the member decreased by 39.1% and 53.4%, respectively, indicating that with the increase in eccentricity, the load-bearing effect of the member significantly declined, and at the same time, the linear stiffness of the specimen also showed a decreasing trend. However, in the late loading stage, compared with the axial compression member, the descending trends of the curves of the eccentric compression members were slowed down. This is due to the fact that the increase in eccentricity distance makes the stress gradient of the cross-section larger. When the edge reaches yielding, the part close to the edge can continue to withstand greater stress, which is conducive to the stress redistribution in the cross-section, and the member shows better ductility.

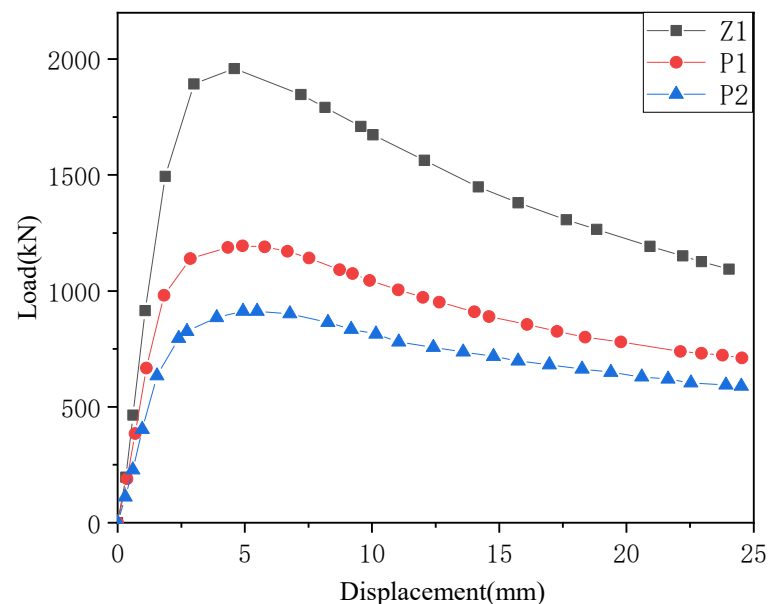


Figure 9. Load–longitudinal displacement curves of the test specimens.

3.3. Load–Strain Curves

The load–longitudinal strain relationships of the members were plotted by using the strain values from the strain gauges pasted on the steel tubes, as shown in Figure 10. At the early stage of loading, the strains on both sides were compressive strains. With the application of load, the bending moment in the column cross-section gradually became larger due to the bending and compressive stiffness centreline shift of the member, and tensile strains began to appear in the tensile side of the steel tube in the column, and gradually increase. When the steel strains on the compression side of the steel tube reached the yield strain, the compressive stiffness centre offset, the bending of the member accelerated, and the tensile strains on the tension side of the steel tube also accelerated.

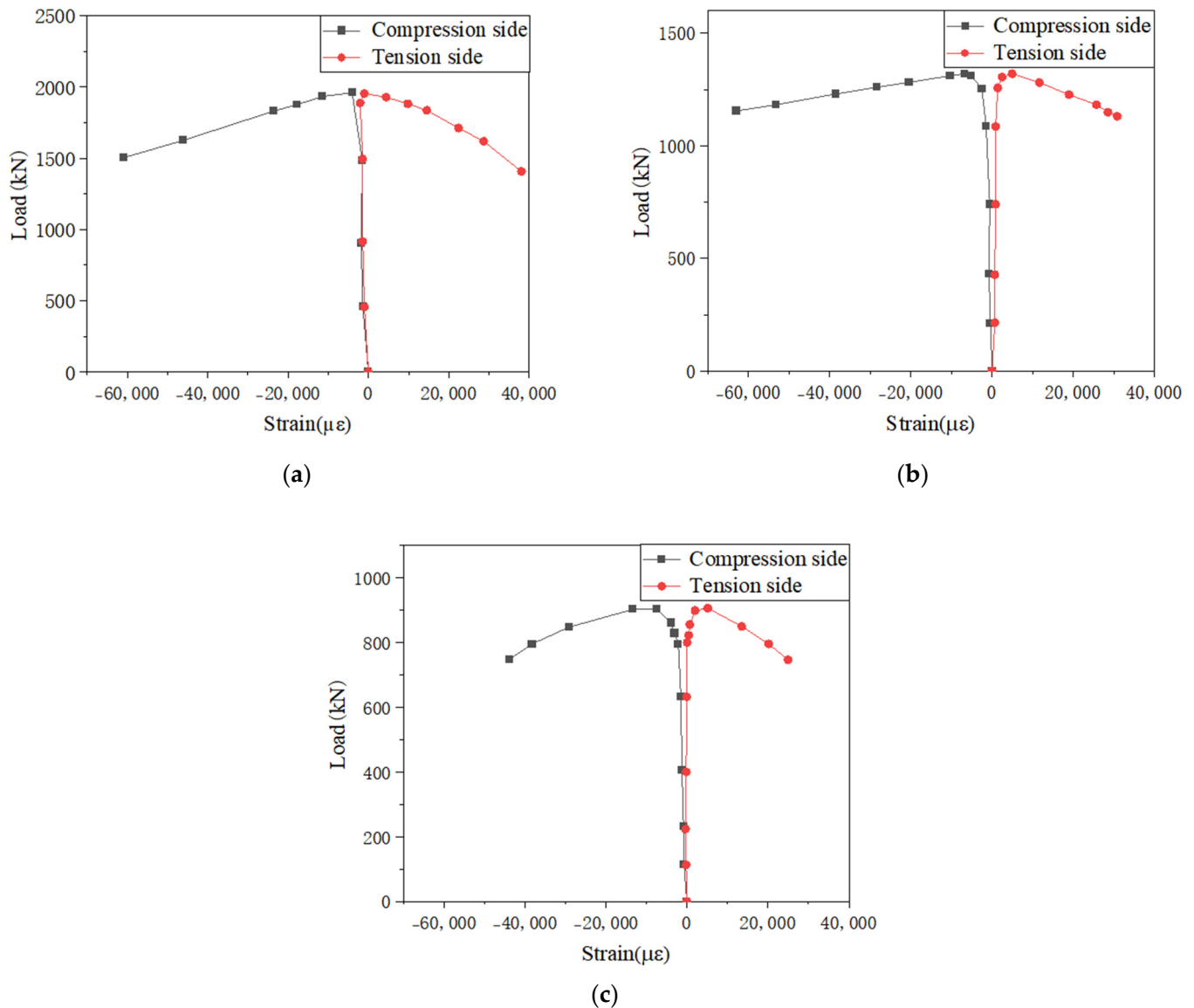


Figure 10. Load–longitudinal strain curves of the test components. (a) Test component Z1; (b) test component P1; and (c) test component P2.

4. Finite Element Analysis

The finite element software ABAQUS 6.13 was used for modelling verification and the parametric analysis of RLASCC-ST-LCs.

4.1. Model Buildings

The compressive stress–strain constitutive relationship of recycled large aggregate self-compacting concrete adopts the following constitutive model proposed by Han [36]:

$$y = \begin{cases} 2x - x^2 & x \leq 1 \\ \frac{x}{\beta(x-1)^2 + xy} & x > 1 \end{cases} \quad (1)$$

$$x = \frac{\epsilon}{\epsilon_0}; y = \frac{\sigma_c}{\sigma_0} \quad (2)$$

$$\sigma_0 = f_c \quad (3)$$

$$\epsilon_0 = \epsilon_c + 800 \cdot \zeta^{0.2} \cdot 10^{-6} \quad (4)$$

$$\varepsilon_c = (1300 + 12.5 f_c) \cdot 10^{-6} \quad (5)$$

$$\beta = (2.36 \times 10^{-5})^{[0.25 + (\zeta - 0.5)^7]} \cdot f_c^{0.5} \cdot 0.5 \geq 0.12 \quad (6)$$

where $f_{cu,com}$ is the RLA-SCC cube compressive strength, and can be obtained from the following formula:

$$f_{cu,com} = \left(\frac{f_{cu,old}}{f_{cu,new}} \right)^{0.86\eta} f_{cu,new}(1 - \eta) + \left(\frac{f_{cu,new}}{f_{cu,old}} \right)^{0.86\eta} f_{cu,old}\eta \quad (7)$$

where $f_{cu,new}$ is the SCC cube compressive strength (MPa),

$f_{cu,old}$ is the compressive strength of RAC cube made at the same period (MPa),

η is the recycled aggregate replacement ratio (%), and can be obtained from

$$\eta = (0.4889 - 0.0025d_{RLA}) \times 100\% \quad (8)$$

d is the recycled aggregate particle size (mm),

ζ is the confining factor.

Steel is regarded as an elastic-plastic material with various homogeneities, and the native model relationship is chosen to model mild steel, which is commonly used in actual construction projects, by adopting the model relationship from Ref. [37].

Due to the relative slip phenomenon between the steel tube and the core recycled concrete, the contact action between the external steel tube and the core concrete is set as “surface-to-surface contact”. In determining the contact behaviour between the outer steel tube and the core recycled concrete, the inner surface of the steel tube is chosen as the primary surface and the concrete as the secondary surface because the stiffness of the steel tube far exceeds the stiffness of the core concrete. The normal behaviour is defined as “hard contact” and the tangential behaviour as “penalty function” with a friction coefficient of 0.3. The contact between the steel tube and the upper and lower end plates is defined as “bound” contact, as shown in Figure 11. Two reference points, RP1 and RP2, were established, respectively, with the upper and lower end plates for simulation of the knife strand split region for coupling, the model of the upper end of the lower end of the hinged boundary conditions, and the loading mode selection of displacement loading, as shown in Figure 12.

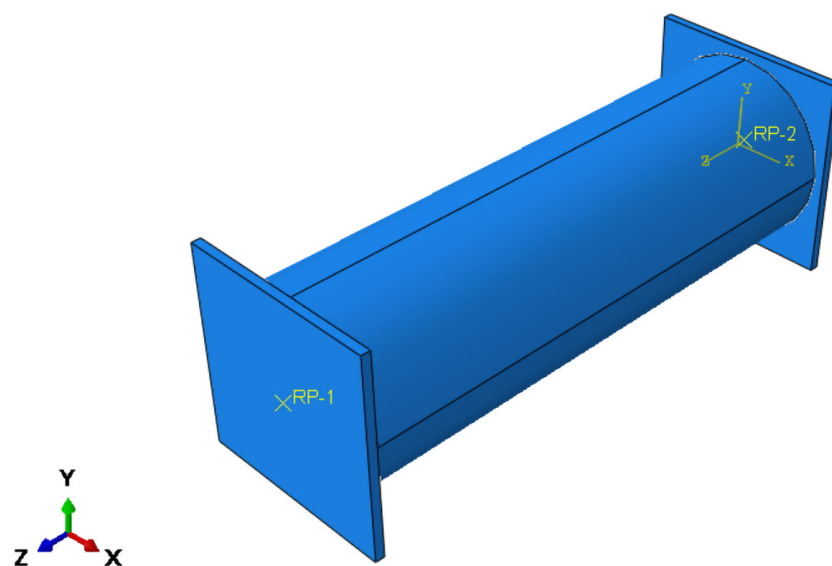


Figure 11. Boundary conditions.

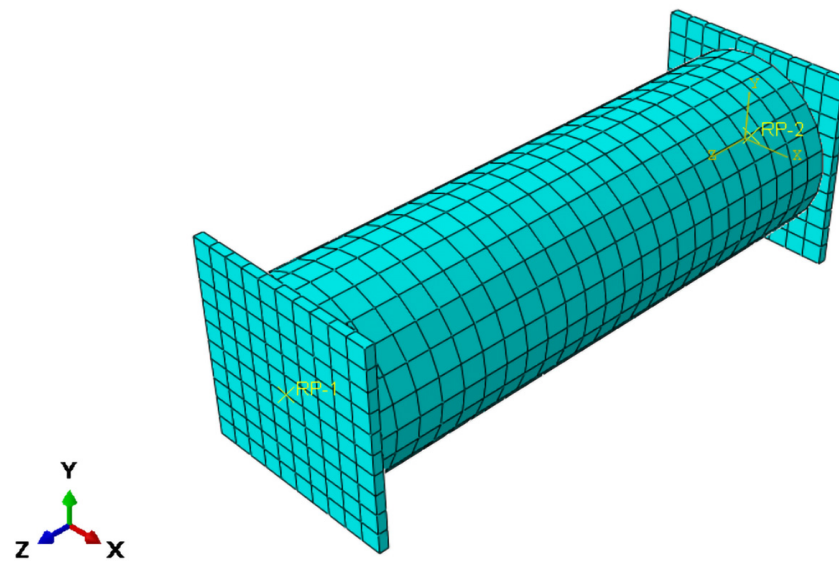


Figure 12. Grid division.

4.2. Model Verifications

The test results are compared and analysed with the finite element model data, as shown in Figure 13. It can be found that the trends of both curves are basically the same, but the simulated value of the curve is higher than the experimental value, which is because the simulated data are from the model in the ideal state of the force of the identified situation. Although the model takes into account the effect of the initial defects, due to the test of the instability and diversity of the factors, such as recycled aggregate distribution not being homogeneous, the self-consolidating concrete and the steel tube's own properties and other problems, all the test values are lower than the finite element model values, but all the errors are within 5%, which is an acceptable error range, proving the reasonableness of the finite element analysis.

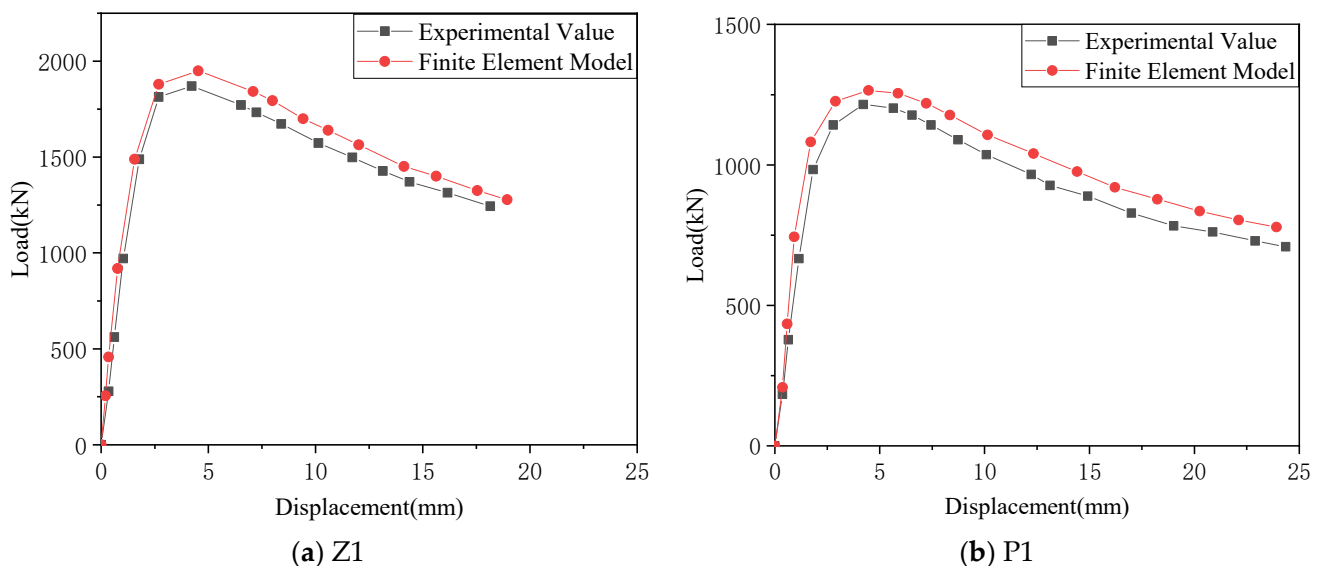


Figure 13. Cont.

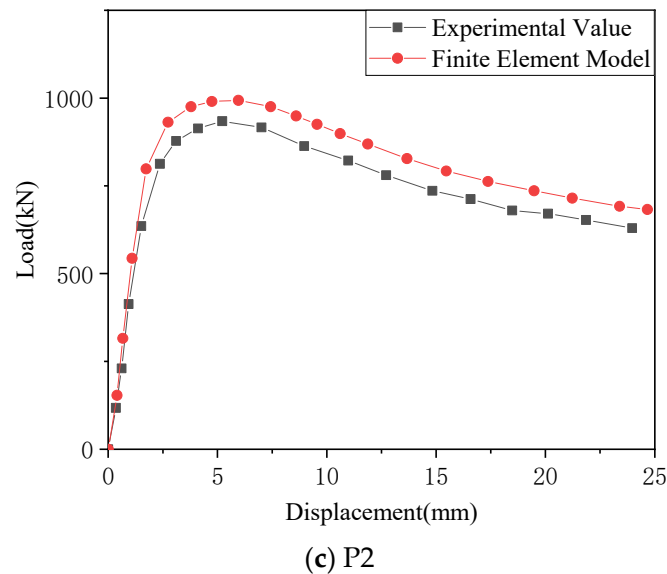


Figure 13. Test results and finite element model data validations.

4.3. Analysis of Influencing Factors

4.3.1. Steel Thickness

The thickness of the steel tube is one of the important parameters affecting the load carrying capacity of members. The thicknesses of steel tube were chosen as 4 mm, 5 mm, and 6 mm, and the simulated load–deformation relationship curves are shown in Figure 14. When the wall thickness of the steel tube increased from 4 mm to 5 mm, its ultimate load capacity increased by 12.1%, and when the wall thickness of the steel tube increased from 5 mm to 6 mm, its axial compressive ultimate load capacity increased by 4.5%. The ultimate bearing capacity increased with the increase in steel tube wall thickness. It could be seen from the curves that the elastic stiffness of the members did not significantly change with the increasing wall thickness of the steel tubes, and after the elastic-plastic stage, the stiffness of the members increased with the increasing wall thickness of the steel tubes, and at the same time, the restraining effect of the core concrete also increased accordingly.

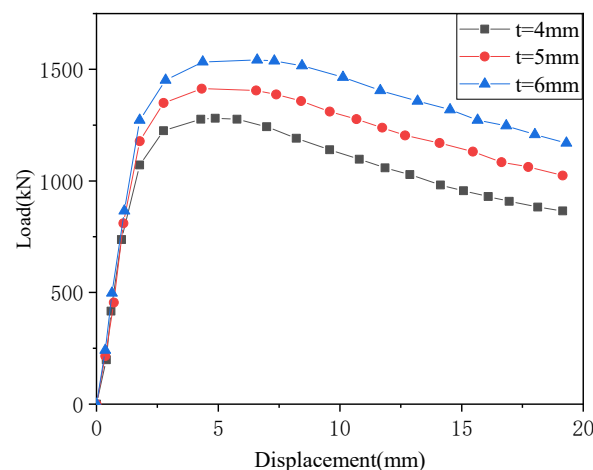


Figure 14. Load–displacement curves for different steel tube thicknesses.

4.3.2. RLA Particle Sizes

In order to study the effect of recycled large aggregate particle size on the load carrying capacity of the members, recycled aggregates within the range of 55 ± 5 mm, 70 ± 5 mm, and 85 ± 5 mm were selected, and the load–deflection relationships of the members were obtained, as shown in Figure 15. When the particle size of the recycled large aggregate was

raised from 55 ± 5 mm to 70 ± 5 mm, the ultimate load capacity was reduced by 1.08%, and when it was raised to 85 ± 5 mm, its ultimate load capacity was reduced by 1.78%. When the aggregate particle size was changed at the same time, the stiffness of each member was almost unchanged, the effect on the peak load was negligible, and the change in each member was almost the same in the descending section, so it can be seen that the change in the aggregate particle size has little effect on the stiffness of the members.

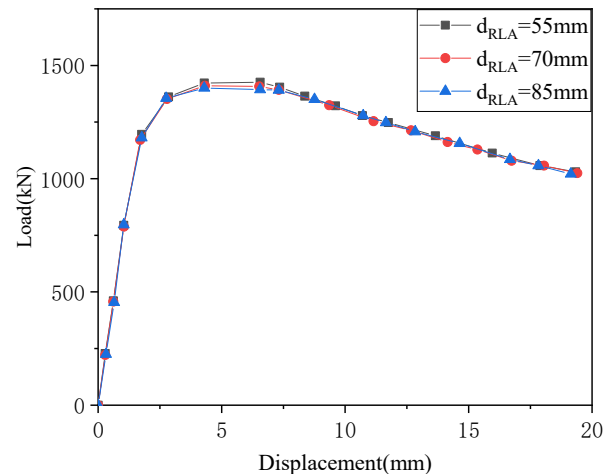


Figure 15. Load–displacement curves for different RLA particle sizes.

4.3.3. Strength of Self-Compacting Concrete

The change in self-compacting concrete strength inevitably has an effect on the overall strength of core concrete. The self-compacting concrete strengths were set to be C30, C40, and C50, and the ultimate loads of the members increased by 6.96% and 12.4% when the strength of self-compacting concrete increased from 30 MPa to 40 MPa and 50 MPa, respectively. From Figure 16, it can be seen that with the increase in the self-compacting concrete strength, the stiffnesses of the members in the elastic stage tended to be the same, and after entering the elastic-plastic stage with the change in the self-compacting concrete strength, the stiffness of the members also became larger, but the ductility of the members did not change significantly.

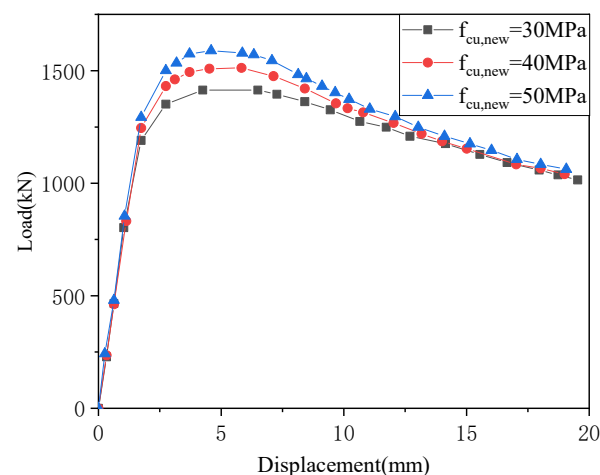


Figure 16. Load–displacement curves for different SCC strengths.

4.3.4. Length-to-Diameter Ratio

The aspect ratio is used as an important factor that must be examined for RLASCC-ST-LCs. The L/D ratios of 8, 10, and 12 were set for modelling analysis. From the curves in the Figure 17, it can be concluded that when the L/D ratio increased from 8 to 10, the

ultimate load capacity was reduced by 4.78%, and when the L/D ratio increased to 12, its axial compressive ultimate load capacity was reduced by 10.51%. The ultimate load capacity of the RLASCC-ST-LCs decreased with the increase in the L/D ratio. It can be seen from the curves that the stiffness of the members in the elastic phase decreased with the increase of the L/D ratio, the ductility of the members was affected to a certain extent due to the second-order effect, and the trend of the latter part of the curves appeared to have a certain degree of fluctuation.

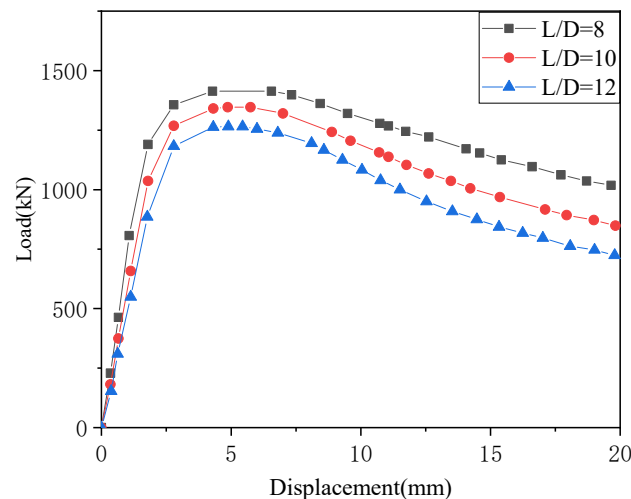


Figure 17. Load–displacement curves for different length-to-diameter ratios.

5. Prediction Formulas of Ultimate Bearing Capacity

Considering the influences of load eccentricity and slenderness ratio on medium-long column members, this paper combines the relevant calculation formulas of ultimate bearing capacity in Technical Specification [38] and Reference [36], and proposes the theoretical formulas for the bearing capacities of the RLASCC-ST-LCs as follows:

$$N_l = \varphi_e \varphi_l N_s \quad (9)$$

$$N_s = A_{sc} \cdot f_{scy} \quad (10)$$

$$f_{scy} = (1.5 + 1.02\xi) f_{c,com} \quad (11)$$

where

N_l is the bearing capacity of the RLASCC-ST-LC (kN),

φ_e is the bearing capacity reduction coefficient considering the influence of eccentricity,

φ_l is the bearing capacity reduction factor considering the influence of slenderness ratio,

N_s is the short column bearing capacity (kN),

f_{scy} is the index of strength (MPa).

The bearing capacity reduction factors φ_l and φ_e are calculated according to the following formulas:

$$\varphi_e = \begin{cases} \frac{1}{3.92 - 5.16\varphi_1 + \varphi_1 \frac{e_0}{0.3r_c}} & e_0/r_c \geq 1.55 \\ \frac{1}{1 + 1.85 \frac{M_2}{N r_c}} & e_0/r_c \leq 1.55 \end{cases} \quad (12)$$

$$\varphi_l = \begin{cases} 1 & L_e/D \leq 4 \\ 1 - 0.0226(\frac{L_e}{D} - 4) & 4 < L_e/D \leq 30 \\ 1 - 0.115\sqrt{\frac{L_e}{D} - 4} & L_e/D > 30 \end{cases} \quad (13)$$

In order to verify the accuracy of the proposed formulas, the data obtained from the simulations are collected and the errors are verified with the calculated values of the proposed formulas. Through the data summary of Table 4, it can be found that the calculation results are in good agreement with the simulation data, the errors are within 8%, and they are within the acceptable accuracy range. It is concluded that the RLASCC-ST-LC bearing capacity formulas proposed in this section are reasonable.

Table 4. Comparison of bearing capacity data.

Numbering	$N_{simulation}$ (kN)	$N_{calculation}$ (kN)	$N_{calculation}/N_{simulation}$
Z1	1873	1899	1.014
Z2	2065	2098	1.016
Z3	2236	2274	1.017
Z4	2071	2128	1.028
Z5	2059	2116	1.028
Z6	2166	2220	1.025
Z7	2245	2283	1.017
Z8	2192	2218	1.012
Z9	2309	2366	1.025
Z10	1970	2050	1.041
Z11	1887	1970	1.044

6. Conclusions

In summary, this paper presents a comprehensive investigation on the damage behaviours of RLASCC-ST-LCs under axial loading. The finite element software ABAQUS was used to establish the model, and the influences of steel pipe thickness, recycled large aggregate particle size, self-compacting concrete strength, and the length-to-diameter ratio on the performance law of the member were explored. The following conclusions can be drawn.

- (1) RLASCC-ST-LC buckling damage under longitudinal loading is basically the same as the damage of ordinary steel pipe concrete. With the increase in load eccentricity, the load carrying capacity of the member is weakened, and the initial stiffness is reduced. The increase of the eccentricity causes the stress gradient of the cross-section of the member to become larger, and thus favours the redistribution of stress in the cross-section, so that the member shows better ductility.
- (2) The finite element modelling approach proposed in this paper can better simulate the compressive damage process of RLASCC-ST-LCs. The ultimate load capacity of the steel tube increased by 12.1% when the wall thickness increased from 4 mm to 5 mm, and by 4.5% when it increased to 6 mm. The ultimate load capacity of the recycled large aggregate decreased by 1.08% when the particle size increased from 55 ± 5 mm to 70 ± 5 mm, and by 1.78% when it increased to 85 ± 5 mm. When the strength of self-compacting concrete increased from 30 MPa to 40 MPa and 50 MPa, the ultimate load of the members increased by 6.96% and 12.4%, respectively. When the L/D ratio was raised from 8 to 10, the ultimate load capacity decreased by 4.78%, and when the L/D ratio was raised to 12, its ultimate load capacity decreased by 10.51%.
- (3) The ultimate bearing capacity of the specimens increased with the increases in the thickness of the steel pipe and the strength of the self-compacting concrete, and decreased with the increase in the length-to-diameter ratio, but the effect of the RLA particle size on the ultimate bearing capacity was limited.
- (4) Based on the correction of the calculation formulas in the design code, this paper proposes the theoretical calculation formulas for RLASCC-ST-LC ultimate compressive load capacity. The results show that the results obtained from the theoretical calculations are in good agreement with the available data, verifying the accuracy of the formulas proposed in this paper.

These findings intuitively reflect the performance of RLASCC-ST-LCs under axial loading, which provides a valuable basis for application in practical engineering, explores an effective way to promote sustainable development, and provides a reference for deepening research on this type of structure.

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