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Rehabilitation of Composite Column Subjected to Axial Load

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Abstract

Concrete Filled Double Steel Tube (CFDST) columns are a modern technique of composite structural element that has fire resistance and has been adopted in high-rise building structures. The Concrete Filled Steel Tube (CFST) columns also have high strength and ductility due to composite action. This type of column CFST can sustain a heavy load with high performance and has been adopted in recent years in many countries around the world. The aim of the present work is to study the behavior and strength of rehabilitation of composite columns that are made from concrete core and surrounded steel tubes under the effect of axial compression loads with different height to diameter ratios such as 5.46, 10.91, and 16.37, respectively, by experimental tests. Double skin methodology is adopted to repair the damaged columns that were tested up to 85% of the ultimate load. Strength column capacity of double skin columns, axial and buckling deformations with axial and buckling strains are investigated. Test results showed that the repaired specimens up to 85% of the ultimate load had the same strength carrying capacity as compared with the control specimens, which had the same geometry. The ductility of an 800 mm specimen's height is greater than the other tested specimens, while the stiffness of short specimens becomes high.

Keywords: Double Skin Column; Composite Column; CFST; CFDST; Ductility.

1. Introduction

The term "composite structure" describes connections of two or more materials that differ in modulus of elasticity and Poisson's ratio to form a structural element such as a composite beam, column, slab, or shear wall. The CFST structural members combine the best mechanical properties of connected materials such as steel and concrete and provide the composite action. The composite action results from the mechanical interlock, friction, and adhesion at the interface of two materials [1-5]. The composite action occurs when the outer surface of concrete touches the surrounding surface of the steel tube, which makes the two materials work as a composite. The composite action relies on the degree of connections between the concrete column and steel tube. When there is full bound between these two materials due to full interaction, which means working as a unity, so that there is no slip at the interface of these two materials. In other cases, when there is partial interaction, that leads to developing slip at the interface. An imperfect connection between concrete column and steel tube at interface bonding due to the initial elastic phase under the effect of uniaxial load will have an unfavorable effect on the composite action that leads to a reduction in the elastic strength and an addition to the reduction in initial stiffness so that the interface bonding becomes more critical as the concrete strength increases [6].

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Different methodologies are available to repair the CFST column to prevent the buckled column or a reduction in CFST strength capacity. This type of composite column has good resistance against seismic loading and gives more ductility [1-5]. This type of column has the best mechanical properties of both steel and concrete materials and provides a composite action. The composite action between steel and concrete resulted from the mechanical interlock, friction, and adhesion. There are many methods that can be adopted to strengthen, re-strengthen, or repair composite columns based on the type and level of damage. The conventional repair method is to weld steel plate surrounding the damaged area of the column, but another technique that can be used is to cast large reinforced concrete jackets around the deformed columns [7].

Carbon fiber reinforced polymer CFRP sheet that wraps around the damaged column is another methodology known as structural respire. The compressive behavior of the damaged column by adopting this method was increased by the CFRP jackets. Ductility of the repaired column is enhanced, especially for long columns. Tao & Han (2007) [8]. Repaired of damaged columns by using concrete jackets and steel tubes that were placed surrounding them. This method causes an increase in axial loads. Also, notice that the stiffness and strength capacity of the repaired column increased so that it could be reinstated to its original condition [1]. Confining of damaged column in addition to the used schemes of the FRP family such as GFRP, CFRP, AFRP, and BFRP (Glass, Carbon, Armed, and Basalt fiber reinforced polymer), there are different ways that include internal steel sections, stiffeners, tie bars, and external rings [9].

Among all of the above methods, these methods vary in effectiveness. External rings may be an effective and significant practical solution due to their increasing the strength capacity of the column, ductility, and stiffness. The steel rings are welded externally around the steel tube. Rings can enhance the strength capacity and increase the column stiffness with a decrease in the degradation strength rate. The presence of steel rings creates confinement of the CFST columns. Almasslawi et al. (2020) [10], investigated the behavior of a rapidly buckling CFST column that was subjected to an axial compression load. The column type that was focused on in experimental tests was the concrete-filled circular steel tube (CFCST) that was adopted to repair the buckled CFST columns. The buckled CFST columns were placed centrally inside the CFCST that consisted of a larger diameter steel tube, and concrete was poured into the gap between the deformed CFST column and the larger tube. The main parameters were adopted as tube thicknesses in which all specimens were tested to failure under the effect of axial compression loads. The study included the improvement of strength capacity due to the composite action of the CFST column. The buckled column was, as usual, placed inside a large steel tube and the concrete filled the outer buckled column (inside the large steel tube) in which the study took into account different steel tube thicknesses of large diameter. Test results were compared with control undamaged columns. According to the findings, the adopted methodology for repairing damaged columns restores the strength capacity to 97–100% of the original capacity of the undamaged CFST columns.

Ekmekyapar & Al-Eliwi (2017) [11] investigated stub columns in the case of CFDCST as a concrete field with double circular steel tube through experimental. In this re-strengthening, the deformed column is centered inside a larger column with a diameter calculated to give the same strength as an un-deformed column in which the space between the deformed and larger steel tube is filled with concrete. Different parameters were adopted, such as core compressive strength, shell compressive strength, outer diameter of the steel column, and the yield strength of steel tube. The concrete that was used to repair the deformed columns is classified as normal and high strength concrete. To check out the re-strength of deformed columns, the tested columns were up to 85% of the failure load, then the repair procedure was applied and the new column tested. The experimental results showed that the adopted method of double steel tube improved the ductility and stiffness of the composite columns, so that it can be used to repair the deformed column. Based on the test results, a different concrete grade can be used to repair the deformed column. The presence of the outer steel tube gave more confinement to the deformed column and an increase in strength capacity.

Ramanagopal (2018) [12] studied the performance of the double tube stub column by an experimental approach. Different parameters were considered, such as the concrete grade of 30, 40, and 50 MPa with a diameter/thickness ratio of 33.3 and a height/diameter ratio of 3. Experimental results showed that the stiffness of the composite column was enhanced because of the extra inner carbon steel tube. Typical failure modes of the outer tubes were local outward buckling and occurred near mid-height for some of the column specimens. Yan & Zhao (2020) [13] investigated the compressive strength of the short double skin of CFST columns. A total of twenty-four specimens were cast and tested under axial compression load. Different parameters were considered, such as the outer steel tube diameter, the thickness of the steel tube and concrete compressive strength of 29.0, 37.5, and 51.0 MPa. Test results showed that the confinement coefficient decreases as the compressive strength of concrete increases; this coefficient also decreases if the diameter/thickness ratio is small. Increases in concrete compressive strength and steel tube thickness significantly improve the CFST strength capacity.

Yan et al. (2021) [14] studied the confinement of CFST and CFDST as double skin columns based on the compressive strength model. Various parameters were adopted, such as the outer diameter and thickness of the steel tube and the concrete compressive strength. Test results indicated that the confinement of CFDST was less than CFST. He et al. (2019) [15] studied the confinement effects on CFST columns by using different grades of compressive strength of concrete. A total of six specimens were cast and tested under the influence of axial compression load with different parameters like concrete compressive strength and diameter/thickness ratio of steel tube. The concrete grade ranged between 29.5, 43.5, 58.0, and 81.6 MPa in the first group, and 32.0 and 64.0 MPa in the second. The steel tube diameter and thickness were 165.2 and 3.7 mm for the first group, while 230 mm in diameter and 2.3 mm in thickness for group two. According to experimental investigation, the confinement factor influenced the confinement effect, in which a higher confinement factor gave more confinement effect. Also, the low compressive strength of concrete gave better behavior and more ductility as compared with high grade concrete.

Ke et al. (2022) [16], investigated the axial compression performance of CFST columns under the effect of temperature load. Fourteen specimens after exposure to high temperatures. It has been taken into consideration different parameters such as heating temperature, steel tube diameter and concrete cover thickness. Based on test results, it was concluded that the load-displacement curves of the specimens were significantly affected by high temperatures, while the influence of steel tube diameter and concrete cover thickness was relatively weak. Naji et al. (2021) [17] looked at the behavior of reinforced concrete columns that have been rehabilitated with a concrete liner, a confinement FRP fabric, a steel liner, or a metal corner cage. The investigation showed that the usage of fiber reinforcement polymer has no effect on the weight of the structures that gave the best results and behavior.

Hadi et al. (2021) [18] studied the influence of CFRP wrap on pre-damaged reinforced concrete columns. According to test results, the mechanical behavior, particularly for ductility, stiffness, and ultimate load capacity, was improved and increased due to the enhancement with CFRP wrap. Jamkhaneh & Kafi (2019) [19] assessment of partially encased composite column behavior under compressive and bending moment loading. Test results showed that the failure mode was similar for all tests the concrete cracking coupled with the steel flange buckling. Al-Adawy et al. (2021) [20] studied the performance of short concrete composite columns under the effects of uniaxial or biaxial eccentric loading. A comparison between the behavior of each tested column before and after being repaired by applying different types of sheets of carbon fiber reinforced polymer layers was made. The comparison showed that wrapping composite steel columns with carbon fiber reinforced polymers is considered a very effective technique for repairing.

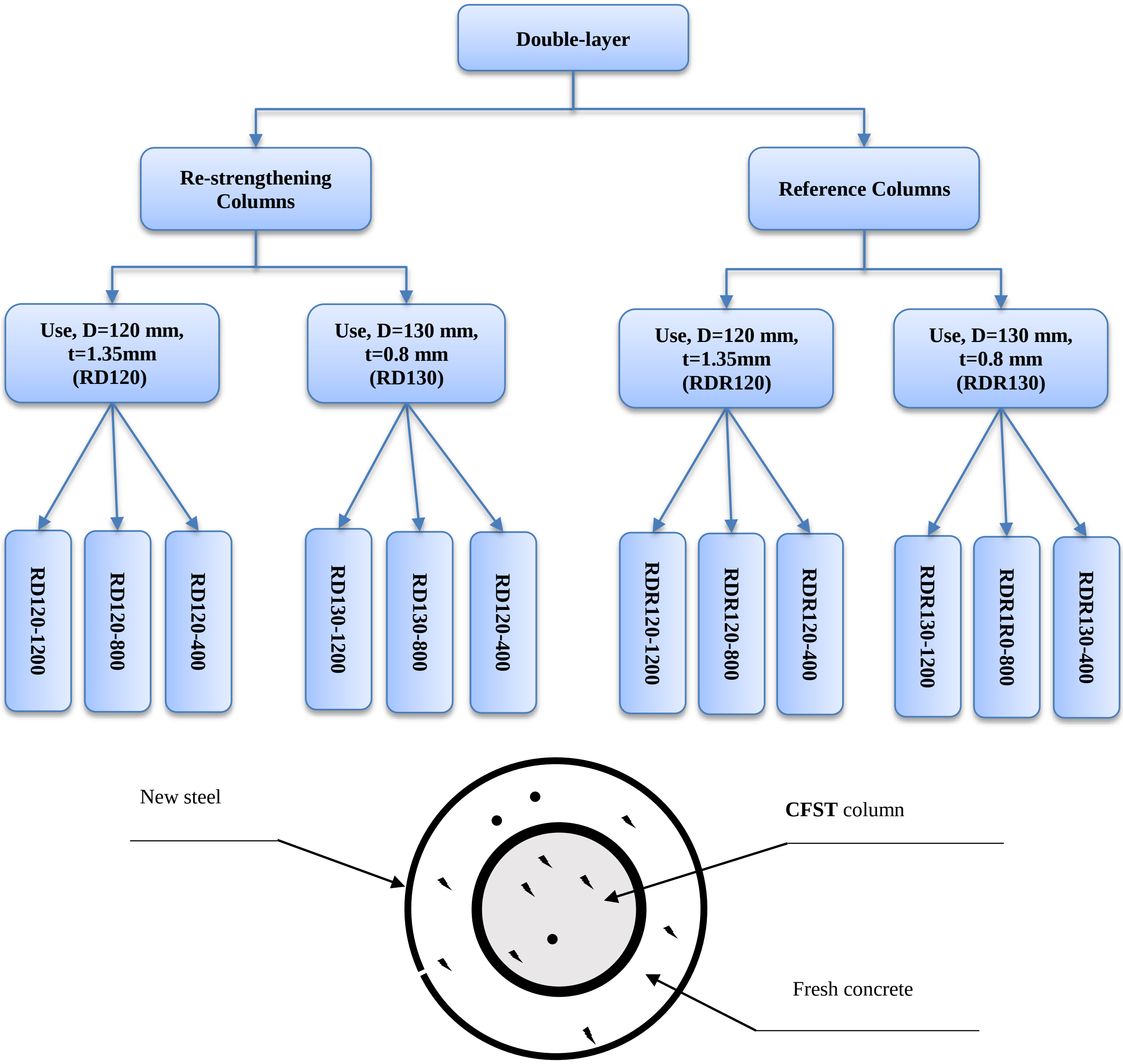
The gap that try to fillet by this research is to proposed methodology to respire the damaged composite column by apply double layer skin with same concrete grade of inner concrete.

2. Experimental Program

The adopted raw materials to cast the concrete and form the steel tube section, in addition to the manufactured composite double layer for repair of tested composite columns, preparations are summarized. Firstly, the steel was formed by steel plate and welded by argon, and then the concrete core was casted inside the steel tube to form a composite column. The single layer composite column specimens were tested for each height up to failure, and some of these specimens were tested up to 85% of the ultimate load. Load control was adopted to apply load to each specimen up to failure. The total height of the adopted columns is 400, 800, and 1200 mm. The inner tube thickness (1.35 mm) and concrete core diameter (76 mm). The specimens were tested under the effect of axial load up to 85% of the ultimate strength capacity of each specimen. The layout of experimental work is shown in Figure 1. The concrete and steel tube mechanical properties are listed in Table 1, in which the concrete mechanical properties tests are based on ASTM C39 [21] for compressive strength. Specification of ASTM C496 [22] for splitting tensile strength. Standard specification ASTM C293 [23] for modulus of rupture. Testing material specifications according to ASTM C597 [24] for modulus of elasticity. The mechanical steel tube was tensile tested based on ASTM A370 [25].

Table 1. Mechanical properties for concrete and steel tube

Concrete			Steel tube(inner and outer)	
Compressive strength f'c (MPa)	Modulus of elasticity (MPa)	Modulus of rupture (MPa)	Yielding tensile strength fy (MPa)	Modulus of elasticity (MPa)
48	41500	7.55	295	205000



Re-strengthening CFST column

Figure 1. Layout of experimental work

Table 2. Models marks

Specimen mark	Specimen description	Specimen geometry								
		Concrete diameter (mm)		Steel tube inner and outer (mm)						
				Diameter		Tube thickness		Height	Height/diameter	Dia./Thick.
		Inner	Outer	Inner	Outer	Inner	Outer			
CFST100%	Composite	73.3	-	76	-	1.35	-	400 800 1200	5.46 10.91 16.37	56.30
CFST85%	Composite	73.3	-	76	-	1.35	-	400 800 1200	5.46 10.91 16.37	56.30
RD120	Composite	73.3	117.3	76	120	1.35	1.35	400 800 1200	3.33 6.67 10	88.89

								400	3.33	
RDR120	Composite	73.3	117.3	76	120	1.35	1.35	800	6.67	88.89
								1200	10	
								400	3.08	
RD130	Composite	73.3	128.4	76	130	1.35	0.8	800	6.15	162.5
								1200	9.23	
								400	3.08	
RDR130	Composite	73.3	128.4	76	130	1.35	0.8	800	6.15	162.5
								1200	9.23	

Figure 2 shows the specimens of hollow steel section, Figure 3 shows the double layer composite columns and Figure 4 shows the test setup. Each specimen, a hollow steel tube, was filled with concrete to represent the concrete core and make the top surface smooth so that the applied load is distributed uniformly. The bottoms of the steel tubes were closed by applying silicone and thick steel plates that were removed before the specimen was tested, and then, after that, the concrete was poured into the insides of the steel tubes. The top of each specimen (rounded 5 mm) was left empty to level the concrete with epoxy. Each specimen was placed at the center of the machine test to avoid eccentric loading. Each specimen was setup in the middle of the testing machine, and after that the axial compressive load was applied at a constant rate in which axial, buckling displacement, and strain were recorded for each applied load step up to 85% of the failure load for each specimen. Double layer specimens were tested and an additional to repair of tested specimens up to 85% to check out the rehabilitation method to re-strength the tested composite column specimens. Two different outer diameters are adopted, such as 120 mm with 1.35 mm thickness and 1.35 mm with 0.80 mm thickness, to repair the tested specimens up to 85% of the ultimate load.

Figure 2. Hollow steel sections

Figure 3. Composite sections CFSTs double layer

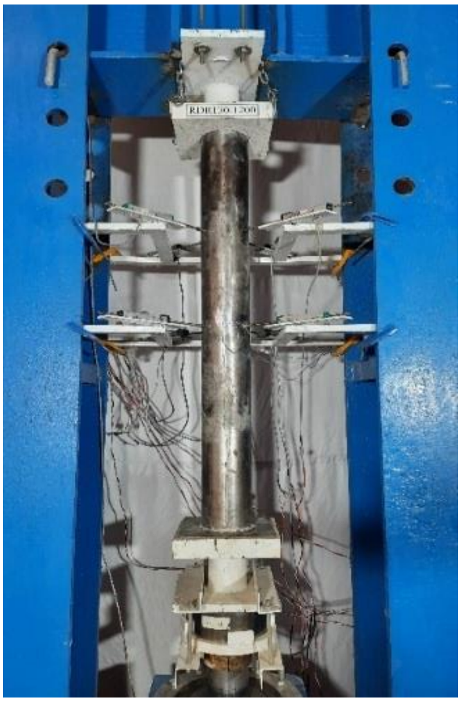


Figure 4. Test setup

3. Results

Axial and lateral deformation as buckling are recorded during tests for all specimens and an additional to axial and lateral strain for each specimen. Tables 3 and 4 lists the test results as axial displacement and axial strain, buckling displacement and buckling strain based on maximum and failure load in which the specimen CFST100% represent the single layer specimen that tested up to failure (100% ultimate load), CFST85% is the specimen that tested up to 85% from the ultimate load, RD120, RD130 are the specimens that repaired the CFST85% by external diameter 120 and 130 mm respectively, specimens RDR120 and RDR130 represents the control double layer specimens by adopted 120 and 130 mm outer diameters respectively. Figures 5 to 16 shows the axial, buckling deformation, axial and lateral strain for all tested specimens. The specimens behaved as nonlinear after pass the linear zone that end at the point of inflection when the strain become more (or displacement) so that the specimen become weak and reduce in stiffness due to reduce in modulus of elasticity of the specimen. The location of inflection point for each specimen differ than other specimens that relies on many parameters such as slenderness ratio and strength capacity of the specimen. Test results showed that increased in outer tube thickness lead to increase in strength capacity and reduce in deformations such as in control specimens RDR120 when compared with strength capacity of specimens RDR130. The strength capacity of repaired specimens closes with that of control specimens (specimens RDR120 compared with RD120, specimens RDR130 compared with RD130) that is mean the repaired methodology design is suitable and accurate.

Table 3. Test Results-Axial displacement and axial strain based on maximum and failure load

Specimen mark	Specimen height (mm)	Test results					
		Maximum load (kN)	Failure load (kN)	Maximum axial displacement (mm)	Axial displacement at failure (mm)	Maximum strain	Strain at failure load
CFST100%	400	391.46	370.82	2.86	3.79	4812	5200
	800	367.71	328.03	3.66	6.08	7600	8926
	1200	327.33	268.25	6.72	10.44	2330	3260
CFST85%	400	331.35	---	2.35	---	---	---
	800	309.46	---	2.73	---	---	---
	1200	288.12	---	4.74	---	---	---
RD120	400	844.2	746.70	5.92	8.50	2190	2729
	800	835.33	758.70	4.32	8.02	8036	12020
	1200	762.51	740	7.42	7.55	8327	11340
RDR120	400	833.90	782	6.84	8.22	3161	4395
	800	831.67	795	5.76	8.22	8000	10130
	1200	797.18	787.00	6.26	7.21	9990	13110

RD130	400	815.00	650.15	7.31	10.13	769	1524
	800	811.58	696.69	7.40	10.16	2180	3680
	1200	683.20	632.11	10.48	13.43	845	1525
RDR130	400	814.94	768.06	6.82	8.16	1032	1473
	800	778.99	694.45	7.72	9.14	3986	5140
	1200	704.55	686.19	6.89	8.00	1860	2019

Table 4. Test Results-Buckling displacement and buckling strain based on maximum and failure load

Specimen mark	Specimen height (mm)	Test results					
		Maximum load (kN)	Failure load (kN)	Maximum buckling displacement (mm)	Buckling displacement at failure (mm)	Maximum strain	Strain at failure load
CFST100%	400	391.46	370.82	1.69	1.95	527	600
	800	367.71	328.03	8.00	8.75	3864	5690
	1200	327.33	268.25	11.98	24.44	2036	3014
CFST85%	400	331.35	---	1.44	---	---	---
	800	309.46	---	1.73	---	---	---
	1200	288.12	---	6.37	---	---	---
RD120	400	844.2	746.70	3.19	3.87	3845	5700
	800	835.33	758.70	4.5	6.18	4767	7474
	1200	762.51	744.00	6.24	7.08	760	1272
RDR120	400	833.90	782	2.35	3.93	3930	5920
	800	831.67	795	4.25	10.62	4410	6900
	1200	797.18	787.00	8.22	9.16	5872	8355
RD130	400	815.00	650.15	4.30	4.80	1380	2136
	800	811.58	696.69	4.63	5.10	1850	2962
	1200	683.20	632.11	7.90	12.10	415	490
RDR130	400	814.94	768.06	2.83	3.12	780	1020
	800	778.99	694.45	2.19	2.85	2842	4056
	1200	704.55	686.19	5.44	5.84	590	660

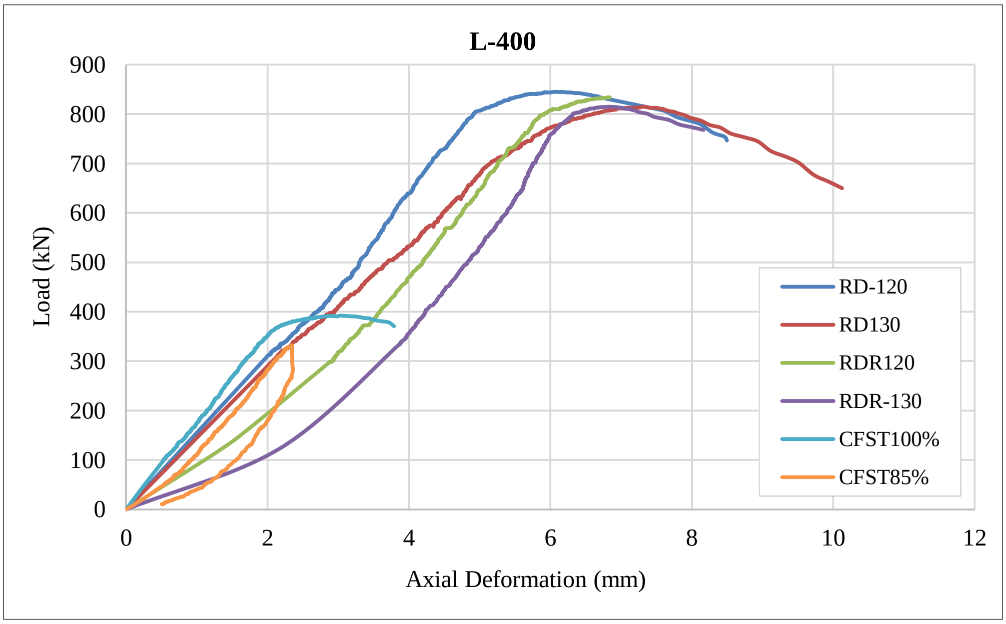


Figure 5. Load-axial deformation of 400 mm specimens

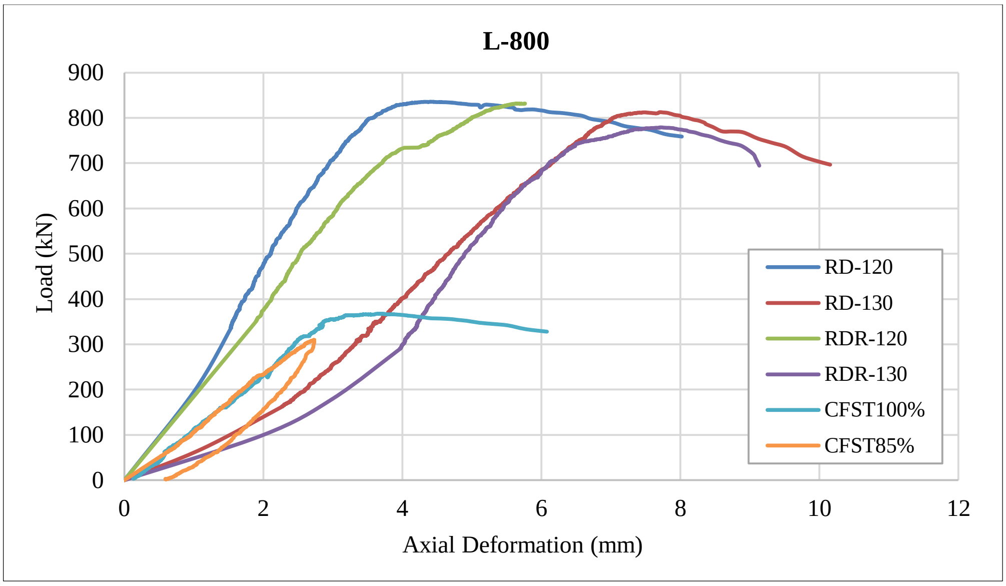


Figure 6. Load-axial deformation of 800 mm specimens

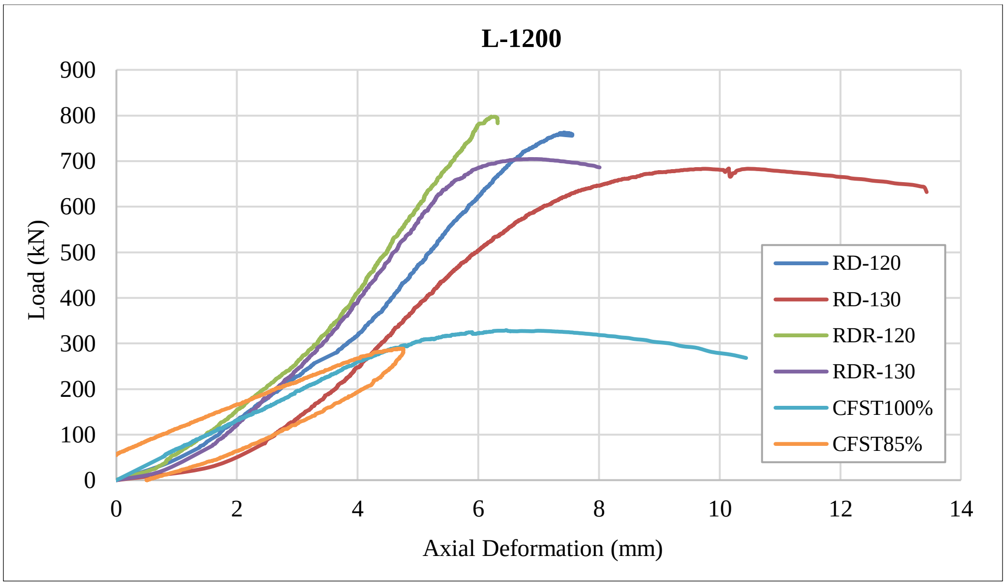


Figure 7. Load-axial deformation of 1200 mm specimens

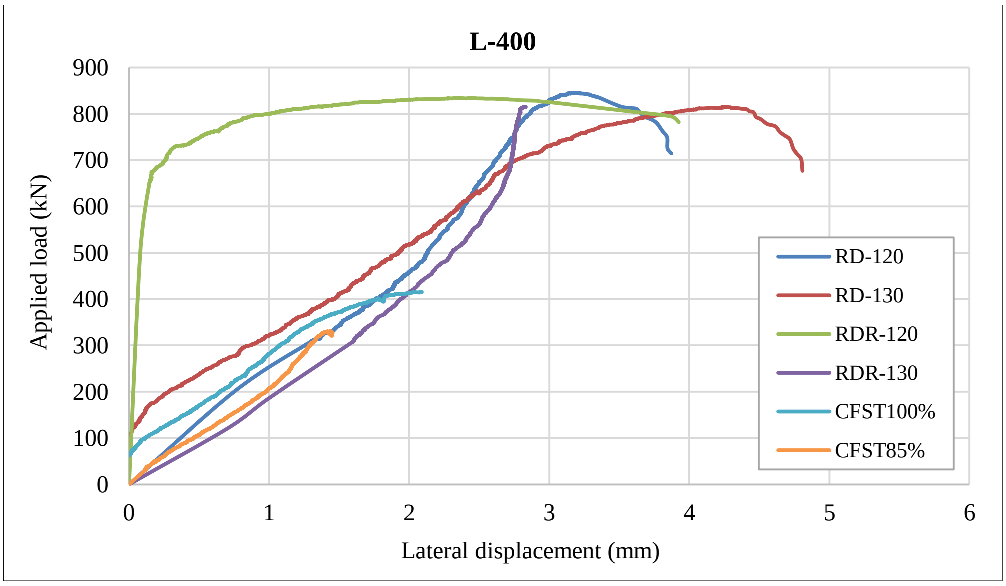


Figure 8. Load-buckling deformation of 400 mm specimens

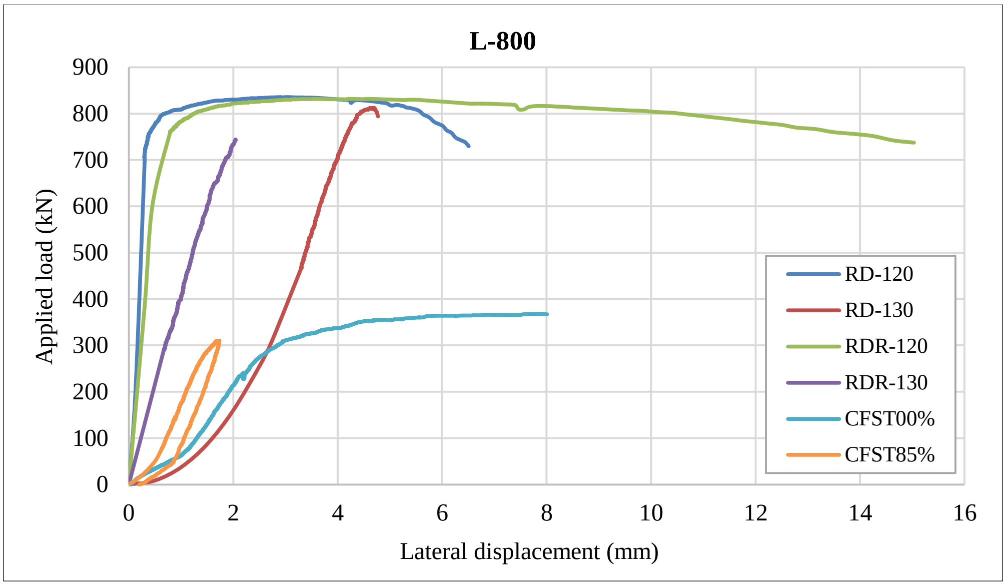


Figure 9. Load-buckling deformation of 800 mm specimens

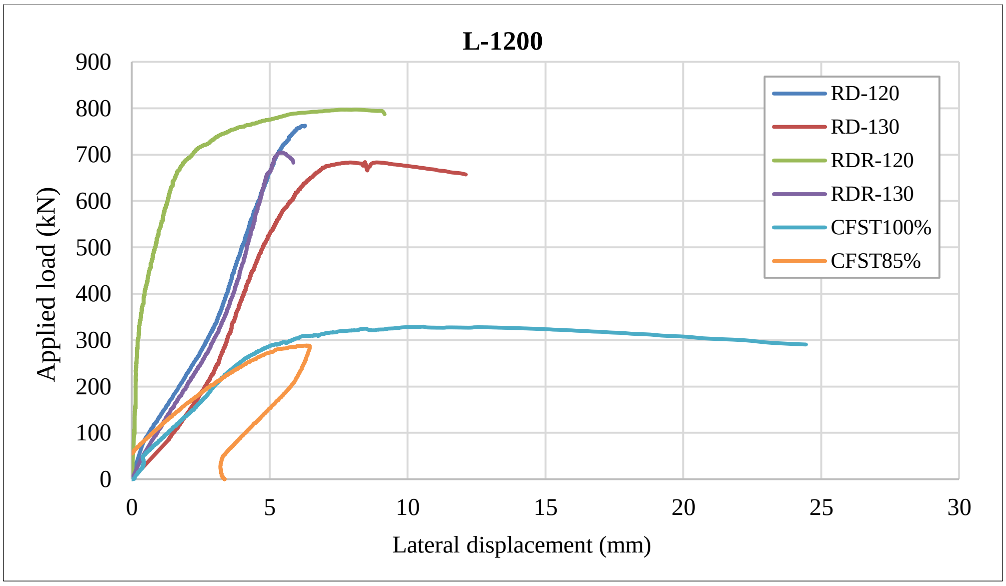


Figure 10. Load-buckling deformation of 1200 mm specimens

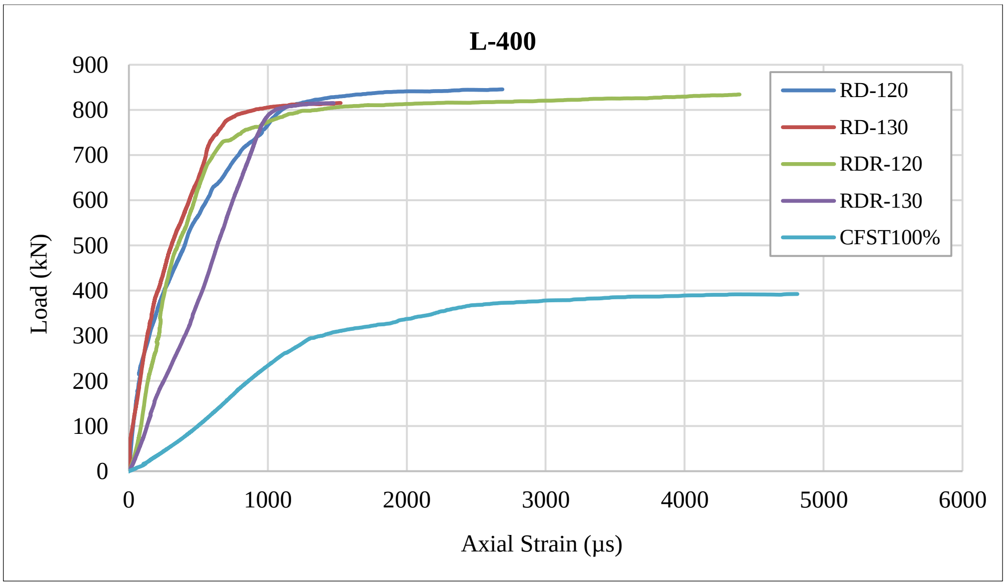


Figure 11. Load-axial strain of 400 mm specimens

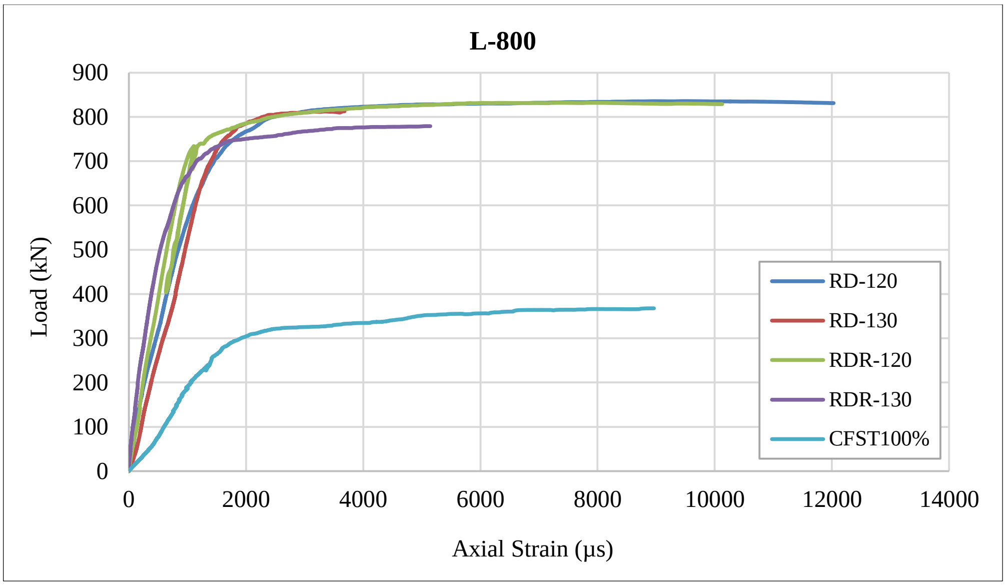


Figure 12. Load-axial strain of 800 mm specimens

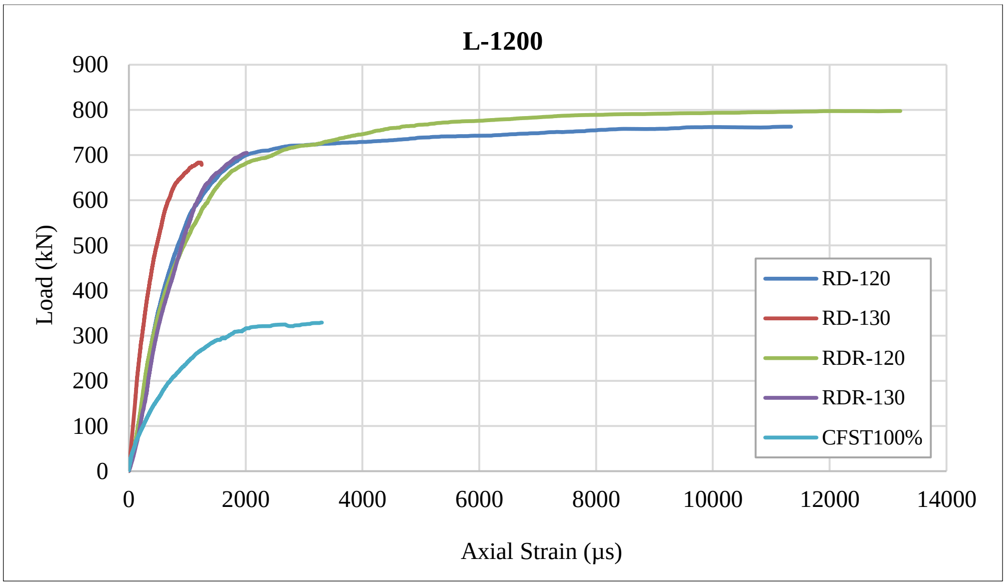


Figure 13. Load-axial strain of 1200 mm specimens

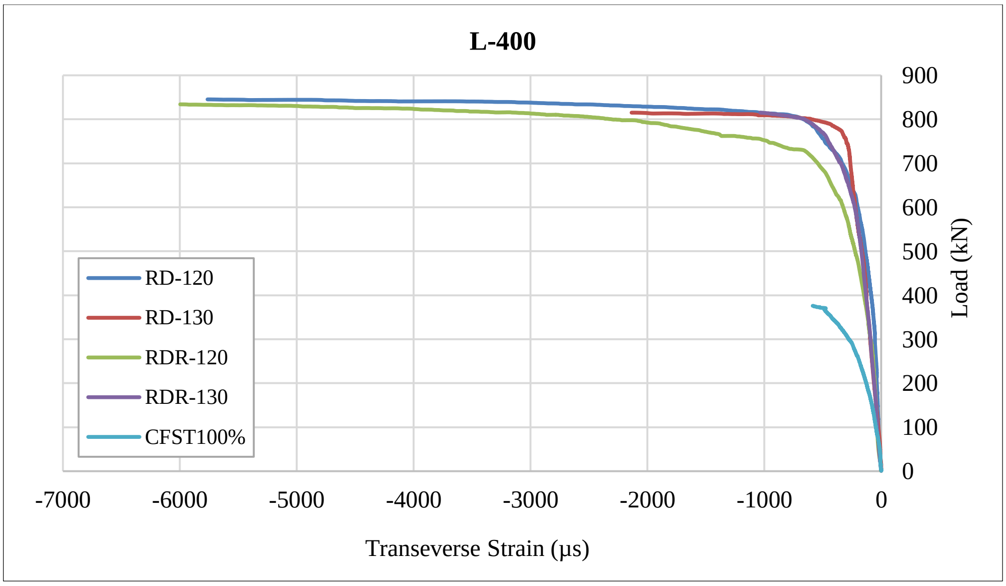


Figure 14. Load-transverse axial strain of 400 mm specimens

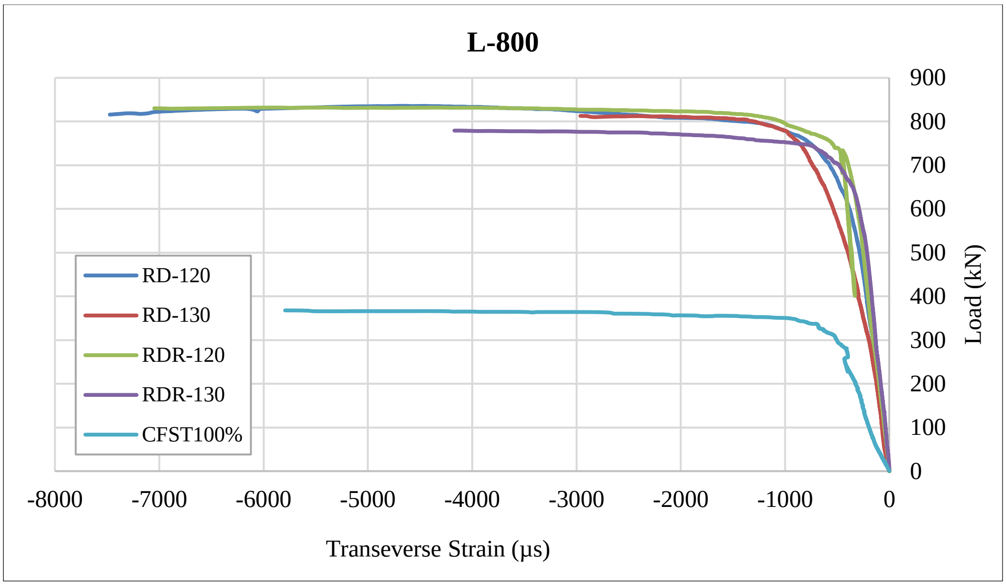


Figure 15. Load-transverse axial strain of 800 mm specimens

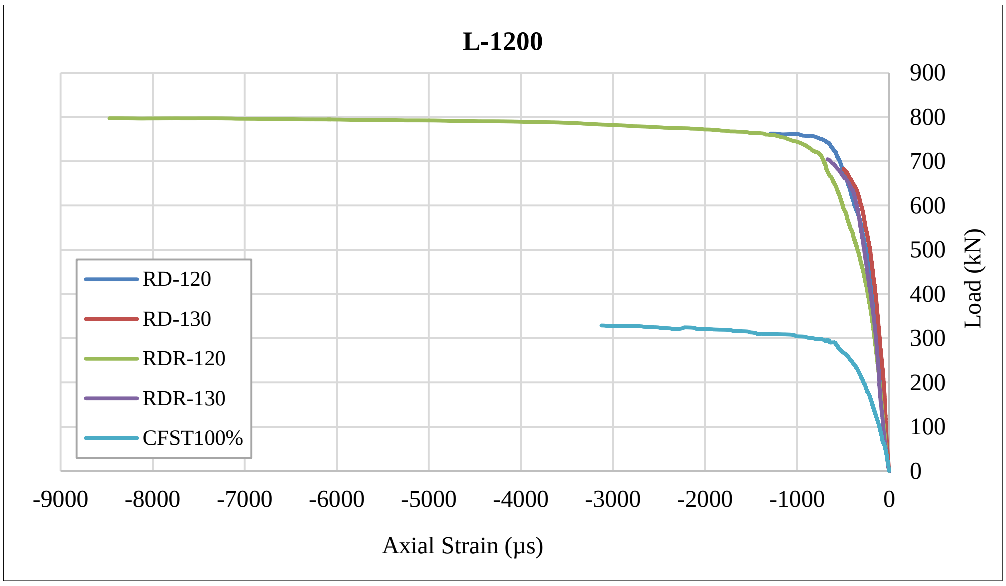


Figure 16. Load-transverse axial strain of 1200 mm specimens

Table 5 lists the ductility and stiffness for all tested specimens. Figures 17 to 20 represents the ductility and stiffness comparisons for all specimens for both axial and lateral (buckling) displacements. Ductility of the column as structural element classified as ability to carry considerable deflection prior to failure. The ductility of column as composite indicate and give sign of quality and is there failure that lead to prevent the collapse of the column under large deformations. The presences of steel tube in composite specimens improved the concrete property especially ductility and an additional to make confinement of concrete to increase the compressive strength of concrete core.

Table 5. Ductility and stiffness of all tested Specimens-Axial and buckling

Specimen mark	Specimen height (mm)	Ductility (Axial)	Stiffness (Axial) (kN/mm)	Ductility (Buckling)	Stiffness (Buckling) (kN/mm)
CFST100%	400	1.33	136.87	1.15	231.63
	800	1.66	100.47	1.09	45.96
	1200	1.55	48.71	2.04	27.32
CFST85%	400	---	141.00	---	230.10
	800	---	113.36	---	178.88
	1200	---	60.78	---	45.23

RD120	400	1.44	142.60	1.21	264.64
	800	1.86	193.36	1.37	185.63
	1200	1.02	102.76	1.13	122.20
RDR120	400	1.20	121.92	1.67	354.85
	800	1.43	144.39	2.50	195.69
	1200	1.15	127.35	1.11	96.98
RD130	400	1.39	111.49	1.12	189.53
	800	1.37	109.67	1.10	175.29
	1200	1.28	65.19	1.53	86.48
RDR130	400	1.20	119.49	1.10	287.96
	800	1.18	100.91	1.30	355.70
	1200	1.16	102.26	1.07	129.51

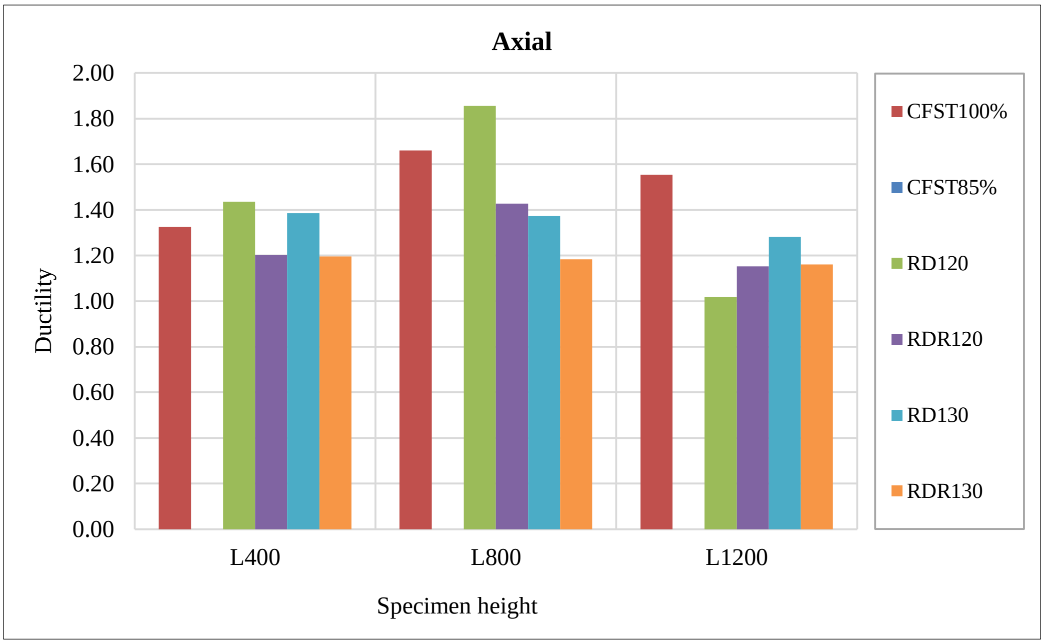


Figure 17. Ductility variations for all tested specimens-axial

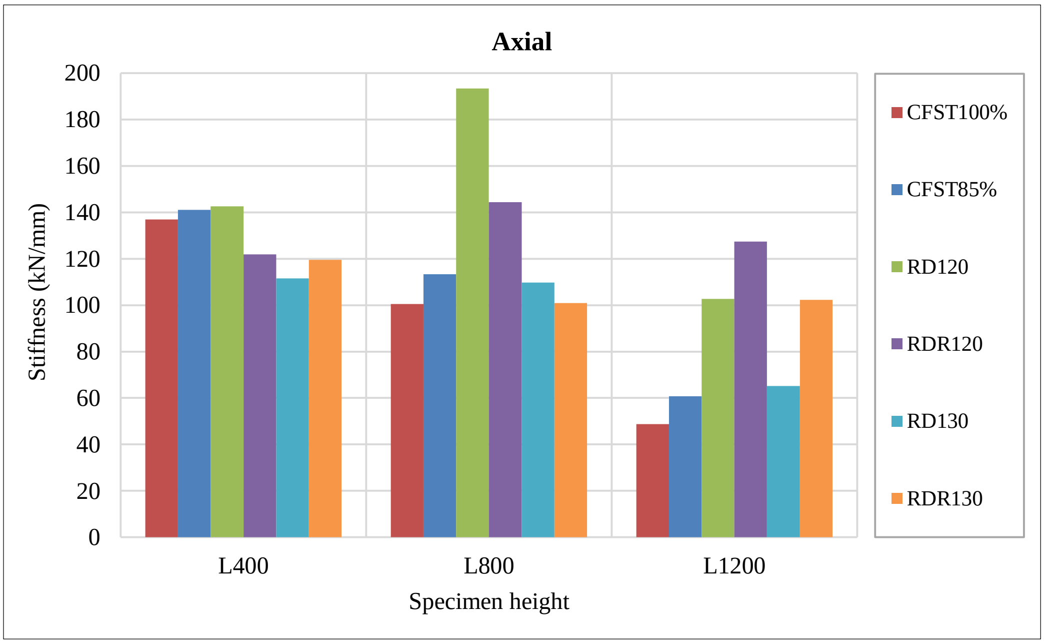


Figure 18. Stiffness variations for all tested specimens-axial

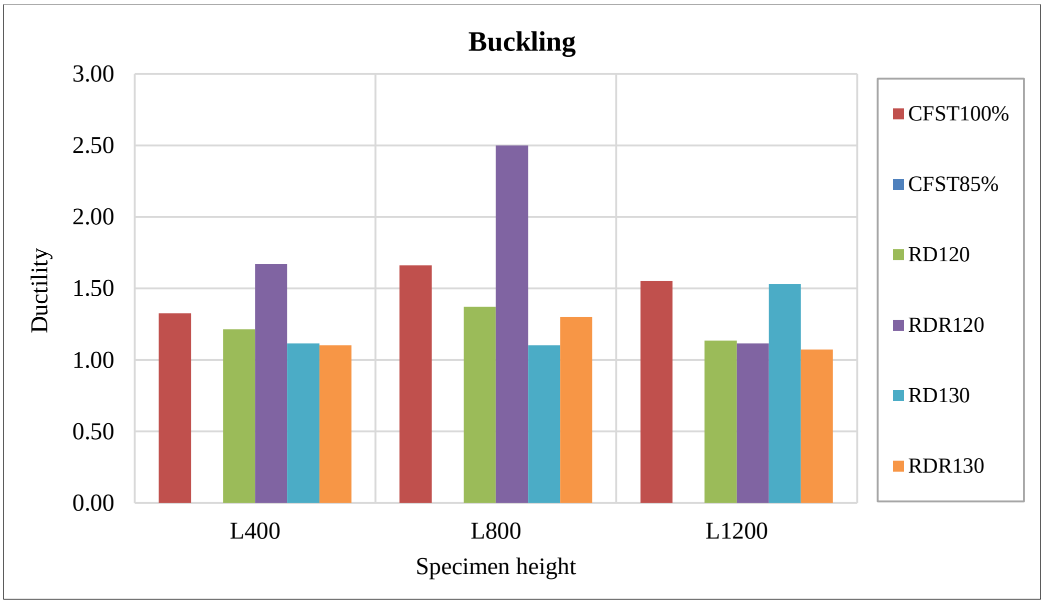


Figure 19. Ductility variations for all tested specimens-buckling

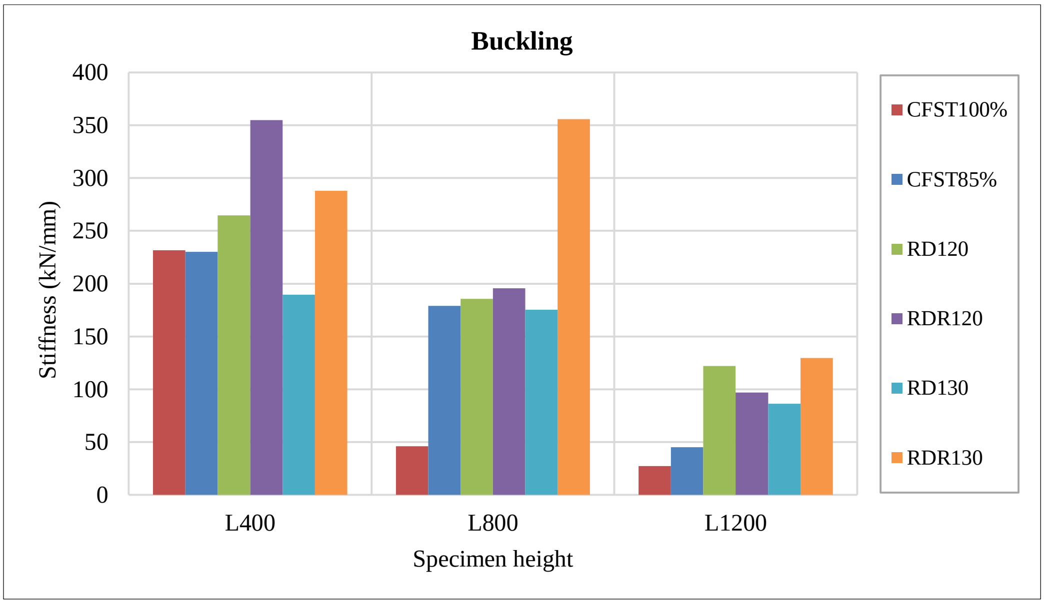


Figure 20. Stiffness variations for all tested specimens-buckling

The ductility U for all tested specimens calculated by divided the deflection at ultimate load Δ_u by the deflection at yield Δ_y as follow:

$$U = \frac{\Delta_u}{\Delta_y} \tag{1}$$

The RD120 and RD130 specimens gave ductility more than the repaired specimens with same diameter and tube thickness such as specimens RDR120 and RDR130. The ductility of specimens RD120 has ductility higher than that of specimens RD130 in case of specimen's height 400 and 800 mm. The higher ductility occurs in case of specimens have height 800 mm while the stiffness as axial and lateral in case of short specimens with height 400 mm. The stiffness of the tested specimens is the resistance of an elastic body to deflection or deformation by an applied force. The stiffness μ is defined as the ratio of the ultimate load pu to ultimate load deflection Δu as follow:

$$\mu = \frac{P_u}{\Delta_u} \tag{2}$$

The RD120 and RD130 specimens has stiffness greater than the repaired specimens have same diameter and tube thickness such as specimens RDR120 and RDR130 with height 400 and 800 mm. The stiffness of specimens RD120 is higher as compared with RD130 specimens in case of specimen's height 400 and 800 mm.

3.1. Mode of Failure

The failure mode for all specimens are illustrated in Figures 21 to 23. The CFST specimens that tested up to 100% full load, steel surrounded tube having yield and buckling for the global. In case of specimens that repaired by double layer as concrete and outer steel tube RDR120 and RDR130, global buckling occurs for all tested specimens due to progress of axial strain that developed that lead to make fracture in the outer steel tube. In case of double layers as control or repair, the presence of outer steel tube provides more confinement to the concrete core that the specimens behaved as composite action and make the specimens buckled failure mode.

a-RD120 b-RDR120 c-RD130 d-RDR130

Figure 21. Failure mode of all tested specimens with height 400 mm

a-RD120 b-RDR120 c-RD130 d-RDR130

Figure 22. Failure mode of all tested specimens with height 800 mm

a-RD120 b-RDR120 c-RD130 d-RDR130

Figure 23. Failure mode of all tested specimens with height 800 mm

4. Discussions

The smallest height specimens gave high strength capacity as compared with the same tested group due to the short specimen's low slenderness ratio when compared with the 800 and 1200 mm that led to increasing the buckling load resistance. The strength capacity of control specimens such as RD120 and RD130 compared with the repaired specimens, RDR120 and RDR130, is rounded to the same degree. That means the repaired methodology that was adopted successfully re-strengthened the tested specimens up to 85%. According to investigation results, the strength capacity for each specimen under control and repair is compared in Table 6. The ductility of CFST100% with an 800 mm height greater than other specimens within the CFST100% group shows the maximum displacement value is worthless compared to other specimens. Furthermore, the specimens have a maximum rounded thickness of 800 mm, indicating high ductility. The stiffness increases when the height becomes 400 mm or less. This kind of specimen has a low slenderness ratio that is directly related to more buckling. Axial longitudinal displacement (800 and 1200 mm) increases with height. The axial displacement relies on both (PL/AE) for the same rigidity, with displacement getting more.

Table 6. Strength carrying capacity comparisons within same material

Specimen mark	Specimen height (mm)	Strength load capacity (kN)	% Decreased in strength capacity based on height	% Repair/control	% Increase due to repair
CFST100%	400	391.46	---	N/A	---
	800	367.71	6.07	N/A	---
	1200	327.33	16.38	N/A	---
CFST85%	400	331.35	---	N/A	N/A
	800	309.46	6.61	N/A	N/A
	1200	288.12	13.05	N/A	N/A
RD120	400	844.2	---	---	115.65
	800	835.33	1.05	---	127.17
	1200	762.51	9.68	---	132.95
RDR120	400	833.90	---	98.78	113.02
	800	831.67	0.27	99.56	126.18
	1200	797.18	4.40	104.54	143.54

RD130	400	815.00	---	---	108.19
	800	811.58	0.42	---	120.71
	1200	683.20	16.17	---	108.72
RDR130	400	814.94	---	99.99	108.18
	800	778.99	4.41	95.99	111.85
	1200	704.55	13.55	103.13	115.24

5. Conclusion

Based on the experimental investigation for rehabilitation of tested composite column specimens under axial loadings, test results showed that when applied double layers as concrete core surrounded the inner steel tube of a tested composite single layer column and confinement by external steel tube gave close results as for strength capacity when compared with the control specimens RDR120 and RDR130, respectively. Repaired test specimens up to 85% of the ultimate load gave the same strength carrying capacity as compared with the control specimens that had the same geometry. The ductility of an 800 mm specimen's height is greater than the other tested specimens, while the stiffness of short specimens becomes high. The control specimens (RDR120 and RDR130) and the repaired specimens gave a strength carrying capacity higher than the single layer composite specimen that tested up to ultimate load (specimen CFST100%), whereas the double-layer specimens gave more confinement to the tested specimens up to 85% by the presence of a concrete core layer and an outer steel tube. The outer steel tube gave confinement to the outer and inner concrete cores, which made an increase in concrete compressive strength by applying revised hoop stress. The ductility and stiffness of control and repaired specimens were higher than those of specimens that were tested up to failure load (specimens CFST100%) due to the presence of an outer concrete core and steel tube. Based on the methodology that was adopted for the rehabilitation of damaged columns, it was recommended to apply the same methodology for repairing the damaged composite columns because it gave a strength carrying capacity higher than the original specimens because the present study was limited to the same compressive strength as the inner and outer concrete core diameters.

6. Declarations

6.1. Author Contributions

A.N.H., and S.R.A. contributed to the design and implementation of the research, to the analysis of the results and to the writing of the manuscript. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in the article.

6.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

6.4. Conflicts of Interest

The authors declare no conflict of interest.

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