INFLUENCE OF CYCLIC LOADING ON THE BEHAVIOR OF COMPOSITE FRAME ENCASED WITH MILD STEEL

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Abstract:
Concrete-filled steel tube (CFT) has gained popularity in the building industry in recent years due to its unique static and earthquake resistance features. The interaction between steel tube and concrete provides the most benefits such as: the occurrence of local buckling of steel tube is delayed by the restraint of concrete, and the strength of concrete is boosted by the confining effect supplied by the steel tube. Hence, in the present study a mild steel-encased portal frame that employs shear connections inside the frame to increase ductility and ultimate load bearing capacity was studied. The ultimate load bearing capacity of the concrete-filled mild steel tube (CFT) frame structure was determined experimentally. During the horizontal cycle load testing of the portal structure, an axial force was applied. The material used was 1.2mm thick mild steel plates that are used in different widths. The concrete used was M25-grade. The specimens have shear connections that distribute shear, causing the two elements to work together as a unit was provided with spacing varied such as 75mm, 100mm, and 125mm, and they were evaluated at a pace of 0.1 tons per cycle. The shear connector section's strength and stiffness can be improved without the need of additional steel. All the loads versus axial elongation were investigated by drawing hysteresis curve. In comparison to standard concrete, it was observed that CFT supports had greater rigidity, ductility factor and ultimate load bearing capacity. Shear connections can greatly improve the specimen's total strength and load-carrying capacity.

Keywords: Portal frame, mild steel encased; Shear connectors; ductility; hysteresis curve

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1. Introduction

Many methods of seismic load resistance have been researched over the decades in the areas of materials, geometry, and design. Experimentation with shear studs in this category has shown a lot of potential in respect of load carrying ability and ductility. Amit H. Varma conducted an experiment to investigate the behavior of high-strength, square concrete-filled steel tube (CFT) beam-columns. Also studied were the effects of axial load level, steel tube yield stress, and width-to-thickness ratio on the stiffness, strength, and ductility of high-strength CFT beam-columns. Fiber-based models were shown to be accurate only if the assumed stress-strain and hysteresis criteria for CFT steel and concrete fibres were accurate [1].

Concrete-filled tube (CFT) columns were loaded with an axial compressive force and a bending moment. ABAQUS, a nonlinear finite element tool, was used to compare constitutive models to experimental data. There are three distinct varieties of CFT column cross section: round, square, and square with reinforcing links to increase rigidity. While a circular CFST column has a more restricting effect, a square CFST column that has been reinforced with ties is just as effective [2]. Four test models, comprising circular CFST columns and RC shear wall mixed constructions under constant axial stress and cyclic lateral load, were studied experimentally. Among of the variables examined included the amount of axial force applied on the composite column as well as the height-to-width ratio of the RC shear wall. The effect of these variables on the ductility, stiffness, and energy dissipation of the specimens was investigated. All of the test specimens broke under shear-dominant loading. As the axial load level or height-to-width ratio increased, the specimens' ductility and energy dissipation ability decreased [3-4]. The ordinary and high strength concrete enclosed by ordinary or high strength circular steel tubes. Increasing the concrete compressive strength enhances the ultimate axial loads but lowers the section and axial ductility performance of CFST columns, whereas rising the tube diameter-to-thickness ratio lowers the ultimate strengths of CFST columns and their axial ductility performance [5-6].

Alireza conducted a study in which they analyzed the seismic performance of buildings built with CFST columns and steel beams. According to the findings, frames containing CFST columns are more durable, flexible, and resilient than those with conventional RC columns. The extensive experimental and theoretical research into the behavior of hollow and concrete-filled thin-walled steel tube members when subjected to transverse impact. Both the axial and rotational movement of the members' ends were constrained. In this work, the scope of the research is broadened to include an examination of stainless steel tubes of similar nominal dimensions that were subjected to identical experimental testing. The effectiveness of each item is compared. Numerical models are then created of the steel and stainless steel tubular members, both of which are hollow and filled with concrete. It examines the effects of axial pre-load, rotational restraint at the member ends, axial restraint, metal material qualities, and concrete filling. In accordance with modern static structural steel requirements, a general design technique is created for metal tubular members with or without concrete filling subjected to transverse impact [7-8].

The response of CFST, RCC, and steel buildings to lateral loads. Concrete filled steel tubes (CFST), reinforced concrete construction (RCC), and steel construction are all studied and compared in this paper. The response spectrum analysis was applied to the 10, 20, and 30-storey models. According to the findings, the CFT structure can support a considerable weight notwithstanding the its short column cross section. According to Rui Wang, the impact properties of concrete filled steel tubular (CFST) members. It was determined that the lateral deflections of CFST members are significantly affected by the axial load when subjected to lateral impact [9-10]. Y.F. Yang conducted research into the behavior of a concrete-filled steel tube column under concentrically partial compression. The conclusion that the suggested model for forecasting the bearing capacity of partially loaded CFST columns with L/D less than 6 is generally careful to some level and relatively acceptable [11].

The seismic behavior of a composite column and a steel column in a typical multi-story building. Multi-story framed structures comprised of steel beams, RC slabs, and Concrete Filled Steel Tubes are compared for their seismic behavior in this study (CFST). Two 13-storey models were evaluated using the Equivalent Static Lateral Force Technique. Composite columns, authors found, reduce seismic base shear and storey overturning moments by 22-28% [12-14]. The statistical collapse capacity of high-rise SMRFs using CFST columns of varied width-thickness ratio was studied in the response to repeated earthquakes. It was found that the first-mode period of high-rise buildings is controlled by the entire number of floors, just like the long-period component of earthquakes. Over than 60% of high-rise SMRFs can also have their collapse margin increased using thin-walled CFST columns of equivalent flexural stiffness beneath similar earthquake conditions

[15]. The comparison between the seismic behavior of a typical multi-story building with CFRP-wrapped CFST columns and I section-encased CFST columns. Using response spectrum analysis with different types of composite columns, this paper examines the seismic behavior of tall buildings. The 13-storey and 45-storey models were analyzed using response spectrum analysis. They found that composite columns lowered storey displacement by up to 17% and drifted by up to 18% comparing to RC columns [16-17]. The most significant aspect of a reinforced concrete frame is the beam column connection. When the ground vibrates violently, it is subjected to great stress. This has an immediate impact on the structure. Shear connections must be attached to the CFT for the structure to function properly. During this investigation, the distance between shear connections was changed to a number of different configurations. Differences in the outcomes for the identical load increments were observed and evaluated in a practical setting for each specimen.

This research confirms the findings of a previous study by Sheeba Ebenezer on the ductility and ultimate load carrying capacity of a steel encased portal frame with galvanized steel that makes use of shear connectors inside the specimen [18]. Using shear connectors can greatly improve the overall strength and load bearing capacity of the specimen. One bare frame and three portal frames were subjected to a cyclic static stress test to learn more about this peculiar behavior. The shear connector spacing varies between 75mm, 100mm, and 125mm across the specimens, and they were put through their paces at a rate of 0.1 tonne per cycle. It has been shown that decreasing the space between shear connectors increases their ultimate load bearing capacity. The hysteresis curve plot provided by these experiments allowed for a comprehensive evaluation of the experimental failure process over all three portal frames. While the yield strength was raised, no pinching impact was observed in the hysteresis curve. Shear connectors at varying distances from the frame are the primary focus of this research.

2. Materials and Methods

According to IS456:2000 and IS10262:2009, the slump of the concrete was kept at 200 mm so that it could flow easily into the mould. As shear connectors, mild steel rods with a 6mm diameter and 150 mm length, a density of 7850 kg/m^3, and yield strength of 250N/mm^2 were used as shown in Table 1. These rods enhanced the frame's bending capabilities by attaching the concrete to the steel tube in a circular pattern. This was done to reduce the amount of steel used and make the specimen a thin-walled structure.

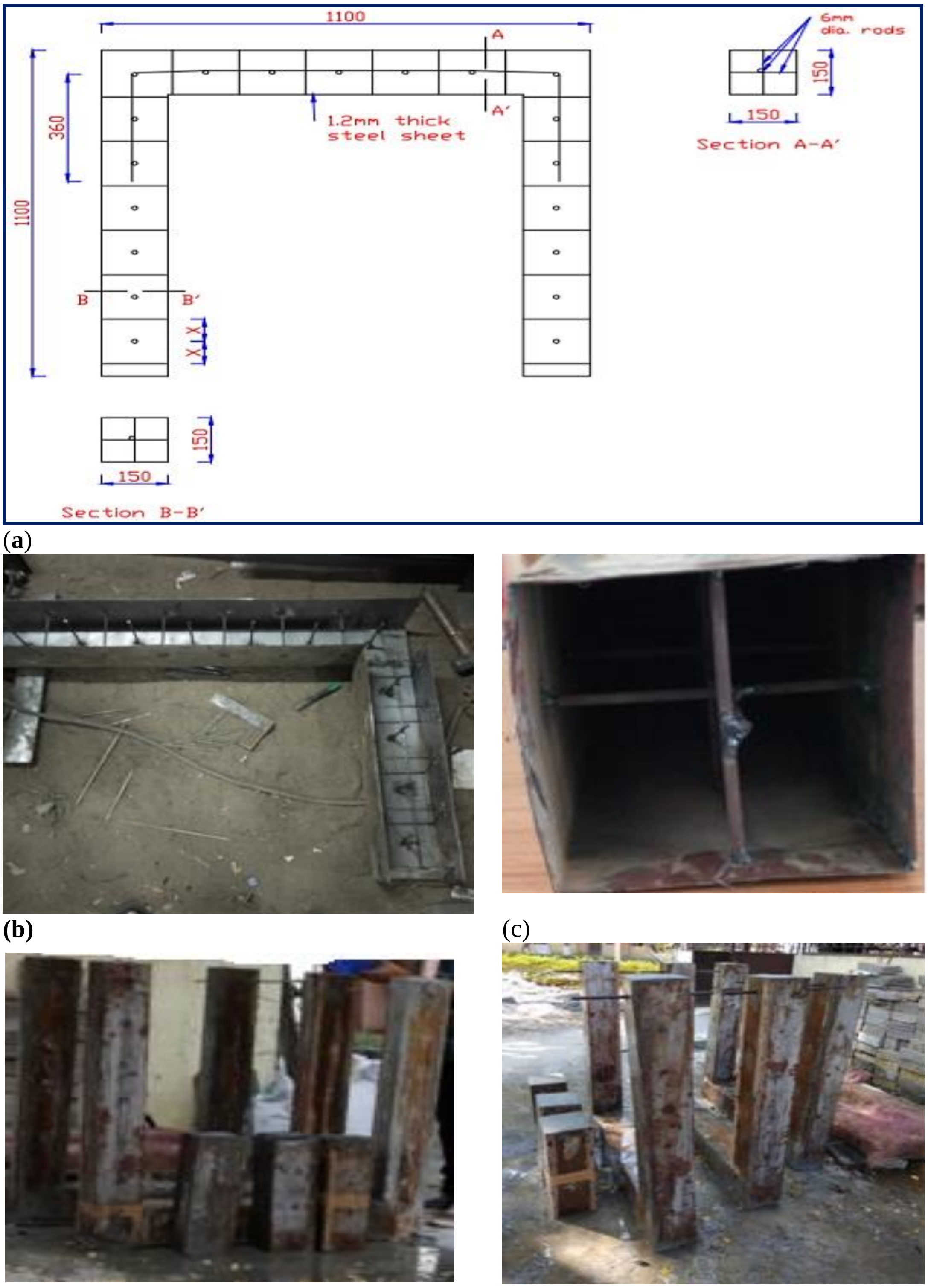
Table 1. Material Properties

Materials	Description/ values
Grade designation	M25 concrete
Type of cement	OPC 53 grade
Maximum Nominal size of aggregate-	20mm
Minimum cement content	300kg/m^3
Maximum water cement ratio	0.50
Workability	75- 100mm
Specific gravity of cement	3.15
Specific gravity of Coarse aggregate	2.74
Specific gravity of Fine aggregate	2.68 (M sand)
Workability	75- 100mm
Density (shear connectors – 6mm dia and 150 mm length)	7850kg/m^3
Yield strength (shear connectors 6mm dia and 150 mm length)	250N/mm^2
Density (mild steel – 1.2 mm thickness)	7850kg/m^3
Yield strength (mild steel – 1.2 mm thickness)	250N/mm^2

The steel casing was first bent into a C shape so that one face of the specimen was open. This was done so that it would be easier to weld the internal shear rods in place. Once the shear rods were in place, a rod was put in the beam, with the development length of it going into the columns. Then, the open face was closed by welding a steel plate to it. This steel plate was also connected to the inner shear rods by welding. The edges of the steel plate were fully welded together to make sure it was as strong as possible and to stop concrete slurry from spilling out while it was being filled. The only part of the specimen that was open was the bottom surface of the columns. This opening was used to fill the specimen with concrete and let the concrete cure. Mild steel plate with a thickness of 1.2 mm and different lengths was used. This plate held the concrete infill in place, which increased the load-carrying capacity.

3. Portal frame specimen

To study the behaviour of the frames, a set of three portal frame sections with shear connectors was filled with concrete and subjected to static cycle pressure as shown in figure 1. Each of the three frames measured 1.1 metres in height, 1.1 metres in breadth, and had a 150 mm x 150 mm rectangular surface. The empty structure was filled with grade M25 concrete, and the exposed surfaces were given a 28-day curing period.



(**a**) (**b**) (c) (d) (e)

Figure 1. Fabrication of the structures: (**a**) General Schematic Diagram of portal frame (all units in mm); (**b**) Bent mild steel sheets); (**c**) Column Bottom opening; (**d**) Concrete poured specimen; (**e**) Curing of specimen

The specimen was held upside-down while the concrete was poured, the enclosure was angled, and it was physically agitated to aid in the correct compaction of the concrete, along with routine tamping with a 16-mm rod. After the specimens were made, the concrete's exposed sides were normally hardened up until the 28th day to guarantee strength during this time.

The spacing and number of shear connectors in the portal frame were chosen to be studied. Other parameters, such as beam size, beam width, column height, column size, shell thickness, steel rod section, and concrete quality, stayed the same throughout the research. The names of the portal frames were SP-75, SP-100, and SP-125. The distance between the shear connectors on the SP-75 was 75mm, on the SP-100 it was 100mm, and on the SP-125 it was 125mm. These shear connectors were welded inside the frame section. The loading frame was designed using IS-800:2007, and two mild steel base plates were made according to the design.

Table 2. Specimen names

Specimen ID	Spacing of Studs	No. of Specimens
Bare frame	-	1
SP-75	75mm	1
SP-100	100mm	1
SP-125	25mm	1

Figure 2. Base plate.

To allow for any possible mistakes that the prototype may have had, these base plates had 20 mm of space in both directions. Additionally, 16mm-diameter aperture 50mm from the base were drilled so that they lined up with the perforations in the supports. To allow for the space for the specimen's perforations, the apertures on the "cup" had to be enlarged to a circumference of 31mm as in Figure.2. This allowed for a better and easier setting of the lateral bolts.

3. Experimental Test Setup

The experiment was set up in such a manner shown in Figure.3 that the displacement could be detected on both the upper surface of the specimen column as well as the centre surface of the specimen column.

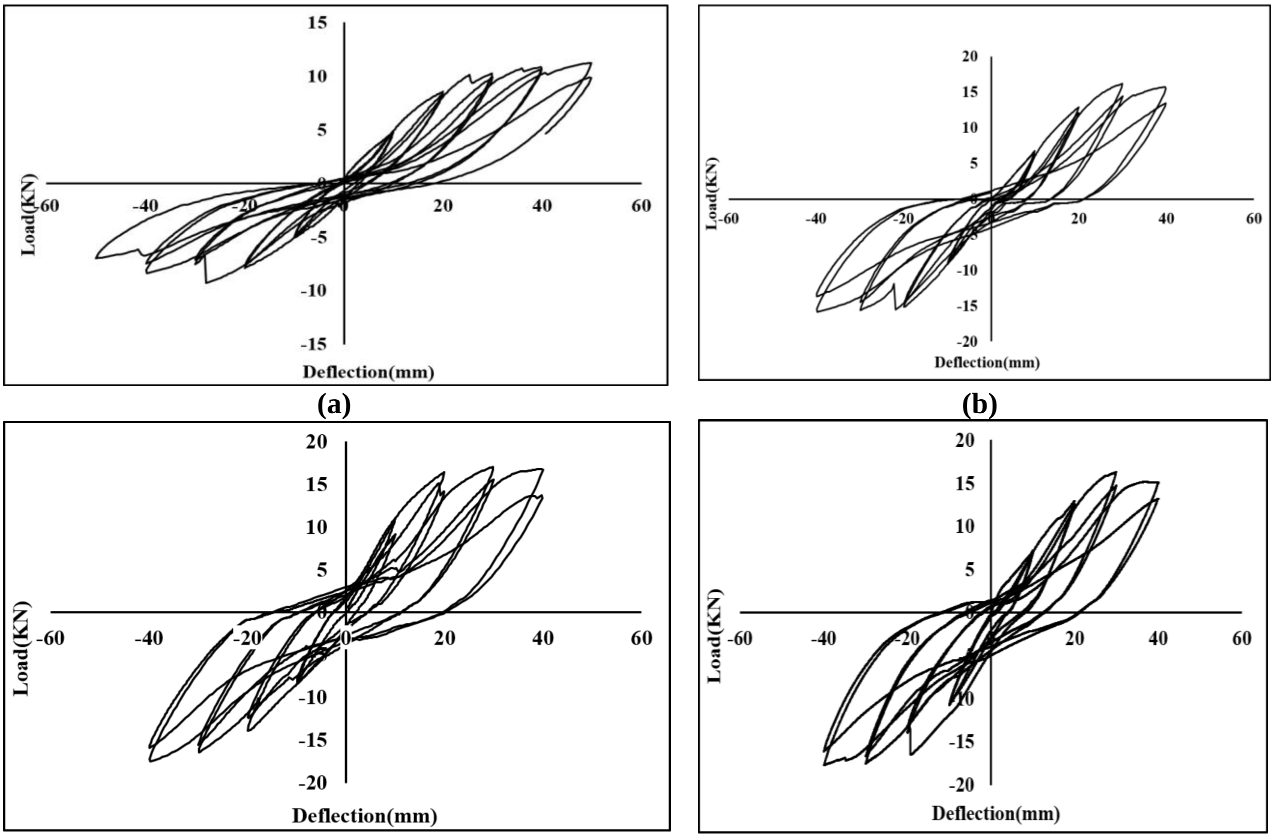
Figure 3. Test setup of composite portal frame.

All of the samples were put through tests on a 50T cyclic loading frame to see how they behaved under static cyclic loading. A hydraulic jack with a load cell was used to put the load on the beam-column joint and measure the deflection. At a rate of about 0.1 tonne per cycle, the load was put on. With a loading gap of 1 kN, each loading was done three times, for a total of 10 loading cycles per specimen. Two LVDTs were put on the sample at the same time. One was put in the middle of the top edge (LVDT 1), and the other was put in the middle of the column (LVDT 2). These were used to keep track deflections for each load. Only the readings from LVDT-1 were used, and LVDT-2 was only used as a control. Three samples and a "bare frame" sample were cast and then tested.

The way the loads were given was static, since the loads were not given based on time intervals. The test was done with the load controlled, which means that the deflection was observed for a given load. Each specimen was loaded at a rate of +/-0.1 tonnes per cycle, and the loading frame could hold up to 50 tonnes of weight. At the start of the experiment, all the connections were made, and the sample was attached to the base plate. This stopped any movement upward.

3. Results and Discussion

The hysteresis curve for the bare frame, SP-75, SP-100, and SP-125 can be seen in figure 4. (a) to (d). The horizontal load displacement hysteresis graphs for all of the specimens are shown here. At first, the hysteresis curves were straight line, which shows how the sample curve looks after it has been changed by elasticity. This was the case from the very beginning. After being displayed to the regions of this loop, a progressive however, some increase occurs, producing a straight formation; lingering deformations can be seen after the frame has been unloaded. When the highest value is reached, the increased loads drop by a significant amount while the horizontal displacements reach a high point, indicating that the material has a high degree of ductility. In the end, when the hysteresis curves are smoothed out, it is determined that all of the specimens had satisfactory performance and that there was no significant squeezing.



(c) (d)

Figure 4. Hysteresis curve: (**a**) Bare Frame (**b**) SP-75; (**c**) SP-100; (**d**) SP-125

Based on the data presented in Table 3 given below, it can be seen that the load-carrying capabilities of the three specimens, including SP-75, SP-100, and SP-125, are greater than those of the bare frame. In a nutshell, the one that had the highest maximal measured load of 16 kilos was the SP-75 when compared to the bare frame. SP-75 has increased its overall effectiveness by 38.86

percent. In a similar manner, the respective values for SP-100 and SP-125 are 29.05% and 20.11%. Figure 5a depicts the highest load that was measured along with the proportional increase in load compared to the bare frame. Additionally, the highest measured displacement is displayed in figure 5b, along with the percentage decrease. The amount of energy that is wasted by the samples is the primary criterion that is used in the process of determining the earthquake behavior, which is defined as an abrupt movement. Because plastic hinges have been developed, specimens SP-100 and SP-125 have an increased potential for wasting energy after they have yielded. This is a direct result of the plastic hinges. The capabilities for dissipating energy of both the bare frame and the SP-75 were shown to be quite low in Table 3 and figure 6.

Table 3. Load-carrying capabilities of the specimens

Assigned Name	Spacing of studs	Maximum Load (kN)	Load increase in % compared to bare frame	Maximum observed Deflection (mm)	Deflection decreases in % compared to bare frame
Bare frame		11	-	46.67	-
SP-75	75mm	16	38.86	39.06	25.08
SP-100	100mm	15	29.05	30.03	42
SP-125	125mm	14.3	20.11	39.81	18.15

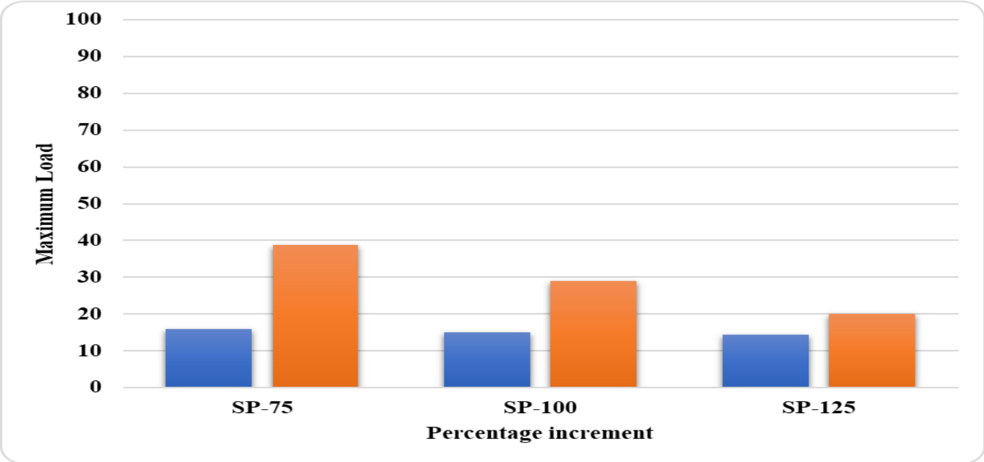


Figure 5. a) Maximum load and percentage increment

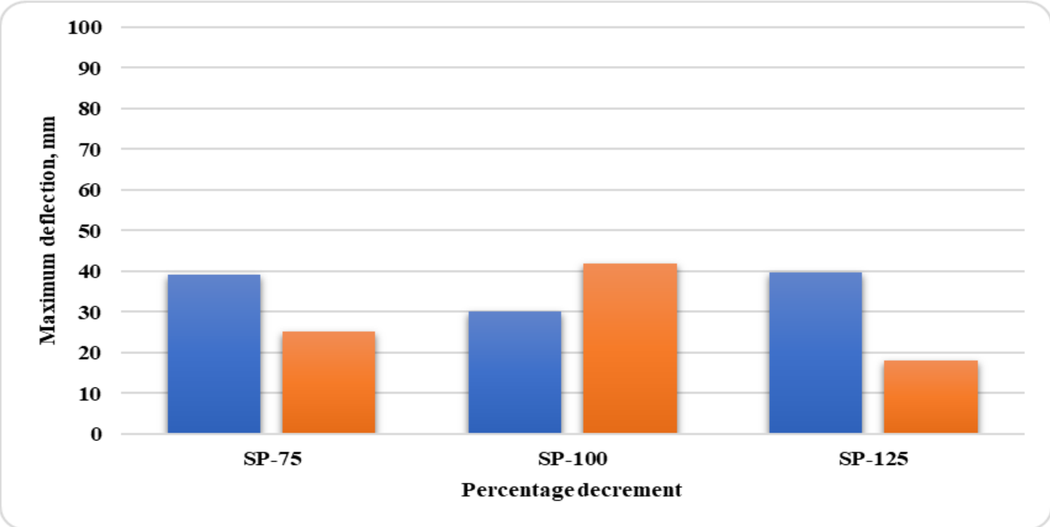


Figure 5. b) Maximum Deflection and percentage decrement

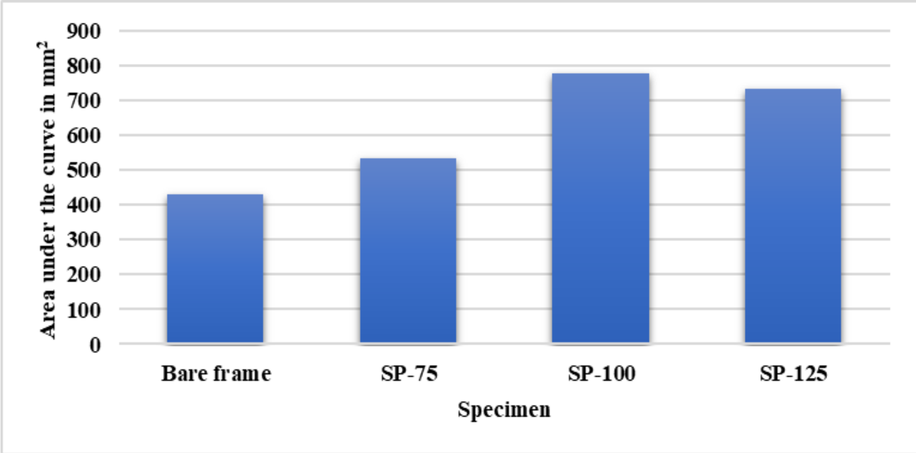


Figure 6. Area under the Curve of four specimens

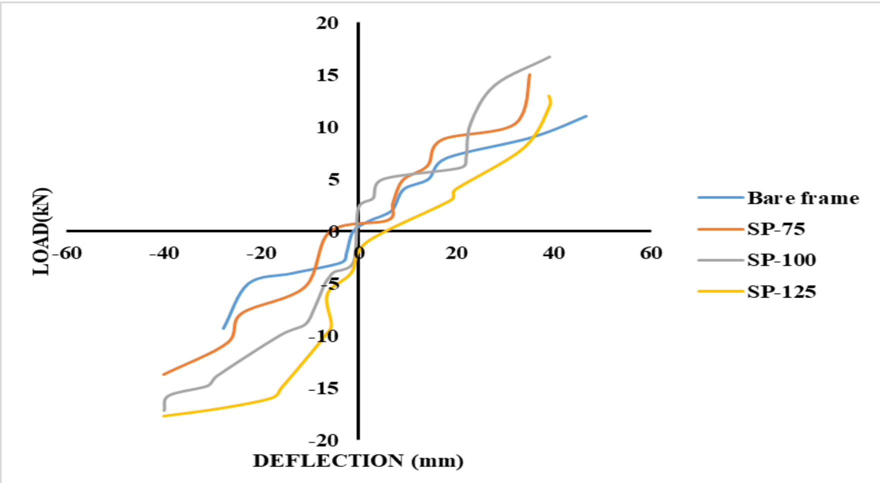


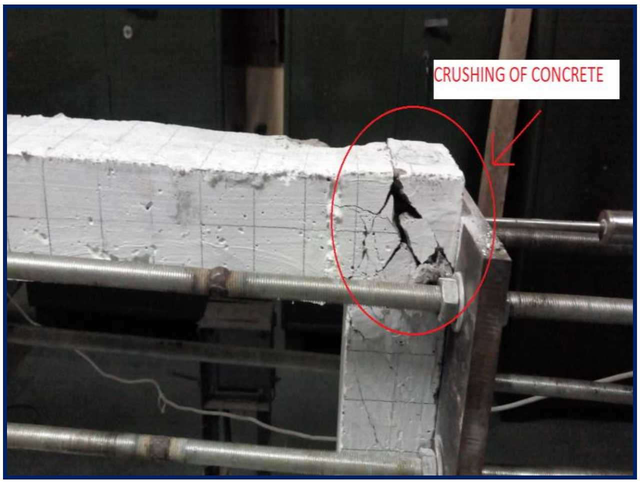
Figure 7. Comparison of backbone curves of bare frame, SP-75, SP-100 and SP-125

Backbone curves show how important horizontal load and ductility should be. It shows that both the ductility and the transverse loads have gone up since the last time. Concerning the negative loads, we can see in Fig. 7 that the area under each curve grows in increasing order. This demonstrates that the fact that the shear connection separation should be decreased results in an increase in the ductility of the specimen. As a result, an increase in the distance between the shear connectors causes the material to become more fragile. It is possible to draw the conclusion that the shear connection separation of 75 mm takes on more load and experiences significantly more displacement. Because of this, it is abundantly obvious that the eventual load carrying capacity rises in direct proportion to the degree to which the distance between shear connections is reduced.

4. Failure mode in deflection

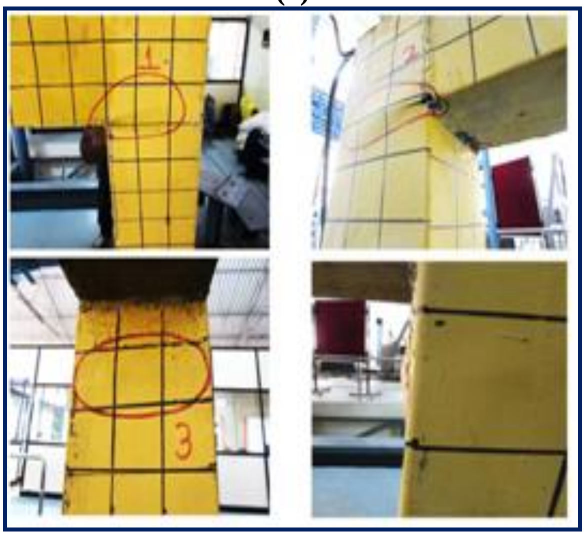
The surface of the specimen's bare frame has concrete that has been crushed, as shown in Figure 8a. As can be seen in Figure 8a, the majority of the cracking in the concrete is occurring in one particular location on the specimen. The location of fractures that can be seen at the junction between the beam and the column for SP-75 is shown in Figure 8b. After the test, the beam-column junction moved into the region depicted inside the red circle in Figure 8c for SP-100. This is the location where the slide occurred. A failure can be seen developing in the specimen SP-125 shown in Figure 8d, just below the junction that connects the beam and the column. Following the completion of all of the tests, an opening was made in the steel conduit so that the face of the inner concrete could be inspected.

The concrete has experienced a great deal of tensile failure, but other than that, it does not appear to have sustained any significant damage. In the past majority of cases, the shear connections located inside a CFT do not work properly because either the concrete or the shear connector does not work properly. It is simple to see that when the negative loads are applied, the area under each curve increases, which demonstrates that the material is more malleable when the shear link separation is reduced. This was proven by the fact that the area under each curve increased. Therefore, the distance between the shear connections increases, the object becomes more vulnerable as a result of this.

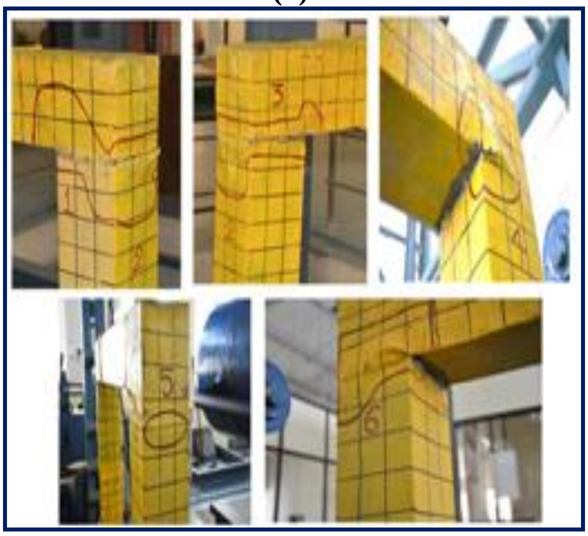


(a)

(b)



(c)



(d)

Figure 8. Specimen failures: (**a**)Bare Frame (**b**) SP-75; (**c**) SP-100; (**d**) SP-125

5. Conclusions

As the spacing between shear connectors reduces, the ultimate load bearing capacity improves owing to the increased confinement effect, and the improvement in ductility is evidenced by the presence of a broad plateau in the load deflection curves obtained for specimens with varying spacing of shear studs. Load cells and LVDTs showed that the part still held load after the crushed concrete damaged its internal structure. The encasing keeps the concrete from falling apart, which can make it safer for people to leave. Some experts think that the steel's ability to hold things together improves its ability to hold load in the end. From this, we can figure out that a 75mm shear connection gap takes a lot more load and, as a result, a lot more movement. To improve the load bearing capacity, the space between shear links needs to be shortened. The big peak in the load displacement curves for the cases with different spacing between the shear studs shows that ductility increases as the spacing between the shear connections gets smaller. Beam column joints should be kept free of steel welds.

The analysis and comparison will be performed subsequently utilizing computer software. It is clear from reviewing the existing researches that the CFST (Concrete Filled Steel Tube) frame structure's behavior requires further study. Thus, this study investigated the utilization of CFST (Concrete Filled Steel Tube) frames to achieve both seismic resistance and cost-effective building.

Supplementary Materials: The following supporting information can be downloaded at: www.mdpi.com/xxx/s1, Figure S1: title; Table S1: title; Video S1: title.

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