INVESTIGATION AND FEM ANALYSIS ON REINFORCED CONCRETE COLUMNS STRENGTHENED WITH CFRP AND STEEL PLATES UNDER AXIAL COMPRESSION

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Abstract

FRP strengthening is an effective solution for the repair and strengthening of RC columns. Therefore, researchers in the past investigated the performance of the externally bonded (EB) FRP-confined columns. However, the data is only limited to columns of smaller size. Furthermore, the confinement with the FRP is less effective for columns having aspect ratios greater than two. Hence, this study evaluates different strengthening configurations for columns with higher aspect ratios. A total of six RC columns of size 150 mm x 450 mm and 1500 mm in length are cast, strengthened, and tested using four strengthening configurations under axial compression. The chosen strengthening configurations include combinations of externally bonded-CFRP fabric, near-surface mounting with CFRP laminates, and steel plate anchored with bolts. The influence of these strengthening configurations on their axial capacity and ductility is experimentally and numerical evaluated. Results demonstrate that the hybrid strengthening configurations performed better than the others for the considered aspect ratio of three.

Keywords: RC columns, aspect ratio, axial compression, EB confinement, NSM, Steel plates, Hybrid strengthening.

Introduction

Reinforced concrete structures are widely used in the construction industry due to the easy availability of raw materials and low construction costs. Among all the members in a structure, columns are the most critical members. Generally, column provides vertical support and stability to the structure and helps transfer load from the superstructure to the substructure. Moreover, the failure of a column in a structure leads to the complete collapse of the structure. Hence, it is necessary to comprehend the damage behavior and suggest possible strengthening methods.

There are several situations where columns get damaged, and strengthening is employed, such as damage of columns due to corrosion of rebar, earthquakes, and fire accidents, as shown in Figure 1.1 – 13. Similarly, in the case of an increase in load demand, a change in functionality of the structure and upgradation of design codes require the strengthening of the column. As a result, researchers studied the possibilities of different strengthening methods.

Conventional Method of Strengthening

Strengthening is a process by which it enhances the load-bearing capacity and ductility of a structural member. There are various methods available for strengthening a structural member. Moreover, methods such as reinforced concrete jacketing and steel jacketing have been used in the past years for strengthening, as shown in Figure 1.4 – 1.5. However, these methods of strengthening have become obsolete due to the significant addition of dead load to the structure, In addition, requires more execution time and manpower.

Literature Review

(**Alsayed et al., 2014**) studied the effect of shape modification on a wall-type rectangular RC column strengthened with CFRP wrap under concentric compression. In this experimental study, RC columns with an aspect ratio of four were converted into an elliptical section called "shape modification." Moreover, this shape modification increases its cross-section by 17.6%. Further, the researchers observed that the effect of the shape modification of the column strengthened with CFRP wrapping played a significant role in enhancing the ultimate strength and ductile behavior as shown in Figure 2.2. Also, they concluded that this method of strengthening improved ultimate strength by 42% and ductile behavior by six times more than the control.

(**Triantafillou et al., 2016)** investigated the effect of different parameters on the FRP confinement of wall-type RC columns under concentric compression. The parameters include aspect ratio, anchor types (light and heavy), number of FRP layers, and section enlargement. In this study, they considered two different sizes of columns: 150 mm x 450 mm and 150 mm x 600 mm, with a height of 800 mm. The researchers concluded that using two heavy anchors on the larger side of the cross-section for the aspect ratio of four shows maximum confinement compared to light anchors. In addition, adding an extra CFRP layer improved the ductile performance of the column. Whereas, in columns with an aspect ratio of three, one heavy anchor was sufficient on the larger side of the cross-section to enhance the ultimate capacity and ductility.

(**Wang et al., 2016**) investigated the size effect on axial stress-strain behavior of plain and RC square columns strengthened with different numbers of layers of CFRP under concentric compression. In this study, they considered different square cross sections of columns having the same height-to-width ratio of three, with sizes of columns of 100 mm, 150 mm, 175 mm, 200 mm, 300 mm, 350 mm, and 400 mm. In this experimental study, they observed that FRP-strengthened columns with a cross-section up to 300 mm did not affect the axial stress-strain behavior compared with the cylinder strength of concrete. However, column cross-sections exceeding 350 mm showed a significant difference with respect to the cylinder strength of concrete. Moreover, the effectiveness of FRP confinement decreased with an increase in the cross-section of the columns.

(**Zeng et al., 2018**) did an experimental study on the behavior of large-scale RC columns confined with CFRP wrapping under axial compression. This experimental study examined the effect of the corner radius and the thickness of CFRP. They concluded that with an increase in corner radius of the columns or number of layers of CFRP, there is a significant increase in ultimate compressive strength and axial strain as shown in Figure 2.3.

Objectives

The objectives of the study are to

- Evaluate the strength and ductility of a wall-type column of aspect ratio threeusing different strengthening methods under axial compression.
- Understand and compare the confinement effect of the steel platestrengthening with the CFRP strengthening.
- Validation of the

experimental results with finite element analysis using ABAQUS.

Experimental Program

Test Specimens

A total of six reinforced concrete columns were cast with a cross section of 450 mm x 150 mm and a height of 1500 mm. These columns had six bars of 12 mm diameter used as longitudinal reinforcement, and 8 mm diameter were used as tie bars, with a centre-to-centre spacing of 75 mm at the top and bottom of the column up to 300 mm and 200 mm spacing in the remaining zone, as shown in figure 3.10. The percentage of longitudinal reinforcement in the column is 1%, the volumetric percentage of transverse reinforcement in the column is 0.67%. Out of six columns, specimens that were tested under concentric axial compression consisted of control, near-surface mounting (NSM) with CFRP laminates, externally bonded (EB) CFRP wrapping, a hybrid of NSM and EB, Steel plates anchored with bolts and hybrid of EB and steel plates anchored with bolts. All these cross-sections of different strengthening configurations are shown in Table 2.

Procedure for Strengthening of RC Columns

Externally Wrapping (EW) Strengthening

The EW strengthening of the RC columns was followed the ACI 440.2R guidelines. Firstly, the corners of the columns were ground up to 25 mm to prevent premature failure of FRP due to stress concentration at the corners of the columns. The surfaces of the columns were cleaned with water. After drying, primer was applied to the column surfaces. After 24 hours of curing, an epoxy mix was prepared (base: hardener = 4:1) and applied to the surface columns. Then, CFRP fabric was wrapped around the column, ensuring the fibre orientation must be 90 degrees, i.e., perpendicular to the applied load, for better confinement, and then another layer of epoxy was appliedover the fabric. Moreover, a 50 mm overlap was maintained between two layers in thelongitudinal direction, and a 100 mm overlap was maintained in the lateral direction to prevent debonding failure (Chellapandian et al., 2017).

Furthermore, during this process of strengthening, no air gap should be present. Thenthe columns were left to cure for at least 48 hours at room temperature. Some of the steps of strengthening.

NSM Laminate (NSM_L) Strengthening

The NSM laminate strengthening of the RC columns was as per ACI 440.2R guidelines. Grooves were cut more than 1.5 times the laminate dimension. A total of eight grooves were cut with dimensions of 21 x 23 mm on the larger face of the column. Moreover, a minimum edge distance between laminate and concrete was maintained to ensure no premature debonding failure happened (Chen et al., 2013). Further, all grooves of the column were cleaned with pressurised water to ensure a dust-free surface. After drying the concrete surface, primer was applied, and it was cured for 24 hours. Further laminate epoxy was prepared (base: hardener = 2.61:1), applied in the groove with C-laminate, and left to cure for at least 48 hours at room temperature. Some of the steps of strengthening are shown in figure 3.20 – 3.24.

Hybrid Strengthening with EW and NSM_L

Hybrid strengthening of the RC columns was as per ACI 440.2R guidelines. Firstly, the column was strengthened with NSM as per Section 3.4.2; further, the NSM- strengthened column was strengthened with EB as per Section 3.4.2

Near Surface Mounted Steel Plate with Anchor Bolts Strengthening

Columns strengthened with steel plates and anchored with bolts were done as follows: Firstly, six grooves were cut with each dimension of 50 x 15 mm, followed by the drilling of concrete up to 100 mm. Further, clean the grooves and drilled holes with pressurised water, and then, after drying, apply primer to the grooved concrete surface only. After 24 hours of curing the primer, apply bolt adhesive chemical (FIS EM Plus 585S) in the grooves, then put the bolt in by rotating to ensure a good bond between the concrete and the bolt. Further,apply a layer of laminate epoxy on the surface of

the grooved concrete and steel plate, then put the steel plate in the groove, then press the steel plates with some mechanical device so that the extra epoxy is removed and left to cure for at least 48 hours at room temperature. Moreover, after curing, tighten the bolts with a mechanical wrench and cut the extra portion of threaded rods with a metal cutter. Some of the steps of strengthening are shown

Hybrid of EB and NSM_SP_AB.

Hybrid strengthening of columns with externally wrapped and near surface mounted steel plate with anchor bolts (NSM_SP_AB) was done as follows: Firstly, cut all six grooves with dimensions of 50 x 15 mm, then do corner grinding of the columns. Moreover, clean the dust with pressurised water. For further strengthening work, follow Section 3.4.4 and then 3.4.1 as shown in figure 3.32 – 3.34.

Experimental Results and Discussions

The experimental results of various strengthening configurations are presented in this chapter. The failure load refers to a 30% of the peak load to capture the post-peak behaviour of the specimens. All the deformations were measured with a common gauge length of 250 mm.

Control RC Column

The load vs. axial deformation graph of the RC control column is shown in figure 4.1. The column reached maximum load of 1879 kN at an axial deformation of 0.41 mm. The specimen failed at a load of 1319 kN with an axial deformation of 0.47 mm.

Near Surface Mounted with Laminate (NSM_L)

The load vs. axial deformation graph of the column strengthened with NSM_L is shown in figure 4.2. The column reached its maximum load of 2002 kN at an axial deformation of 0.46 mm. The specimen failed at a load of 1402 kN with an axial deformation of 0.55 mm. However, a marginal strength improvement of 6.5% and ultimate deformation of 17.5% were observed compared to the control column.

Externally Wrapped (EW)

The load vs. axial deformation graph of the column strengthened with EW is shown in figure 4.3. The column reached its maximum load of 2282 kN at an axial deformation of 0.66 mm. The specimen failed at a load of 1498 kN with an axial deformation of 0.73 mm. Moreover, a significant strength improvement of 21.4% andultimate deformation of 55.6% were observed compared to the control column.

Hybrid Section (NSM_L + EW)

The hybrid-strengthened column consists of near-surface-mounted laminate with externally wrapped. The load vs. axial deformation graph of this strengthened column is shown in figure 4.4. The column reached its maximum load of 2488 kN at an axialdeformation of 0.93 mm. The specimen failed at a load of 1742 kN with an axial deformation of 1.04 mm. Moreover, a significant strength improvement of 32.4% and an ultimate deformation of 121% were observed compared to the control column.

Near Surface Mounted Steel Plate with Anchor Bolts (NSM_SP_AB)

The load vs. axial deformation graph of this strengthened column is shown in figure

4.4. The column reached its maximum load of 2480 kN at an axial deformation of

0.34 mm. The specimen failed at a load of 1818 kN with an axial deformation of 0.49 mm. Moreover, a significant strength improvement of 32% with only a marginal improvement of ultimate deformation of 5% were observed compared to the control column.

Hybrid combination of EW + NSM_SP_AB

This is a hybrid column strengthened externally wrapped with near-surface mounted steel plate with anchor bolts. The load vs. axial deformation graph of this strengthened configuration is shown in figure 4.6. The column reached its maximum load of 2457 kN at an axial deformation of 0.4 mm. The specimen failed at a load of 1731 kN with an axial deformation of 0.82 mm. Moreover, a significant strength improvement of 30.7% with only a marginal improvement of ultimate deformation of 76% were observed compared to the control column.

Comparison of the overall behaviour

The overall comparison of load vs. axial deformation of control and different strengthened columns is shown in figure 4.7. Moreover, from the graph, it is clearly observed that the NSM_L strengthened column showed only a 6.54% increase in peakload with respect to the control column. However, 32.4% of strength increment was observed in the NSM_L + EW strengthened column. The stiffness of the steel plate- strengthened columns with and without EW is higher than the CFRP-strengthened columns. Further, it was observed that the peak load and axial deformation at failure load were maximum for NSM_L and EW-strengthened columns.

S.No	Column Type	Peak Load (kN)	Percentage Increase in Peak Load*	Axial Deformation at Peak Load (mm)	Axial Deformation at Failure load (mm)
1	Control	1879	---	0.41	0.47
2	NSM_L	2002	6.54%	0.46	0.55
3	EW	2282	21.40%	0.66	0.73
4	NSM_L+EW	2488	32.40%	0.93	1.04
5	NSM_S_AB	2480	31.96%	0.34	0.49
6	NSM_S_AB + EW	2457	30.74%	0.40	0.82

***Note:** Percentage increase in peak load is calculated with respect to control RC column

Comparison Load vs. Strain Graph

The comparison of load vs. strain of control and different strengthened configurations is shown in figures 4.8 and 4.9. Due to the carbon laminate, strain in the column strengthened with NSM_L was lower in comparison with the control column. In the EW-strengthened column, at a particular load value, strain in the longitudinal reinforcement was less compared with NSM_L and control. This is because the EW-strengthened column provides lateral confinement to the column, which delays the buckling of the main reinforcement. Moreover, in the case of EW+NSM_L, the strengthen column gave a better result compared with the control, NSM_L, and EW+NSM_L. However, in steel plate strengthening, a significant decrease in strain was observed in the longitudinal and tie rebar.

The strain in CFRP and steel plate is plotted as shown in the figure 4.10. Hybrid (NSM_L+EW) strengthened column shows better utilisation of CFRP material than NSM_L strengthened column as it achieves a higher strain value before failure and, as a result, provide better ductility. However, for steel plate, strengthened columns show lesser strain at a given load in comparison with other strengthened methods.

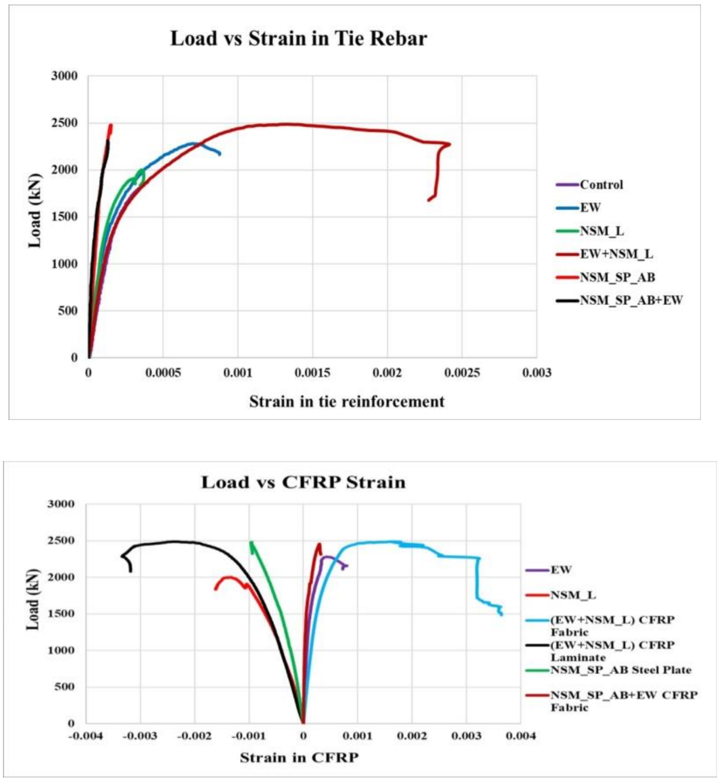

Figure 4-10 Load vs. Strain behaviour of CFRP and steel plate

Finite Element Study

A finite element study was carried out to validate the load vs. axial deformation and failure pattern of all strengthened columns under axial load. 3D nonlinear analysis was carried out using commercially available software, ABAQUS 2023.

Material Parameters

Concrete is a heterogeneous material that exhibits nonlinear behaviour. Moreover, toaccurately replicate the behaviour of concrete in finite element (FE) analysis, a concrete damage plasticity model was used. Further, different parameters have been considered for the CDP model, such as dilation angle (ψ), eccentricity (ε), the ratio ofbiaxial yield stress to uniaxial yield stress (f_{bo}/f_{co}), viscosity parameter (μ) and the coefficient to obtain the shape of the deviatory cross-section (K) for control and strengthened columns (Balla et al., 2023). The dilatation angle in concrete is used to describe the angle of lateral expansion or contraction that happens when concrete is subjected to axial force. In a concrete column, unconfined concrete normally has a dilatation angle of about 35 degrees. However, the dilatation angle reduces from 35 to 1 degree (Balla et al., 2023) when concrete is confined, and effective lateral stress increases as a result, blocking cracks in the concrete (Jamatia & Deb, 2017).

The Hognestad compression model was used to model the compressive stress in the concrete. Concrete damage behaviour is a function of plastic strain and field variables. Compression damage was considered after the concrete reached peak load calculatedas per Equations 5.1 – 5.6. The Belarbi and Hsu model was used for the tensile stress- strain graph, as shown in figures 5.1 and 5.2. Moreover, for steel rebar and steel plate,elastoplastic nature had been used with yield strength of 500 MPa and ultimate strength of 612 MPa. For incorporating damage behaviour in externally wrapped carbon fabric, carbon Hashin damage was used. It consists of damage evolution and damage stabilization. Further, laminate had been considered elastic, having a yield strength of 2945 MPa, and all other properties had been modelled as steel reinforcement shown

Validation of FE model

In order to validate the FE model, the model, initially, validated with existing experimental results with different strengthening configurations, such as control RC columns, columns strengthened with externally bonded, carbon laminate, and hybrid combinations of EB and carbon laminate from available literature (Malleswara Rao & Prakash, 2021). Details of modelling, such as material parameters, modelling details, and boundary conditions, had been discussed in sections 5.2 - 5.4. Further, the summary of load vs. axial deformation graphs had been plotted and validated with experimental data, as shown

Failure mode of externally wrapped strengthened column

Column strengthened with the externally wrapped CFRP column failed by rupture ofthe CFRP wrap in the experiment, whereas similar behaviour was also observed in the FE model as concrete damaged in the centre portion, which led to failure of the CFRPfabric

Failure mode of near-surface-mounted laminate strengthened column

The column strengthened with the near-surface-mounted laminate (NSM_L) failed similarly to the control column, The failure was initiated by the longitudinal bar buckling, followed by the crushing and spalling of concrete. Moreover, in FE model also concrete failed in the central portion of the column as shown

Failure mode of hybrid section (EW+NSM_L)

The hybrid column (NSM_L + EW), the failure was initiated by the buckling of CFRP laminates and crushing of concrete at the mid location. Moreover, in the case of the FE model, concrete prolonged the failure of the column, and as a result, more ductile behaviour was observed in comparison with experimental data as shown

Failure mode of NSM_SP_AB

The failure pattern for steel plate strengthened columns differed from that for CFRP-strengthened columns. Initially, failure was initiated at the end. Further, concrete crushing was observed around the plates, along with the bending of the plates. For the FE model, failure started from the ends, similar to the experiment failure pattern as shown in Figures 5.30 – 5.32.

Failure mode of hybrid (EW + NSM_SP_AB)

For hybrid column (NSM_S_AB + EW), the failure was initiated by the bending of plates at the ends and followed by the rupture of the CFRP wrap. However, this type of failure does not happen in the FE model as embedded constraints have been provided, as shown in Figure 5.33 – 5.35.

Conclusions

The present study focuses on assessing the effectiveness of various strengthening configurations for wall-type columns with an aspect ratio of three in terms of their strength and ductility. Additionally, all the experimental results have been validated using a finite element model in ABAQUS. The conclusions from this study are listedbelow.

- Confinement through external wrapping of the columns enhances the axial deformation at the peak load. Implicitly, it also improves the ductility of the columns. Moreover, it prevents the buckling of the longitudinal bar, thereby improving the column's load-carrying capacity.
- In the case of wall-type columns, the NSM_L + EW strengthening configuration performed the best in improving strength and ductility compared to other CFRP strengthening methods.
- The stiffness of steel plate-strengthened columns is higher than that of CFRP-strengthened columns. However, quality control shall be maintained to avoid delamination of steel plates or pull-out of the anchor bolts, resulting in reducedstiffness.
- NSM_SP_AB and

NSM_SP_AB+EW columns reached almost the same strength as NSM_L + EW columns. Therefore, steel plate strengthening can replace CFRP strengthening for wall-type columns for better fire resistance. However, steel plate strengthening is more time-consuming and involves more laborious work than CFRP strengthening.

Scope for Future Study

- Further study is required on columns strengthened with NSM_S_AB and NSM_S_AB+EW using counter sunk bolts.
- In FE analysis, up to peak load, there is a better response in comparison with experimental results. Proper epoxy should be modelled for better prediction of FE results in the post peak region.

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