BEHAVIOUR AND STRENGTH OF STEEL JACKETED RC COLUMNS UNDER CONCENTRIC AXIAL LOAD

FARJANA AKTER

MASTER OF SCIENCE IN CIVIL ENGINEERING (STRUCTURAL)

DEPARTMENT OF CIVIL ENGINEERING
BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY
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Submitted by-

FARJANA AKTER

ID No. 0413042302F

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The thesis titled **"Behaviour and Strength of Steel Jacketed RC Columns Under Concentric Axial Load",** submitted by Farjana Akter, Student Number 0413042302F Session April, 2013 has been accepted as satisfactory in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Structural) on 10 June, 2018.

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Dedicated

To
My Beloved Parents

ACKNOWLEDGEMENT

The Author sincerely expressed her deepest gratitude to the Almighty.

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ABSTRACT

Reinforced concrete (RC) columns often need strengthening to improve their load carrying capacity and ductility to sustain the applied loads. This research investigates the behavior and strength of the RC columns strengthened with steel jacketing method.

An experimental investigation was carried out on deficient RC columns with steel angles and strip jacketed under concentric axial loads. The experimental program consists of eight square deficient RC columns with similar longitudinal steel ratio and low concrete strength. Mimic to the practical situation, they were made deficient further by preloading up to 70% of their ultimate load. The failure behavior, post-peak responses, and ultimate loads were observed during testing. The effects of the parameters: steel angle ratio, spacing of the horizontal strips, preloading state and partial strengthening method applied at column ends were investigated from the experimental results. Finally, the experimentally obtained capacities for the preloaded strengthened columns were compared with theoretical capacities calculated using available analytical models.

The performance of the particular steel jacketing method was found to be very efficient. Significant improvement in ultimate load and ductility was obtained in the study. The failure in most of the strengthened specimens was due to the buckling of the steel angle followed by crushing of the concrete. From the test results, it has been found that the strengthened columns improve their load carrying capacity (ranging from 87% to 186%) and ductility (ranging from 31% to 67%) than the unstrengthen RC columns. This enhancement in ultimate capacity was observed to be reduced by 27% due to the application of 70% preloading to the RC columns before strengthening. The increase in steel angle ratio from 4% to 7% provided highest contribution in improving the ultimate strength of the preloaded column with respect to the unstrengthen column. On the other hand, decreasing the strip spacing from 300 (2d) to 150 (d) mm resulted in an increase in the capacity of the preloaded columns by an average value of 15% only. However, the ductility of the columns were improved significantly by about 60% due to the reduction in strip spacing. The available capacity prediction models for unloaded strengthened RC columns can be safely applied for preloaded strengthened RC columns.

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NOTATIONS

f_c, f_{co}	Compressive strength of the unconfined concrete
f_c'	28 day cylinder crushing strength of concrete
f_y	Yield stress of longitudinal steel bar
b	Smaller side dimension of RC column
d	Larger side dimension of RC column
L	Length of the column
L_1	Leg length of steel angle
S	Spacing of the strips
t_1	Thickness of steel angle
S_2	width of strip
A_c	Cross sectional area of concrete
A_j	Cross sectional area of steel jacket
f_{jy}, f_{yL}	Yield strength of steel jacket
f_l	Confinement pressure
N_L	Axial force carried by angles
E_L	Elastic modulus of caging steel
Ec	Elastic modulus of concrete
f_{cc}	Compressive strength of confined concrete
n_a	Maximum axial force in angles
N_c	Axial load carried by concrete
M_p	Plastic moment of the angles depends on the plastic axial loads
μ	Displacement ductility index
$\Delta\mu$	% Increase in displacement ductility w. r. to the unstrengthened column
∇_u	Displacement at ultimate load
$\Delta\nabla_u$	% Increase in displacement at ultimate load w. r. to the unstrengthened column
P_u	Ultimate load
ΔP_u	% Increase in ultimate load w. r. to the unstrengthened column
P_n	Ultimate strength of unstrengthened column

CHAPTER 1
INTRODUCTION

1.1 General

Reinforced concrete (RC) columns are the primary load-bearing structural elements in a typical building structure. Basically, they are designed to carry out required level of structural performance throughout of its life-time. But over the lifespan, they may go through damage or degradation due to the human activity or natural actions resulting from corrosion of steel bars or cracking or spalling of concrete. If other structural element such as beam or slab in a building is damaged, this will affect only one floor, but damage to a RC column could bring down the total collapse of the entire building structure. For this reason, performance of the column is very much important from the structural safety point of view. Now, if the existing column is damaged or deficient or incapable of withstanding the current loads, it is necessary to strengthened it for life safety. That's because complete replacement of a building in a given area may not be possible due to the historical importance or financial problems.

Structural strengthening of deficient RC columns is an innovative technical solution that improves the global behavior by optimizing the original strength, stiffness and ductility. There are different types of strengthening techniques available for deficient RC columns in practice such as RC jacketing; composite jacketing with blended materials (FRP, CFRP, and GFRP etc) and the steel jacketing. Among them, the method of steel jacketing has become one of the most efficient, accepted and broadly used column strengthening technique (Giménez *et al.,* 2009) all over the world.

Again, steel jacketing methods are accessible in various forms in practice. A very traditional form of this method is wrapping a sheet of steel along the full length of the deficient RC columns. Using partially or fully encased steel tube or steel box along with solid walls or collars surrounding the RC column is additional form of this method. However, a popular method of steel jacketing is to make steel cage around the RC column by steel angles and strips. It is regarded as the uttermost economical and swift strengthening technique over the other form of this method.

Even though this steel caging method is widely used in many countries, it has to date, received very little attention from the scientific affiliation. Limited experimental, numerical and analytical investigations have been found to be conducted by few researchers in the last few years. They mainly concentrate on concentrically loaded strengthened RC columns. However, the actual behaviors of the strengthened deficient RC columns under concentric axial loads are still remaining unknown. That's because, most of the researchers carried out their study on newly made, fresh and undamaged strengthened RC column which is very much contrasting to practical situation. Although, some design guidelines are prescribed in EC-4 (1994), EC-8 (2003), and ACI 318 M-99 (1999), the authentic strengthening methods are still obscure in Bangladesh. An attempt has been made in this study to conduct an experimental investigation on strengthened deficient RC columns till failure. Keeping analogy to the actual site circumstances, the RC column were partially damaged by initial loading of approximately 70% of their ultimate axial load. The damaged columns are then repaired and strengthened with vertical steel angles and horizontal strips. Size of steel angles, spacing of strips, and preloading state of the column at the time of strengthening will be the main variable parameters in this study. The efficiency of partial strengthening method at column ends will also be studied. This partial strengthening method has been considered to meet the ACI seismic design requirements for column ties.

1.2 Objectives of the Study

The main objectives of this study are:

a) To conduct experimental investigation on deficient RC columns strengthened using steel angles at the four corners connected with discrete horizontal steel strips;

b) To investigate the effects of steel angle ratio, strip spacing, partial strengthening and preloading state on the strength and failure behavior of the strengthened RC columns under concentric axial loads;
c) To compare the strength of jacketed RC columns obtained from the experimental investigations and the strength obtained from the capacity prediction models available in the published literature.

1.3 Scope of the Study

In this study, an experimental program is designed with eight square RC columns having a cross-section of 150 × 150 mm. The columns are reinforced with 1.4% of longitudinal steel. Seven of the test column specimens are strengthened while one kept unstrengthened. Five of the test columns are preloaded up to 70% of its ultimate capacity. Then, damages of the columns are repaired with locally available materials before strengthening. All specimens are tested using universal testing machine for concentric axial loads only. Ultimate load carrying capacity, failure behavior and ductility characteristics are investigated for each specimen during testing. The effects of 70% initial loading state of column, strip spacing and the size of longitudinal corner angles on the behavior and strength of the deficient RC columns are analyzed based on the test results. One different strengthening method applying steel jacketing only at column ends is also investigated. Besides, a suitable configuration of this strengthening method is suggested based on the current test results. Finally, the experimental results are compared with the capacity prediction models.

This study will be useful in predicting the increase in strength as well as to indentify the most important parameter affecting the behavior of steel jacketed preloaded RC columns. Also, it may be helpful to identify the most efficient methods of placing transverse plate along with the varying sizes of longitudinal steel angles that can be used in practice.

1.4 Report Organization

This report is organized in total five chapters. An overview of each remaining chapter is as follows: **Chapter two** includes a brief literature review on the steel angles and strips jacketed reinforced concrete columns. The behavior and failure mechanisms of the jacketed RC columns under concentric axial loads are also presented here.

Chapter three presents the test program and details of the test parameter to be examined. It presents a brief description of the specimens casting process, repairing procedures and strengthening methods using steel angles and strips. The mechanical material properties of the steel and concrete used throughout the test are presented here also. Those properties were measured through additional laboratory test. The test set-up for steel jacketed RC column loaded concentrically is also included. Besides, a brief description of the instrumentation and data acquisition is incorporated in this chapter.

Chapter four presents an overview of the results, observations and failure modes of each test column. Results are obtained from UTS machine through load-shortening curve. It also includes a detailed parametric study on account of the effects of pre-loading state at the time of strengthening, angle size, strips spacing and column ends strengthening parameter. Finally, the conclusions and some recommendations for future research have been included in **Chapter five.**

CHAPTER 2
LITERATURE REVIEW

2.1 General

Strengthening of reinforced concrete (RC) columns in buildings are required due to design deficiencies, material deficiencies, poor construction and quality control, corrosion of steel reinforcement and overloading above the admissible level. The methods of jacketing by reinforced concrete or other high performance materials are being widely used as strengthening solution in now-a-days. The benefits of jacketing are not only increased the axial load-carrying capacity but also improved the column lateral load-carrying capacity, stiffness, ductility or a pronounced combination in them. Jacketing generally serves to improve the column global behavior by exerting the confinement pressures on concrete under compression.

This chapter gives a review of the different jacketing method including reinforced concrete jacketing, FRP jacketing and steel jacketing regarding their certain advantages and some specific disadvantages. A general behavior of confined concrete and steel under compression are discussed here. Also, the composite behaviors of the RC columns strengthened by steel jacketing are included in this chapter. The failure mechanisms of the steel jacketed RC columns under the case of direct axial loading are presented. The description of the analytical models proposed by different researchers along with their experimental and numerical investigations is presented in this chapter.

2.2 Strengthening Methods for RC Columns

Deficient RC columns in buildings can be strengthened in several ways, such as RC jacketing, FRP jacketing, steel jacketing etc. The brief descriptions of these methods are presented below.

Reinforced concrete jacketing is usually referred by the application of a thin layer of reinforced concrete around an existing RC column. For ensuring the proper bond between the surface of old and new concrete, adequate numbers of anchored bars/shear keys, dowels and adhesive materials are used. It is expected that confinement can be improved easily, as the transverse reinforcement can be placed in the exterior of the longitudinal bars at a certain spacing required. However, the confinement through RC jacketing on rectangular or square cross section are not as effective as for circular cross sections. Literally, it is easy to install, and improves the ductility, shear capacity and load carrying capacity. In contrast, one of the most remarkable disadvantages of RC jacketing is the section enlargement, which is often not accessible. In addition, RC jacketing needs dowelling the reinforcing bars to the footing, eventually in many cases the failure mode is shifted there and becomes vulnerable to the seismic loading, thus retrofitting of that specified footing is required.

Fiber reinforced polymer (FRP) jacketing is recently considered as new and highly reliable materials in the construction industry. The fibers are a type of unidirectional flexible sheets or fabrics (can be woven or unwoven) that contains fibers in at least two different directions (Islam and Hoque, 2015). The fibers are then wrapped to the concrete using resin. The significant benefit of this jacketing method is light weighted about one-fourth of the steel. Other benefits include easy installation in limited space, minimal surface preparation that result reducing the labor costs and provide the substantial ductility. However, the disadvantages over this jacketing are: vulnerable to fire, linearly elastic behavior, which causes member failure without yielding or plastic deformation results low ductility. Furthermore, the fibers and resin are very expensive as compared to steel or concrete. FRP jacketing is effective only for the columns with circular or elliptical in shape. Unlike steel and RC jacketing, it has incompatible thermal expansion coefficients.

Steel jacketing is made by fixing four steel angles around the four corner of the rectangular or square RC column. The steel strips are welded horizontally to tie the angles and spaced at a rational spacing to form a steel case. The small gap left between the steel angle and the surface of the concrete column is then grouted using cement mortar to ensure full contact between the two of them. Figure 2.1 shows the RC column strengthened by steel caging (without additional elements at the ends of the cage). This method does not require highly trained labor and is very easy to inspect. In addition this method requires a limited space around the column section. It requires less fire protection than wrapping with FRP which needs a special protection from fire hazards [Tarabia and Albakry (2014), Islam and Hoque (2015)]. It also requires minimal concrete surface preparation. It is commonly used strengthening technique of RC columns with rectangular and/or square cross-section. Badalamenti. *et al.* (2010), Campione (2012), Abdel-Hay and Fawzy (2014) and Tarabia and Albakry (2014) showed that if properly designed, this retrofitting technique will enhance lateral strength, ductility, shear capacity as well as the axial load-carrying capacity of the existing deficient RC columns. This study is mainly devoted on this method. The comparisons of the three different jacketing methods are summarized below.

Table 2.1 Comparison of the different jacketing methods

Description	Concrete Jacketing	Steel Jacketing	FRP Wrapping
1. Preparation of the column surface for jacketing	Significant dismantling of concrete cover is required. At least 40 mm cover concrete to be removed.	Not major dismantling work involved. Mainly plaster to be removed.	Only plaster to be removed. For square/rectangular columns, corners to be rounded off.
2. Drilling of holes	Large amount of drilling is required	Small amount of drilling is required	No drilling required.
3. Increase in weight	Extremely high (the weight becomes 225% for just 50% increase in strength.	Very high (the weight becomes 165% for 50% increase in strength.	Negligible. No increase in weight at all.
4. Increase in size	Very high	High increase in strength.	Negligible.

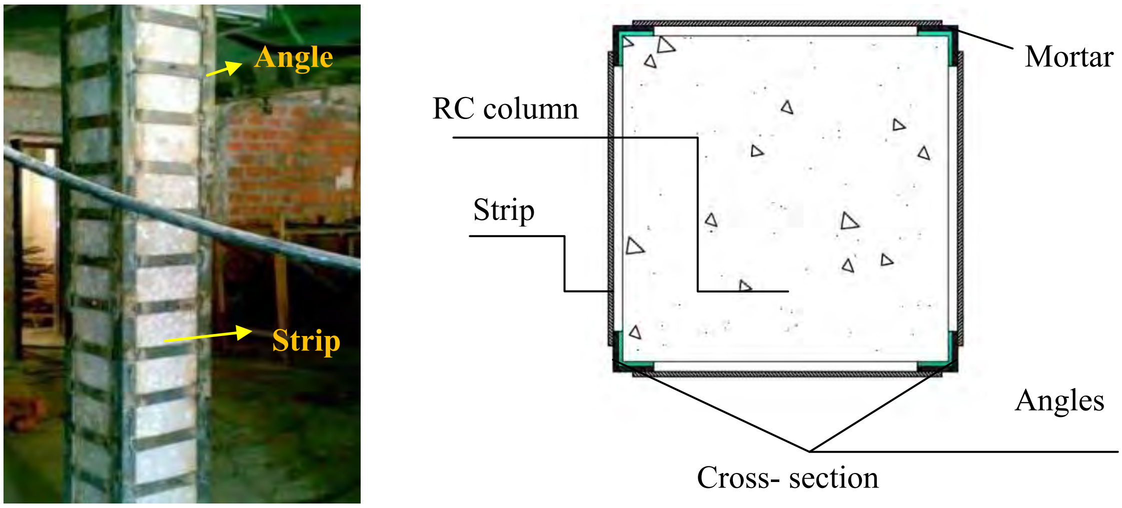


Figure 2.1 RC column strengthened by steel jacketing with angles and strips

2.3 Concrete Confinement under Compression

The behavior of confined concrete depends on the design adequacy of the transverse reinforcements provided in the column. The transverse reinforcements in terms of concrete confinements are generally applied to the member in compression and vitally functioned in attributing confinement pressure to the core concrete. Concrete confinement has a very large effect on gaining of the member's ultimate strength and ductility in potential plastic hinge region.

Generally, concrete expands outward as the strain is advanced inward as per Poisson's ratio. During this mechanism, the outward expansion is resisted by the lateral hoops or any other confining reinforcement by generating confinement pressure to the concrete core. That's how concrete is confined and permitted the member to earn greater aerometric (biaxial) strains under compression.

According to the Montuori and Piluso (2007), the confinement produced arching pressure to the core concrete under compression as in Figure 2.2 (b). This pressure is generated due to the action of lateral hoops only. With the decrease in spacing between the hoops, the pressure in confined concrete is increased. The increased confinement pressures results higher resistant against the concrete lateral expansion and leads to achieve ultimately higher compressive strength of the column.

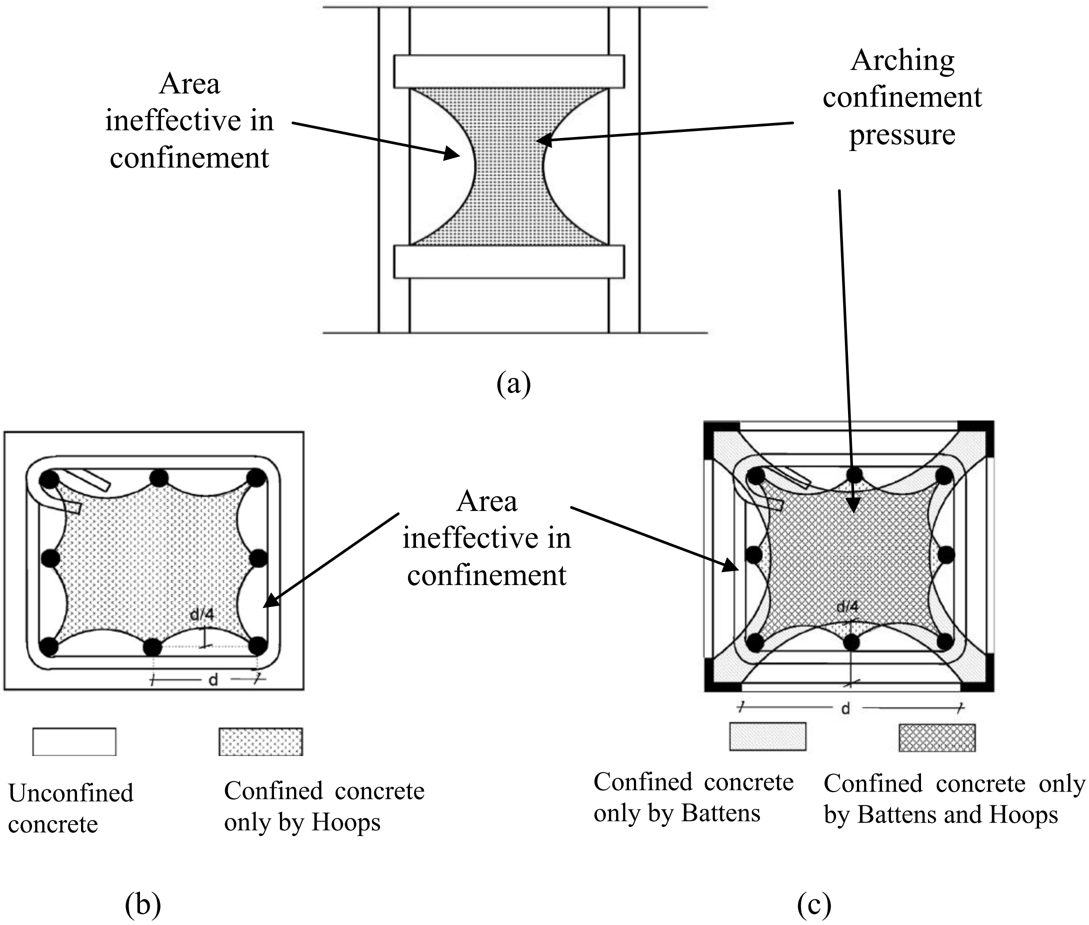


Figure 2.2 Confinement pressure due to steel jacket at, (a) elevation; (b) cross section before strengthening; and (c) cross section after strengthening at strip level. (Redrawn from Montuori and Piluso, 2009)

In steel angle and strip jacketed reinforced concrete columns, arching confinement pressure is produced in concrete due to the fixity provided by the steel cage against the concrete lateral expansion. According to the Montuori and Piluso (2007) and Euro code 8 (2003), this pressure is generated to the confined concrete by the action of steel strips only and by the combined action of strips plus hoops passively (Figure 2.2, c). Again, the confining pressure varies with the size and spacing of the lateral strips. This pressure also fluctuates along the length of strips (Nagaprasad *et al*., 2009). The strips provide discrete non-uniform confinement along the length of the column. So, the measure of confining pressure in steel angle and strip jacketed RC columns is a complex mechanism. However, the analytical models has been developed periodically by Braga *et al.* (2006), Montuori and Piluso (2007), Nagaprasad *et al.* (2009), Badalamentri *et al.* (2010), Li *et al.* (2009), and Calderon *et al.* (2009) for measuring the confinement pressure in steel caged concrete column. Their main focus was the distribution of the confinement pressure at the strips level and along the length of the angle between two strips.

2.4 Steel Behavior under Compression

Steel behaves much differently than concrete when under compressive loads. The slenderness of steel rebar increases its susceptibility to buckling and is therefore not meant to sustain heavy axial loads. When steel is compressed it is able to withstand increasingly larger loads until it reaches its yielding stress, at which point the steel will continue to experience large displacement while the applied load does not increase. In bending, steel fails once in fractures; however, while undergoing compression the steel does not get the opportunity to fracture due to buckling. The lateral steel that confines the concrete core simultaneously resists the buckling of the longitudinal steel rebar by reducing its effective length while increasing its compressive capacity

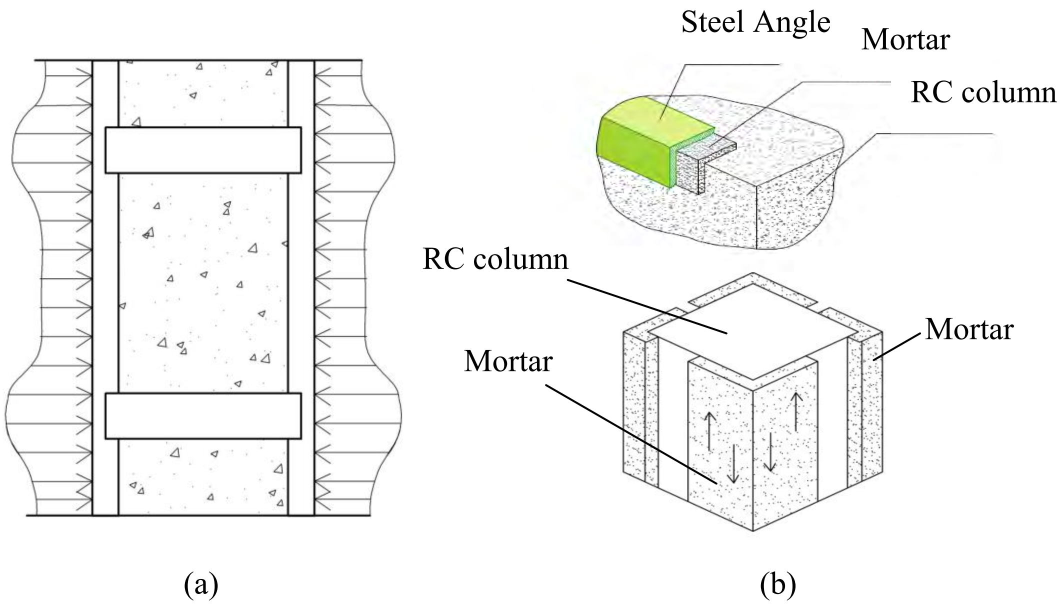


Figure 2.3 Load transmissions, (a) by confinement; (b) by shear stresses (Redrawn from Calderon *et al.,* 2009)

2.5 Composite Behaviour of Steel Jacketed RC Column under Compression

The behavior of RC columns jacketed with steel angles and strips is governed by two fundamental mechanisms. First one, confinement imposed by steel caging. The steel cage produces confinement pressure on the RC column. Since it prevents the expansion of the concrete caused by Poisson's effect under axial loads, compressive strength of the concrete is increased as well. The confinement pressure will be highest in the area of the cage covered by the steel strips, since these are the zones of greatest stiffness as regards transverse deformation. This effect is illustrated in Figure 2.3 (a).

Now, the load transmitted is occurred between the column and cage via the intermediate mortar layer by shear stresses (see Figure 2.3, b) and its variation depends on the effects of surrounding confining pressures. Since the confinement effect is highest in the zones nearest to the strips, the transmission by shear stresses will be more effective in these zones. This is the second fundamental mechanism keeps influence on the behavior of RC columns.

However, the strength contribution of steel angles is considered on the fact that angles are subjected to the combined effects of axial force and bending moment. The bending moment is the consequence of the reduced lateral expansion of the concrete core. Whether the angles are subjected to the loading directly or indirectly, the compressive force is enhanced due to the resistance of the column in axial. In indirectly loading case (angles are shorter than column), frictional action is induced which is proportional to the confinement pressure at the level of the strips in the cage.

2.6 Failure Mechanisms

The failure of the strengthened column occurs when the steel cage is no longer able to confine the concrete. Most of the researchers (Abdel-Hay and Fawzy, 2015, Tarabia and Albakry, 2014 and Calderon *et al.,* 2009) have shown that there are two possible failure modes for steel angle and strip caged RC columns when the column is subjected to the direct loading case:

2.6.1 Yielding of the Angles

Failure due to yielding of the steel angles (Figure 2.4, a) occurred as the axial loading in combination with the bending produced by the transverse deformation of the concrete in the column (Poisson's effect) absorbed. Calderon *et al.* (2009) spelled out that angles fail when a set of three plastic hinges are formed within a section of an angle bounded by two strips (Figure 2.4, a). He also showed this mechanism in most of the other laboratory tests and finite element models (Adam *et al.* 2009).

(a) (b)

Figure 2.4 Possible failure modes, (a) yielding of the angles; and (b) yielding of the strips. (Calderon *et al.,* 2009)

2.6.2 Yielding of the Strips

Failure due to yielding of the steel strips (see Figure 2.3, b) occurred due to the pressure caused by the transverse deformation of the concrete under Poisson's effect. The reason behind this type of failures is the use of quite smaller distance between the strips. Adam *et al.* (2009) also noted this failure mode in finite element models. When the length of the angles is smaller than the length of the column, concrete exerts compressive pressure on surrounding steel case which leads to the yielding of the steel strips. This is another one reason behind this type of failure.

2.7 Experimental and Numerical Investigations

Most of the researchers conducted experimental and numerical investigations on square reinforced concrete columns strengthened by steel cage to study the performance and the ultimate capacity. The investigators - Badr in 2006, ISSa *et al.* in 2008, Adams in

2009, Gimenez *et al.* in 2009, Calderon *et al.* in 2009, Campoine in 2012, Belal *et al.* in 2014, Abdel-Hay and Fawzy in 2014, Tarabia *et al.* in 2014, progressively offer their contribution in time to time with the intention of continuing the work begun by Ramirez (1996) and Cirtek (1999) some years ago. They mainly focused to know the effect of strengthening configurations on load carrying capacity, ductility, lateral strength and flexural strength by changing parameters like strip thickness, size and spacing, concrete strength, angle size and thickness, type of the injected grout material between column and steel case, and the existence of the connection between the steel cage and both heads of the column and partial strengthening**.** It has been found from the literature that load carrying capacity depends on aforementioned parameters. The experimental investigations conducted by the researchers reveal that the overall increase in axial strength ranges from 18.65% to 109% and that of lateral strength from 63% to 68%. However, some works and their findings are described as follows.

Ramirez *et al.* (1997) conducted experimental investigation on strengthened defected concrete columns. In the experiment, four angles were fixed at the corners of the column. Four steel plates were welded on to the angles. Two bonding methods were used to connect the steel plates to the original defected concrete column. In the first method, the gap between the steel plates and concrete was injected using epoxy resin and fine sand. In the second method epoxy adhesive was used. It was concluded that the steel plate jacket with injection proved to be a more reliable method. Debonding of the plates was observed at low bearing load. A continuous cracking noise was coming from the mastic layer between plate and angle throughout the test. The testing was ended with a sudden failure of the column. This was due to the low workability and brittleness of the used adhesion material.

Cirtek (2001) conducted a test program consisted of 39 specimens measuring dimensions of 300 × 300 ×1500 mm. The head and base of each column were shod in order to prevent early failure. The longitudinal reinforcement was welded to the steel shoes. The steel angles and bandages were used in the strengthening scheme. The bandage of a fully banded column had continuous steel angles and the bandage of a partially banded column had non-continuous ones. One of the main conclusions of this work was that the load-carrying capacity of the columns strengthened with bandage could possibly be increased by almost approximately 55%. Also, a mathematical solution adopting an iterative method was presented at the end of the study.

Badr (2006) studied the experimental behavior of eight rectangular reinforcement concrete columns with low compressive strength concrete. The columns were strengthened by placing four vertical angles in the column corners. The horizontal strips plates were distributed along the length of column and welded on the angles. The aspect ratios of the rectangular concrete column specimens were equal to two. The parameters of the study were the size of corner angles, spacing of the strip plates and the usage of anchor bolts in the middle of the long side of the columns. The results of the strengthened columns were compared with the analytical analysis suggested by Wang for confined concrete. The comparison showed a good agreement for the ultimate load of the strengthened columns. It was proved that decreasing the spacing of the horizontal steel strips improved the behavior of the strengthened columns. Also the use of anchor bolts to connect strip at the middle of the long side of the column, raised the strength of the columns by 16%.

Issa *et al.* (2008) conducted experimental, theoretical and numerical investigation to evaluate the behavior of reinforced concrete columns strengthened externally with steel jacket or fiber composite under axial loads. The experimental program presented six rectangular reinforced concrete columns with the same dimension of $150 \times 200 \times 1200$ mm. The steel jacket consisted of four vertical angles at column corners and horizontal steel plates welded to the corner angles and distributed along column height. The main parameter was the type of the external strengthening method. For the steel jacket the variables were the size of corner angles and the spacing between the steel plates. From the experimental study, it was concluded that increasing the area of corner steel angles and decreasing the spacing between the steel pattern plates of steel jackets increase the ultimate carrying capacity, and ductility of strengthened columns.

Adams *et al.* (2009) performed experiments on axially loaded RC columns strengthened by steel cages as well as numerical models using finite elements method to verify the obtained experimental results. Also, a parametric study was carried out to analyze the influence of each of the parameters on the behavior of RC columns strengthened by steel cages. The study considered these parameters: the size of the angles; the yield stress of the steel of the cage; the compressive strength of the concrete in the column; the size of the strips; the addition of an extra strip at the ends of the cage; and the friction coefficient between the layer of mortar and the steel of the cage. The obtained results of this parametric study were that the slippage between the steel cage and the column can be reduced by increasing the size of the strips due to the greater stiffness of the steel cage in the transverse direction. This improvement in confinement would also result in a better transmission of loads between the cage and the column by the shear stress mechanism.

Gimenez *et al.* (2009) conducted full-scale tests on 70% preloaded RC columns strengthened with steel angle and strip cages. The number of strips at the ends of the columns was increased to prevent premature failure occurred in a previous study. With this increase, the ultimate load of the strengthened column was increased. The variables of the study were as follows: preloading state of the column before applying the strengthening method, fitting a capital at the joint between column head and the cage, the adhesive between the surface concrete and the column. It was concluded that the addition of two strips of a smaller size in the sections near the heads considerably improved the ultimate load and ductility of the column. It also helped the strengthened column to take place failure in the central section as well as to earn composite behavior in these elements.

Khalifa and Tersawy (2013) developed a practical based analytical model and designed an experimental program on seven low strength reinforced concrete columns. Two series of strengthening procedure were considered in this study. First series contained four steel angles and uniform interval of strips. Steel casing by four plates connected with or without dowels were included in the second series. This study concluded that the load carrying capacity could be enhanced up to 66% using steel angle and strip strengthening series. This capacity proved to be doubled with steel casing by four plate’s series. This study also concluded that the increase of strip thickness and reduction on strip spacing resulted more effective strength and ductility than the increase in the steel angle dimensions. The presence of dowels exhibits comparatively slower failure of the column in steel casing techniques. Finally, the experimental and analytical results were compared and showed to be obtained a good agreement in them. The proposed analytical model accounted the composite action for concrete confinement and enhancement of the local buckling of the steel elements.

Tarabia and Albakry (2014) studied the behavior and efficiency of reinforced concrete square columns strengthened by steel angles and strips (steel cage). The main studied parameters were: size of the steel angles, strip spacing, grout material between column sides and angles, and the connection between the steel cage to the specimen head. Two different concrete strengths of 57.8 MPa and 47.5 MPa were also considered in this study. All the specimens were tested under concentric axial loads till failure. This study was concluded that jacketing by steel angles and strips proved to be a very efficient strengthening method. The gain in the axial load capacity of the strengthened columns was obtained from 1.35 to 2.10 of unstrengthened column. This gain was due to the confinement effect of the external steel cage, and the ability of the steel angle to resist an extensive part of the applied axial load. The failure in most of the strengthened specimens was due to the buckling of the steel angle followed by crushing of the original columns. The axial ductility of the strengthened column was also obtained to be increased by 50%.

Abdel-Hay and Fawzy (2014) investigated the effect of partial strengthening scheme on the capacity and global behavior of the partially defected RC columns. The experimental program was consisting of seven R.C columns with a dimension of $200 \times 200 \times 1500$ mm. The columns were made partially defected by placing stirrups in top and bottom thirds of the column only. The middle third of the column was fabricated without stirrups. The main studied parameters were: the type of steel jacket used and height of partial strengthened part of column. Three different types of steel jackets were used in the study. They were (1) using four steel angles at corners connected with straps, (2) using external ties with different spacing, and (3) using four steel plates with different thicknesses welded together and connected to the column by anchor bolts. All the columns were loaded concentrically till failure. This study concluded that increasing the strengthened part at mid height using external ties (minimum clear spacing not less than 150 mm) proved to be improved the overall behavior of the partially strengthened RC column. This study also observed that the failure of strengthened columns occurred outside the strengthened part. Finally, the study performed a finite element analysis using ANSYS program and showed a fair agreement with the experimental results.

Belal *et al.* (2015) performed both experimental and numerical investigations on seven specimens under compressive axial loading. The specimens were strengthened with different steel jacketing configurations. Three different vertical steel elements (angles, channel and plates) were chosen with the same total horizontal cross sectional area. Three studied variables were: shape of main strengthening system (using angles, C-sections and plates), size and number of batten plates. This study concluded that angles and channels proved to be performed similarly, but steel plates resulted in less capacity for the column, due to the thinness of the plate. Batten plates had variable results based on which cross-section was used. The jacketing system with channels resulted higher strength than angles. But the angles were found to be benefited more from improved confinement stress due to the discrete thicker plates. Additionally, the columns with angles experienced less deformation than from the other steel jacket/cage cross-sections. Additional consideration was recommended when using C-sections with batten plates or plates only, since their thinner thicknesses may present buckling problems.

Jodawat *et al.* (2016) conducted an experiment program on fifteen specimens in order to evaluate performance of jacketing systems in increasing load carrying capacity of cracked members. Two types of jacketing system were used in the experiment: steel plate jacketing and angle batten jacketing. Six specimens were kept preloaded with 85% of failure load. It was concluded that the angle batten system had higher strength gain and better confining effect than the thin plate system. Also, greater increment in strength and axial deformation was achieved for no preloaded specimens.

Ezz-Eldeen (2016) conducted both experimental and numerical investigation on fifteen column specimens in order to evaluate the efficiency of steel angles and strips jacketing method under eccentric loads. Four different eccentricities were used in the study. Then the twelve columns were divided in to three groups and strengthened them with three different angle set (two same angles in each set) in compression side and two same angles in tension side separately. It was concluded that the increasing the covered area of the steel jacket increased the load carrying capacity of the strengthened columns.

Finally, a parametric analysis was conducted using ANSIS finite element model for proposing practical used dimension in strengthening columns subjected to different eccentricities. Columns with cross sectional areas ranging from 25 × 35 cm to 25 × 120 cm were analyzed for this purpose and presented in a tabular form for practical application.

Latheef and Sumayyath (2017) conducted numerical analysis on nine RC column models. The columns were strengthened with steel angles and battens using 9 Taguchi models in the study. The numerical analysis was performed using ANSIS 15.8 finite element software. The intension of this study was to find out best model so that desired protection of life and damage can be protected. The deformation and stress could also be minimized. The used three varying parameters were: no of strips, thickness of strips and size of angles. Different combination of angle size and no of battens at different level were used in the model. This study concluded that an optimize desirable method combination of seven battens with 5 mm depth and angle with 35 × 35 × 5 mm size will give best result for life and damage.

Sen D. (2017) performed an experimental and numerical investigation on six steel jacketed reinforced concrete columns under eccentric loads. The steel jacketing was consists of four steel angles and discrete uniform distribution of steel strips. The results of this study indicated well performance of steel angle and strip jacketing scheme under both concentric and eccentric loading. The capacity enhancement was found about 240% under concentric loads compared to un-strengthened RC column, whereas the capacity enhancement was relatively lower under eccentric loading compared to concentric loading. This study revealed that eccentricity reduces the ultimate capacity of steel cage jacketed RC column of about 15% when eccentricity changes from 0 to 0.45 of column width ratio. The eccentricity also obtained to change the ductile behavior of the jacketed columns. However, the numerical analysis was carried out with ABAQUAS finite element software. The finite element models were showed fair agreement with the observed experimental results.

Table 2.1 shows the summary of the study based on the selective parameters of the current research. This table indicates that the most of the study is carried on accounting the variables of steel angle ratio and strip spacing for the unloaded RC column. The study considering these variables for preloaded column is very limited. Table 2.2 also shows that no study is attempted on partial strengthening method by steel angles and strips at the ends of the column in existing literature review. Although Abdel-Hay and Fawzy (2014) studied the partial strengthening method but the location of the used method was at middle part of the columns only.

Table 2.2 Summary of the study

	Study	Steel angle ratio	Spacing of the strips	Preloading state of the column	Partial strengthening at column ends
1.	Ramirez *et al.* (1997)				
2.	Cirtek (2001)	X	X		
3.	Badr (2006)	X	X		
4.	Issa *et al.* (2008)	X	X		
5.	Adams *et al.* (2009)	X			
6.	Gimenez *et al.* (2009)			X	
7.	Khalifa and Tersawy (2013)	X			
8.	Tarabia and Albakry (2014)	X	X		
9.	Abdel-Hay and Fawzy (2014)				
10.	Belal *et al.* (2015)		X		
11.	Jodawat *et al.* (2016)			X	
12.	Ezz-Eldeen (2016)	X	X		
13.	Latheef and Sumayyath (2017)	X	X		
14.	Sen D. (2017)				

2.8 Capacity Prediction Models

In the past few years, several Capacity prediction models have been anticipated for the determination of load carrying capacity of RC strengthened columns using steel angles and strips caging. The investigators derived capacity prediction models are Braga *et al.* in 2006, Monturi and Piluso in 2009, Eurocode 4 in 1994, ACI 318 M 99 in 1999, Eurocode 8 in 2009, Nagaprasad *et al.* in 2009, Badalamenti *et al.* in 2010, Li *et al.* in 2009, Calderon *et al.* in 2009, Campoine in 2012, and Tarabia and Albakry in 2014 etc.

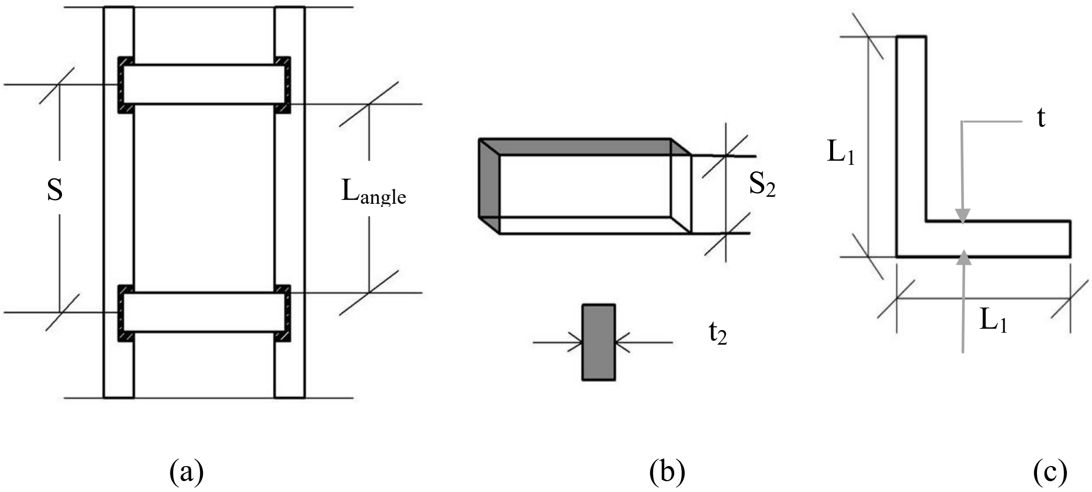


Figure 2.5 Components of steel jacketing, (a) at longitudinal section; (b) at strip section; and (c) at angle section

Most of the investigators addressed separately the increase in loads carrying capacity due to the confinement effects of the concrete core or to the composite action if angles are directly loaded or indirectly loaded approach in their models. The presence of both contributions in gaining of the strength is also demonstrated in some models. Some models accounted the composite actions of angles and concrete core when the angles are subjected to axial forces and bending moment induced from concrete lateral expansion. However, all of the models showed good agreement with the experimental and numerical results. Some capacity prediction models are described followed.

ACI 318 M-99 (1999) evaluates the design strength of retrofitted square reinforced concrete columns with steel jackets. However, the maximum nominal axial strength of column is presented by the Equation (2.2),

$$P_{ACI} = \left[\left(0.85 \times A_c \times f_c'\right) + \left(A_s \times f_y\right) + \left(A_j \times f_{jy}\right) \right] \tag{2.1}$$

This code does not incorporate the validity of the design equation for discontinuous steel jacket along the length of the column.

Euro code No. 4 (1994) determines the ultimate load carrying capacity as the following Equation (2.2),

$$P_{EC4} = \left(0.85 \times b \times d \times f_c\right) + \left(A_s \times f_y\right) + \left(8 \times L_1 \times t_1 \times f_{yL}\right) \tag{2.2}$$

This code does not include explicitly the direct application of reinforced concrete column strengthening method by steel angles and strips. The code rather considers this type of strengthened column analogous to a steel-concrete composite column.

Regalado and Pilares (1999) determine the allowable load carrying capacity due an allowance of incompatibility of deformation between concrete column and the strengthening system. It results a 40% lower ultimate load capacity of strengthened RC column than from Euro code No. 4 (1994), according to the Equation (2.3),

$$P_{\mathrm{Reg}} = 0.60 \times \left[\left(0.85 \times b \times h \times f_c'\right) + \left(A_s \times f_y\right) + \left(8 \times L_1 \times t_1 \times f_{yL}\right) \right] \tag{2.3}$$

Calderon *et al.* (2009) proposed a design equation for determining the ultimate load that is carried by a RC column strengthened with steel angles and battens without any additional elements in the end section. The formula is founded on the analysis of failure mechanisms observed in experimental and numerical approaches performed on full-scale specimens. The proposal was verified by comparing the results obtained from application of proposed formula, laboratory specimens test and FE models test. The application of this proposed design equation is an iterative procedure. This equation evaluates the contribution of the vertical angles through strain compatibility and friction. However, the design formula is expressed by Equation (2.4).

$$P_{Calderon} = (0.85 \times b \times d \times f_c) + (A_s \times f_y) + (2.5 \times f_l \times b \times d) + N_L \tag{2.4}$$

The parameters f_l and N_L are calculated by considering two failure cases: (1) Yielding of the angles and (2) Yielding of the strips.

Case-(1): Failure caused by yielding of angles:

The yielding of the angles occurs when the angels are no longer able to confine the concrete of the column and failure appears due to the formation of three plastic hinges in a section of the angle located between two strips as in Figure 2.5. However, the following equations are used in determining the parameter f_l and N_L for this failure case.

$$N_L = N_o \times \left(1 - e^{-ms_2}\right) \text{ And } \quad m = \frac{\mu \times 4 \times \upsilon_c}{b \times \left(1 - \upsilon_c + \dfrac{b \times E_c}{2 \times t_2 \times E_L}\right)} \tag{2.5}$$

Where, N_0 is the axial load absorbed by the concrete at the beginning of the strip and frictional coefficient, μ = 0.5 and Poisson's ratio, υ_c =0.2 are assumed. The qh and fl is determined by the following Equation (2.6)

$$q_h = \left(\frac{16}{l_{ang}^2}\right) \times M_p \quad \text{and} \quad f_l = \left(\frac{q_h}{b}\right) \times \sqrt{2}\,; \quad f_l = \left(\frac{16}{b \times l_{ang}^2}\right) \times M_p \times \sqrt{2} \tag{2.6}$$

The ultimate load for the case of failure by yielding of the angles will be obtained from Equation (2.4) after adding the parameters N_L and f_l.

Case-(2): Failure caused by yielding of strips:

A strip will yield when it is subjected to an axial load of ($t_2 \times S_2 \times f_{yL}$) perpendicular to the longitudinal axis of the column. The confinement pressure applied by the cage on the column when a strip yields is expressed by the Equation (2.7)

$$f_l = \frac{2 \times t_2 \times s_2 \times f_{yL}}{b \times s} \tag{2.7}$$

The confinement pressure f_l and axial load are calculated and added to Equation (2.4) to obtain the ultimate load carrying capacity of a strengthened column.

Guiseppe Campione (2012) proposed a design equation for determining the ultimate compressive load of a reinforced concrete column strengthened with steel angles and battens jacket. The formula is expressed as:

$$P_{U.campoione} = (n_a \times 8 \times t_1 \times L_1 \times f_{yL}) + (A_c \times f_{cc}) + (A_s \times f_y) \tag{2.8}$$

$$f_{cc} = f_{co}\left(1 + 4.74\frac{f_L}{f_{co}}\right)^{0.87} \tag{2.9}$$

Where, f_{co} is the compressive strength of the unconfined concrete.

$$n_a = \frac{\sqrt{t_1 \times f_{yL} \times \left(t_1 \times f_{yL} \times L_1^2 - \frac{q_{\max} \times S^2}{3}\right)}}{2 \times L_1 \times t_1 \times f_{yL}} \le 1.0; \text{ and } q_{\max} = \frac{16}{(S - S_2)^2} \times M_p \tag{2.10}$$

The calculation of M_p is an iteration process and is determined by

$$M_p = \left\{\frac{L_1^2 \times t_1}{4} f_{yL} - \frac{(N_p)^2}{16 \times f_{yL} \times t_1}\right\}; \text{ Where } N_p = 2L_1 \times t_1 \times f_{yL} \tag{2.11}$$

The parameter (f_L) can be determined by the following:

$$f_L = \left[f_{ys} \times \frac{t_2}{S} \times \frac{S_2}{b} \times e^{\left(1-1.5\frac{S}{b}\right)}\right] \quad \text{(Yielding of Strips)} \qquad 2.12)$$

$$f_L = \left[\frac{16 M_p \sqrt{2}}{f_c} \times \frac{1}{b(S - S_2)} \right] \quad \text{(Yielding of Angles)} \tag{2.13}$$

Tarabia and Albakry (2014) proposed an equation for predicting the load carrying of a column strengthened by steel caging. This equation is developed based on the simple mechanics and strain compatibility of the vertical angles. The presence of connection between the angles and head is also considered in this study. This equation is similar to that presented by Calderon *et al.* (2009) only with different approaches for determining confining pressure (f_l) and axial load carried by steel angles (N_L).

However, the average confining pressure and axial loads carried by steel angles are obtained by using Equation (2.14), (2.15) and (2.16), respectively.

$$f_l = \frac{N_c}{b^2} \times \frac{\upsilon}{\left(1 - \upsilon + \dfrac{b \times S \times E_c}{2 \times S_2 \times t_2 \times E_s} \right)} \tag{2.14}$$

Where, N_c is the axial load carried by the concrete. The axial load is carried by the angle depends whether it is connected to the head or not. If the angle is connected to the head and axial shortening of the column occurs or due to friction, it is referred as directly loaded angle. However, if the angles are not connected to the head of the column it is indirectly loaded angles and the axial force of one angle evaluated as:

$$N_L = 2L_1 \times t_1 \times f_{yL} \quad \text{(Direct Loading)} \qquad 2.15)$$

$$N_L = \sqrt{2} \times f_l \times b \times S \times \mu \quad \text{(Indirect Loading) } [\mu = 0.5] \tag{2.16}$$

2.9 Summary

Abovementioned, literature review illustrated that extensive experimental and numerical research has been conducted on steel jacketed (angles and strips) strengthened RC columns. Different equations have also been designed for predicting the ultimate load carrying capacity of the strengthened RC columns in some study. However, the deficiency is found in the domain of preloading column from the literature review which is illustrated in Table 2.2. In most of the study, the experiment has been carried on fresh, undamaged and unloaded column specimens that are dissimilar to the practical situation. In practical, existing columns must be preloaded, deficient and severely damaged before strengthening. For this reason and in order to enhance knowledge, the behavior and strength of the steel jacketed strengthened RC column must be explored for preloading state. The performance of the preloaded column should be investigated for different strengthening configurations of angles and strips. Also, they should be justified against the capacity prediction models presented in section 2.8.

CHAPTER 3
EXPERIMENTAL PROGRAM

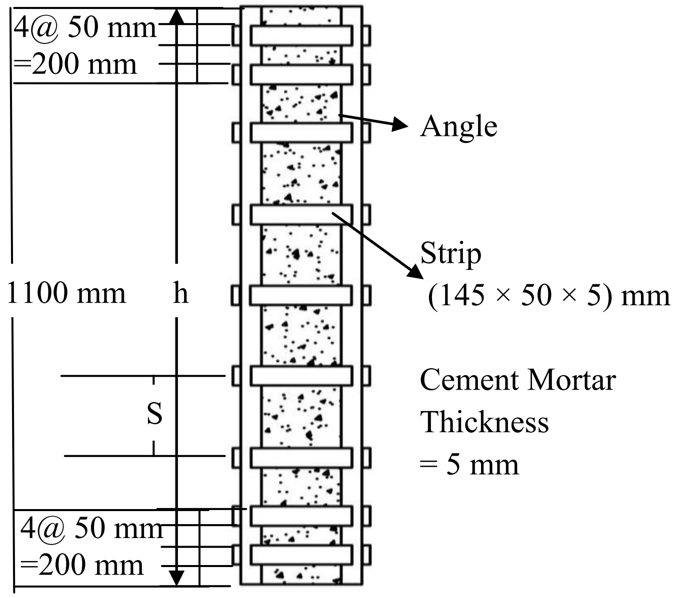
3.1 General

The main objective of this study is to investigate the strength and behavior of the steel angles and strips jacketed RC columns under concentric axial loads. The four parameters considered in this study were, the partial strengthening by column ends, 70% preloading state at the time of strengthening, steel angle ratio and the horizontal strip spacing. For this purpose, eight square RC columns were planned with different strengthening configuration and tested in axial compression. Preparation of the test specimens, test set up and data acquisition system are presented in detail in this chapter.

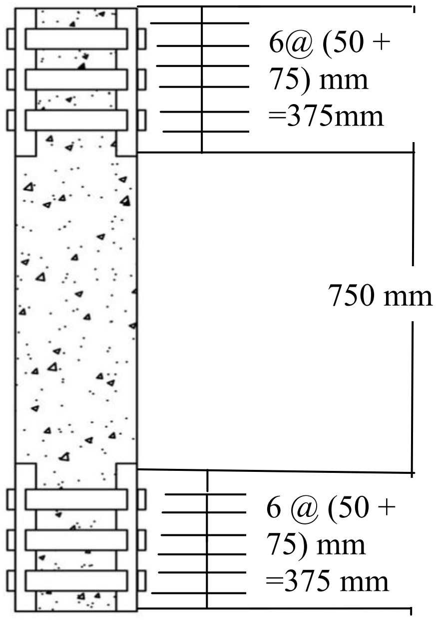
3.2 Description of the Test Specimens

Eight square reinforced concrete columns were constructed with a fixed cross sectional dimension of 150 × 150 mm (Figure 3.1). The height of each of the column was 1500 mm. All of the columns were reinforced with four 10 mm diameter of longitudinal deformed steel bars resulting in 1.4% steel ratio. The transverse reinforcements were in 8mm diameter and placed at a spacing of 150 mm c/c in the middle three fifth zone of the column. However, closely spaced ties (8 mm @ 50 mm c/c) and relatively higher strength concrete was used at the end one fifth zone of the test column to prevent premature failure at the ends. Among the eight specimens one was kept unstrengthened as reference column (designated as NC1). The capacity and behavior of the strengthened columns will be compared with this bare concrete specimen (column NC1).

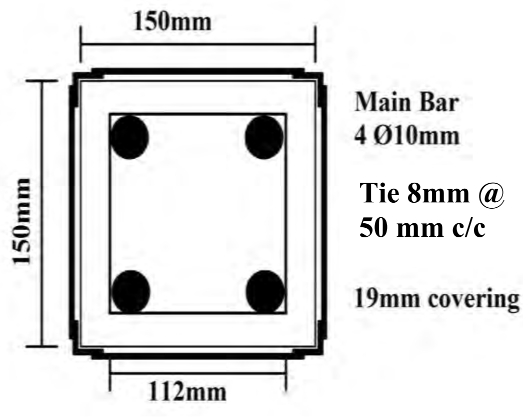
Six of the test specimens were strengthened using steel angles and strips along the full length of the column (as shown in Figure 3.1 a) and one column was strengthened only at the ends (as shown in Figure 3.1 b). In this column (designated as NS1) steel angle was discontinued in the middle one half zone of the column. This partial strengthening scheme was adopted to observe the effect of steel jacketing only at the ends for columns which do not satisfy the seismic tie requirements as specified in ACI.



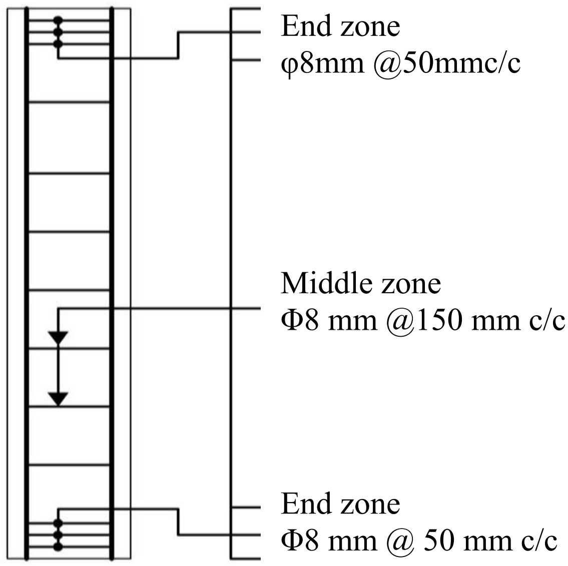
(a) Fully strengthened column



(b) Partially strengthened column



(c) Cross section



(d) Longitudinal section

Figure 3.1 Geometry of the test specimen

Table 3.1: Geometric Properties of the Tested Column

Sl no	Specimen Designation	Angle Size (mm)	Strip Size (mm)	Reinforcement		Steel ratio		
				Main rebar	Tie rebar (mm)	Angle	Rebar	Strip*
1.	NC1	×	×	4-φ10mm	φ8mm@150	×	×	×
2.	NS1	25×5	140×50×5	4-φ10mm	φ8mm@150	4%	1.4%	3.3%
3.	NP1.1	25×5	140×50×5	4-φ10mm	φ8mm@150	4%	1.4%	3.3%
4.	NP2.1	40×5	140×50×5	4-φ10mm	φ8mm@150	7%	1.4%	3.3%
5.	NP3.2	25×5	140×50×5	4-φ10mm	φ8mm@150	4%	1.4%	3.3%
6.	NP4.2	40×5	140×50×5	4-φ10mm	φ8mm@150	7%	1.4%	3.3%
7.	NU3.2	25×5	140×50×5	4-φ10mm	φ8mm@150	4%	1.4%	3.3%
8.	NU4.2	40×5	140×50×5	4-φ10mm	φ8mm@150	7%	1.4%	3.3%

[Strip* = (surface area of one strip / surface area of column)]

Table 3.2: Characteristic Properties of Test Specimens

Column	Variables Used in the Parametric Study					
	(i) 70% Preloading state at the time of strengthening	(ii) Angle Steel Ratio	(iii) Strips Spacing		(iv) Partial Strengthening at Column Ends	
			Spacing [mm] at middle	(s/d) Ratio	Angles Location	Angles Length [mm]
NC1 (control)	×	×	×	×	×	×
NS1	Preloaded	4%	50	×	End	750
NP1.1	Preloaded	4%	150	1.0	Full	1500
NP2.1	Preloaded	7%	150	1.0	Full	1500
NP3.2	Preloaded	4%	300	2.0	Full	1500
NP4.2	Preloaded	7%	300	2.0	Full	1500
NU3.2	×	4%	300	2.0	Full	1500
NU4.2	×	7%	300	2.0	Full	1500

In this test program two different sizes of equal leg angles L25 × 5 mm and L50 × 5 mm were used. Steel angles were placed at four corners of the columns via mortar grouting. Same size of strips were welded on the angles and placed at a fixed spacing in middle column zone. The strips in end zone were placed at 50 mm interval. The dimension of the strip was 140 × 50 × 5 mm.

Typical view of strengthening configuration used in the test columns are shown in Figure 3.1 (a). The geometric and characteristic properties of the test specimens are listed in Table 3.1 and Table 3.2, respectively. Five of the test columns were preloaded up to 70% of their ultimate strength. The damage that occurred during preloading stage was repaired before strengthening. The repairing and strengthening procedures are discussed later in this chapter.

3.3 Specimen Identifications

The columns were identified according to their different strengthening scheme. The seven strengthened columns were divided into three groups: NSXY, NPXY and NUXY.

The letter "N" in first position indicates to **normal** strength concrete. Three different letters (S or P or U) are used in second position of column designation. Here, letter "S" indicates to the partially strengthened column, "P" is for **preloaded** strengthened column, and "U" is used for **unloaded** strengthened column. The third letter "X" stands for angle size variation. When this alphabet is replaced by **odd** numerical numbers (1, 3), it indicates **smaller** size of steel angles (L25 × 5 mm). On the other hand, symbolizing by **even** numbers (2, 4) indicates to the **larger** size of steel angles (L40 × 5 mm).The fourth letter "Y" is used for strips spacing to depth ratio **(s/d)**.

3.4 Explanation of the Test Parameters

The main objective of this study is to investigate the effect of various configuration of steel jacketing strengthening method for deficient RC column. Four geometric variables that can significantly affect the strength and failure behavior of RC columns are selected for investigation in this experimental study. The parameters include: steel angle ratio, horizontal strip spacing, preloading state and partial strengthening scheme. The ranges of each variable along with the column designation used in the study are presented in Table 3.2. Increase in the steel ratio can significantly affect the strength and ductility of the column by enhancing the confinement pressure on concrete. To investigate this effect and observe the performance of the available capacity prediction model, two different steel ratios: 4% and 7% is used in the study. Two different types of strip spacing are used 1d and 2d also to see the affect on strengthened column behavior. Another important parameter is the column preloading state. Most of the studies are mainly on the fresh unloaded columns. Limited studies available in the literature are on preloaded column. Therefore, preloading state is used to investigate the effect on RC column. The partial strengthening is used basically to measure the efficiency when discontinuous angles are placed only at the column ends. This method is considered to meet the ACI seismic design requirements for column ties.

All the columns are expected to have improved strength, ductility, post-peak response and failure mode. They are compared mainly to identify the most efficient parameter which keeps major contribution in gaining strength and ductility from steel jacketing method. Finally the test results will be compared with results obtained from available capacity prediction models. This will help to identify their applicability for the selected ranges of parameters in this study.

3.5 Material Properties

The properties of the material used in the preparation of the test specimen are presented below.

3.5.1 Cement

Ordinary Portland cement sponsored by Seven Rings Cement Company (Product of Bangladesh) was used throughout the investigation. The cement properties conform to the Specifications limits of ASTM C150: Standard Specification for Portland cement.

3.5.2 Fine Aggregates

Local Sylhet sands and plain sands were used as fine aggregates for concrete mixes in this experiment. The fine aggregate was sieved at sieve size (4.75mm) to separate the aggregate particles of diameter greater than (4.75 mm). The obtained results indicated that the fine aggregate grading and a fineness modulus of 2.46. Fine aggregates was conformed the specification of ASTM C33.

3.5.3 Coarse Aggregates

Locally accessible brick khoa was used as coarse aggregate throughout the experimental works. The coarse aggregate was sieved at sieve size of 19 mm and 12.5 mm in order to separate the 19 mm downgrade and 12.5 mm retained of aggregate particles. Coarse aggregate properties was conformed the specification of ASTM C33.

3.5.5 Steel

For all test columns, two different sizes of steel reinforcing deformed bars were used. The 10 mm and 8 mm diameter bar is used as longitudinal and tie reinforcement, respectively. The RSRM steel angle and flat bar (strip) were used during strengthening. The mechanical properties of used steel bars, angles and strips were obtained from tension coupon test and are shown in Table 3.3.

Table 3.3 Properties of steel elements

Steel Properties		f_y (MPa)	ε_y (mm)	f_u (MPa)	ε_u (mm)
Reinforcement	10 mm	320	0.0012	434	0.262
	8 mm	350	0.0016	493	0.019
Angle	40×5	315	0.0018	395	0.274
	25×5	360	0.0024	451	0.322
Strip	145×50×5	270	0.0014	338	0.161

3.6 Preparation of the Test Columns

All the specimens were cast in the same day with same concrete strength of 15.2 MPa. The procedures followed during the castings of specimens are presented in next.

3.6.1 Concrete Mix Proportion

The main properties of interest during the mix design were low strength and high workability of concrete. Since, the column was designed to have relatively higher strength concrete in end zones and lower strength concrete in middle zone; two different strengths of concrete were made during the casting. Their mix designs are presented in Table 3.4

Table 3.4 Concrete mix design at Saturated Surface Dry (SSD) conditions

Material		Normal Strength	Relatively Higher strength
Water	(Kg)	34	43
Cement	(Kg)	50	100
Sylhet Sand	(Kg)	105	152
Plain Sand	(Kg)	28	-
Bricks Khoa	(Kg)	210	305
W/C ratio	(Kg)	0.68	0.43

3.6.2 Formwork

Timber shuttering was used as formwork. The thickness of shuttering was 20 mm. Two different lifts were made for two different concrete zones. They were end lifts for relatively higher strength concrete and middle lifts for normal strength concrete. The formworks with corresponding lifts are shown in Figure 3.2 (d). The joints of the formwork were sufficiently tight to prevent loss of liquid from the concrete. The inner surface of timber shuttering well wetted before casting. A coated of raw linseed oil was given inside of the form work in order to prevent adhesion of concrete and shuttering. Once the formwork was completed, reinforcements were placed according to the drawings. The reinforcements were kept clean, free of shuttering oil and adequately tied. The concrete aggregate materials were kept under saturated surface dry condition for about 24 hours (Figure 3.2, c).

3.6.3 Mixing of Concrete

A mechanical mass-batch mixer was used during the casting of the concrete column specimens. At first, the coarse aggregate, fine aggregate and cement were weighted separately and put into the mixer machine. Mixer machine was rotated for two minutes (Figure 3.2, e). Then water weighted and pouring into the machine. Mixer machine was rotated for more than five (05) minutes to make a homogeneous mixture. Finally, freshly mixed concrete was taken out from mixer machine and slump test was performed. When discharging, no segregation in concrete was appeared to occur. The mixer was fully discharged before recharging the next batch. In this way, total three batches of concrete were prepared for casting. Two batches of concrete were made for end lifts whilst the one batch was for middle. A suitable timing device was used during the mixing procedure.

Table 3.5 Concrete cylinder properties

Concrete Type	Cylinder Designation	Batch Mix No.	Strength (MPa)		Strength Increase (28 Day to Test Day)	
			28 Day	Test Day	(MPa)	(%)
End zone Concrete	HSC - 1	1	24.1	-	9.0	37.3%
	HSC - 2	1	23.9	-	9.2	38.5%
	HSC - 3	1	-	33.1	-	-
	HSC - 4	2	22.9	-	9.1	39.7%
	HSC - 5	2	23.0	-	9.0	39.1%
	HSC - 6	2	-	32.0	-	-
Mean			**23.5**	**32.6**		**38.7%**
Test Zone Concrete	NSC - 1	3	15.2	-	8.8	55.3%
	NSC - 2	3	16.3	-	8.9	53.6%
	NSC - 3	3	14.1	-	9.7	66.4%
	NSC - 4	3	-	24.2	-	-
	NSC - 5	3	-	25.0	-	-
	NSC - 6	3	-	23.9	-	-
Mean			**15.2**	**24.4**		**60.5%**

3.6.4 Casting and Curing of Concrete

The whole concreting procedures i.e. concrete preparing; transporting, placing and compacting had taken in total fifty (50) minutes after mixing. The placing of concrete was done in two levels. At first level, relatively higher strength concrete were placed at end lifts as with Figure 3.3(d). Since it contained relatively lower water cement ratio, it was expected not to spread towards the middle lift. In total two batches of concrete were required to make in casting of these lifts. At second level, concrete was placed at middle lift immediately after the end lifts. The casting of this lift was completed by making only one batch of concrete. However, there was a gap of about twenty five (25) minutes in placing of concrete between middle and end lift. The concrete was placed in three even layer; each of which was vibrated before the next one was placed. Figure 3.3 (e) shows surface leveling after casting of concrete eight column specimens in order to obtain a good concrete surface. The compaction was done by an internal type electric vibrator (Figure 3.3, c). The speed of the vibrator was 7000 rpm.

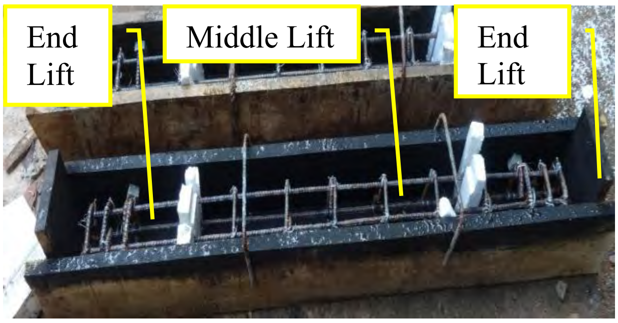
The formwork of the concrete column specimens was removed after twenty four (24) hours of casting. Afterwards, the specimens were wrapped by jute bags and then keeping constantly wet. The curing period was continuous for about 28 days (4 times per day).

3.6.5 Slump Test

A measure of the degree of consistency and extent of workability is called the *slump*. After the making of fresh concrete, it was checked in terms of the slump cone test. During casting, two different slumps were tested for two different batching of fresh concrete. The slump value of fresh concrete was measured 60 mm for column end zone and 150 mm for column test zone concrete (Figure 3.3, a and b).

(a) closed tie (135° hook)

(b) SSD of aggregates



(d) Shuttering and reinforcement

(e) Mechanical batch-mixer machine

Figure 3.2 Pre-concreting steps before the casting of the RC column specimens.

(a) Slump for column end zone concrete

(b) Slump for column mid zone concrete

(c) Compacting of concrete

(d) Concrete casting in end lifts first

(e) Concrete specimens after casting

(f) Specimens after 28 days of curing

(g) Curing of cylinders in water tank

(h) Lime white wash of Specimens

Figure 3.3 Major steps of casting RC column specimens.

3.6.6 Concrete Cylinder Specimens

Twelve concrete cylinders were made during casting according to the specification of AASHTO T 23-08. Among them, six cylinders were made from middle lift concrete whilst the remaining six were made from end lifts concrete. Again, the end lifts concrete was cast from two batches of concrete. For this reason, three cylinders were made from each batch of the concrete. The cylinders were standard 100 mm in diameter and 200 mm in height. All the cylinders were removed from the mold after twenty four (24) hours and marked with appropriate identifications. Curing was accomplished in wide water tank (Figure 3.3, g). Table 3.5 shows the mechanical properties of the concrete cylinder specimens.

3.6.7 Lime Whitewashing of Column Specimens

Conforming to the specifications of IS 6278-1971: Code of Practice for Whitewashing and Color Washing, all the columns were whitewashed by a layer of very thin coat of thin plaster made with lime, water and other ingredients in the casting place (Figure 3.3, h). This whitewash was required in order to trace any kind of produced undesirable cracks on column concrete surfaces during preloading functions. In total two layer of lime coating was needed for the sake of getting a clean, neat and uniform concrete column surfaces.

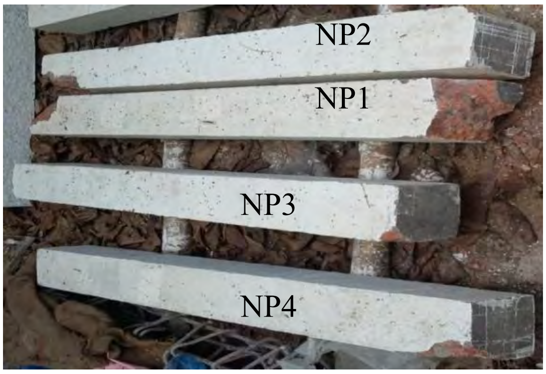
3.7 Repairing Procedures

In total five columns were preloaded concentrically in Universal Testing Machine (UTM). After preloading, four columns were found damaged at their ends. So, they were required to repair before strengthening. Conventional cast-in-situ process was adopted in repairing works. The important factors influenced on this selection were: the availability of skill, the adequacy of time, enough access for repair and cost. The strategy that was followed in repair of the damaged columns described eventually.

At first, visual inspection was performed on the preloaded specimens in order to gather information about the deterioration (Figure 3.8, a and b). The performed damage inspection was based on the following factors: (a) external appearance of the column

(b) cracking status (c) concrete covering (d) concrete spalling (e) steel bar corrosion. Table 3.6 shows the damage description and preload test summary for concentrically loaded column specimens.

(a) upper end [front face]



(b) lower end [back face]

Figure 3.4: Damages of the RC column specimens after the stage of 70% preloading.

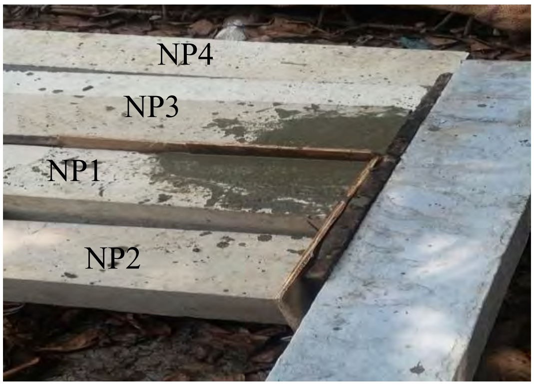
Table 3.6 Damage and preload test summary

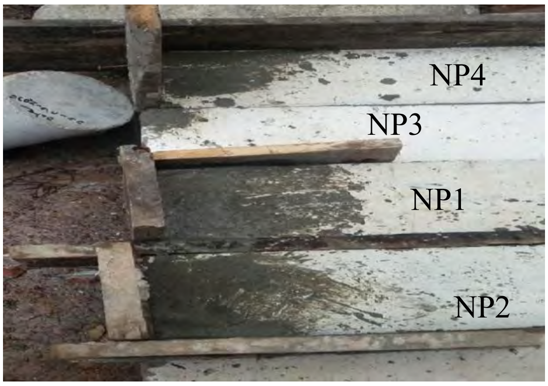
Column	Applied Axial load	Applied Axial load in (%)	Damage Description
NP1.1	335 (kN)	73%	➢ A very large amount of concrete spalling from both end zone ➢ Appearance of bottom end zone reinforcement
NP2.1	350 (kN)	76%	➢ Concrete spalling from both end zone ➢ Appearance of upper end zone reinforcement
NP3.2	312 (kN)	68%	➢ A very light concrete spalling from upper end zone.
NP4.2	325 (kN)	71%	➢ Concrete spalling from bottom end zone corner.
NS1	307 (kN)	67%	➢ No significant damage found.

Second, all loose and spalled concrete cover was removed wherever loose found by tapping on the specimen. Then the longitudinal reinforcement bar was cleaned of concrete to give a minimum 15 mm clear air gap all around including behind the reinforcement. Simple manual methods (hammering the rebar, using wire brushes, chiseling etc) were followed for concrete removing and preparing of the damaged column surfaces.



(a) (b) (c)





(d) (e)

Figure 3.5 Repairing methods, (a) Concrete and cement milk; (b) Application of cement milk; (c) hand compaction of the concrete; (d) Specimens following the repair at upper end; and (e) Specimens following the repair at lower end.

Third, a suitable wooden shuttering was installed around the damaged portion of the specimens. Then, the concrete was prepared by hands. The locally available micro stones and Sylhet sand were used as aggregates. The concrete mix proportion was 2: 3: 5. The measured quantity of coarse aggregates was spread evenly on a clean, paved and watertight platform. Then fine aggregates and cement were placed on the coarse aggregates one after another. The whole dry mass was mixed thoroughly for ten minutes by hand and turned over three (3) times by shoveling and twisted from centre to side then back to the centre and again to the sides. A hollow was made in the middle of the mixed materials. Three fourth (¾) of required quantity of water (680 ml) was added and turned the materials from side to centre with spade. Finally, the remaining water (230 ml) was added and slowly turned the whole mixture for five times until the whole surface of each aggregate become coated with sand-cement mortar. While mixing and handling the concrete, safety shoes was worn by labor. After using the concrete the mixing platform was cleaned properly.

Fourth, the prepared concrete was poured into the shuttering jacket one by one column. Prior to the placing of the concrete, the prepared concrete damaged surface area was washed off with the cement milk (see Figure 3.5, b). Cement milk was used as bonding coat between the concretes and formed by mixing cement with water into fluid. Hand compaction (Figure 3.5, c) was carried out by rodding due to the thin vertical sections. The four damaged concrete columns following the repairs are shown in Figure 3.5 (d) and (e). It was taken two different days for repairing two different face of the damage column.

Table 3.7 Concrete cube properties

Sl no	Mortar Designation	Strength (MPa)		Strength Increase (28 Day to Test Day)	
		28 Day	Test Day	(MPa)	(%)
1	NSCC - 1	14.7	-	-	-
2	NSCC – 2	16.2	-	-	-
3	NSCC – 3	18.1	-	-	-
4	NSCC – 4	-	19.6	4.9	33.3%
5	NSCC – 5	-	21.0	4.8	29.6%
6	NSCC - 6	-	22.4	4.3	23.8%
	Mean	**16.3**	**21.0**		**28.8%**

Finally, water curing was carried out by jute bags for a period of seven (07) days after repair. The dimension of the repaired columns were rechecked after seven days of curing and scaled down. Then the columns were strengthened and next section gives an outlines on strengthening procedures.

3.8 Strengthening Procedures

In total seven test specimens were strengthened as per schemes. Similar procedure was followed during the strengthening of all test specimens.

At first, the column surface was roughened by chipping off the plaster from the side corner. These deeds were done using chisel and hammer manually as shown in Figure 3.6. It was expected that this surface treatment would enhance the bond between substrate concrete and overlay mortar. However, all rubbish from these works was disposed as long as nine (09) meters distance from the working place.

(a) (b) (c)

(d) (e) (f)

Figure 3.6 Initial stage of strengthening procedure, (a) Cutting of steel angles; (b) chipping of plaster; (c) Saturation of concrete surface; (d) Application of cement milk; (e) Jacketing of specimens with steel angles; and (f) Preparation of cube Specimens

Second, the steel angles were placed on the column corners firmly (Figure 3.6, e). Before placing the steel angles, prepared concrete surface was washed with water (see Figure 3.6, c) and kept saturated for two hours. Then, a light cement wash and cement sand mortar was applied on the roughened concrete surface one after another

(Figure 3.6, d). The mortar mix ratio was typical 1: 3. The grouting thickness was 5 mm. As such, one side of entire seven test specimens was jacketed in one day. Next day, other side of the test specimens was jacketed. Water curing was carried out for 28 days by jute bags after passing twenty four (24) hours of column jacketing by steel angles.

(a)

(b)

(c)

(d)

Figure 3.7 Final stage of strengthening procedure, (a) Steel plate cutter machine; (b) Continuation of the welding works; (c) Horizontal strips following the completion of welding works; and (d) Seven test column specimens after strengthening by steel angles and strips

In total six (06) cement mortar cube specimens were made (Figure 3.6, f). The size of the cube specimens were 50mm. All the cubes were prepared inside of the Concrete Laboratory. They were placed in the laboratory moist room for twenty four (24) hours after making. After removing the cubes from the mould, they were immersed in clean water for curing and kept them there until 28 days. Table 3.7 shows the mechanical behavior of the cement mortar cube specimens.

Finally, the steel strips were welded on the angles horizontally as per drawing. The strips were made from flat bar with a size of 50 mm in width and 5 mm in thickness. Then they were pieced apart through a steel plate cutter machine (Figure 3.7, a). In total two hundred and twelve (212) piece strips were prepared for making the case. Fillet welds were employed for welding tasks (see Figure 3.7, b). The diameter of the electrodes was 4 mm as per design. Before the welding, entire steel strips were securely held in horizontal position by means of spot welds. It was done to prevent any kind of relative movement of the strips during welding. Figure 3.7 (c) shows the horizontal steel strips following the completion of welding. All of the seven test specimens after strengthening by steel angles and strips jacketing are shown in Figure 3.7(d).

3.9 Test Set Up, Instrumentation and Data Acquisition System

All the strengthened columns were tested at the Mechanics of Solid Laboratory in BUET during July and August, 2016. The age of the concrete during the testing was over 210 days. Two different test set up were used for preloading and testing stage of the strengthened RC columns. But, in both stage, similar test set ups, test machine and data acquisition system was utilized.

The test was performed using a Universal Testing Machine that carrying loading capacity of 2000 kN. The UTM actuator, which is attached to a moveable crosshead, applies compressive force from the above and monotonically until failure. The loading application generally stopped to a maximum range of 1500 kN to 1600 kN for avoiding any undesirable causalities of the machine. The base of the UTM sits on a strong base above floor. The data acquisition system consists of signal conditioners and a PC running Lab VIEW data acquisition software. The machine tested specimens using Horizon software, which records the applied axial displacement and resistance load automatically by the “Generic Compression Force vs. Deflection” mode.

Before placing the specimen in the UTM, specimen alignment was checked with the aid of a plumb bob. It was done by hanging the bob downward from the top of the specimen so as to get it touched the ground. A ruler was used to measure the horizontal distance between the plumb bob and the column at three different elevations from bottom end. The whole procedures are shown in Figure 3.8(a)

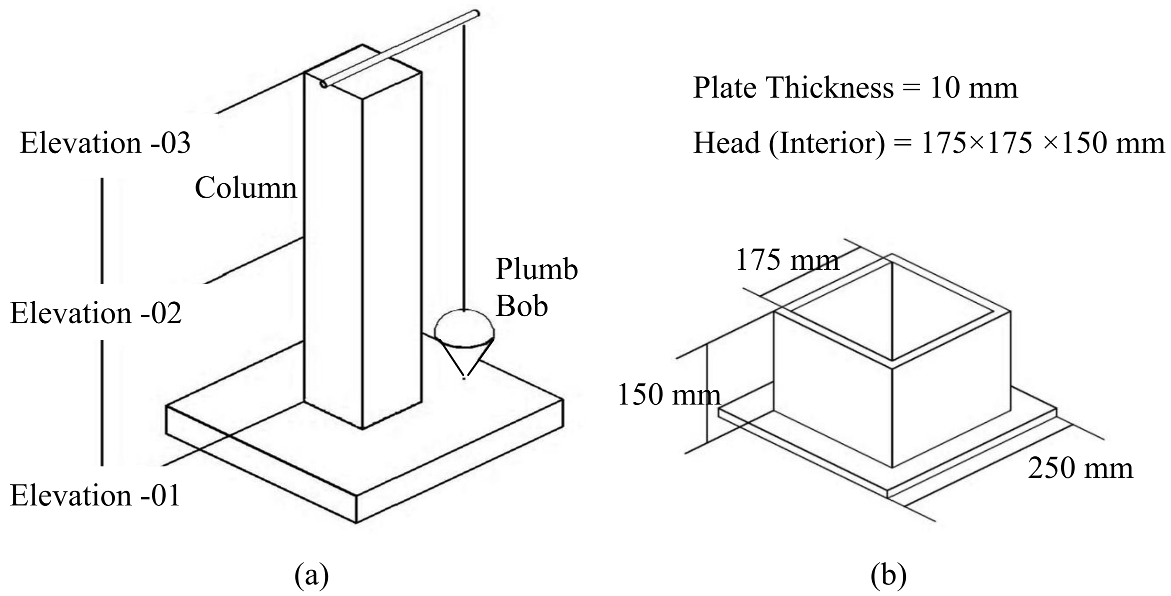


Figure 3.8 Schematic diagrams for, (a) checking of column two-two vertical alignment; and (b) steel head

Similar procedure was used to preload the specimens from NP1.1 to NP4.2 and NS1. They were preloaded concentrically using two steel heads at ends as with Figure 3.9 (c). The detailed dimension of the steel head is sketched in Figure 3.8 (b).

During testing, unstrengthened and strengthened specimens were tested using square thickened plates at ends. The dimension of the plate was 175 × 175 × 10 mm. Figure 3.9 defines the schematic diagram of whole test set up and instrumentation test procedure for unstrengthened column (reference column), preloading of column and strengthened column specimens.

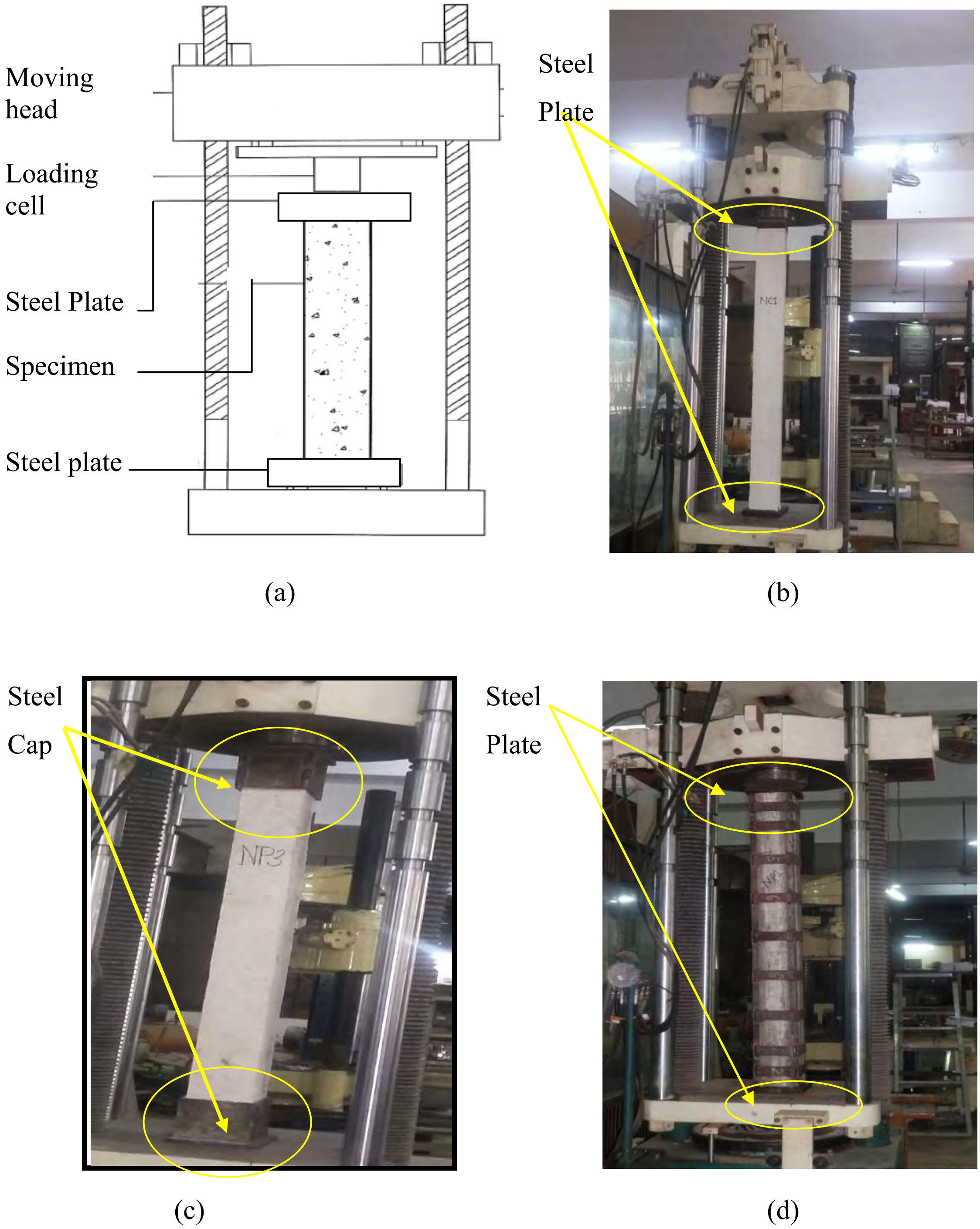


Figure 3.9 General views of test set-up. (a) Schematic diagram of the test set up; (b) Test setup for reference test specimen using steel plates (c) Test setup for preloading stage using steel heads (d) Test setup for strengthened test specimens using plates.

CHAPTER 4
RESULTS & DISCUSSION

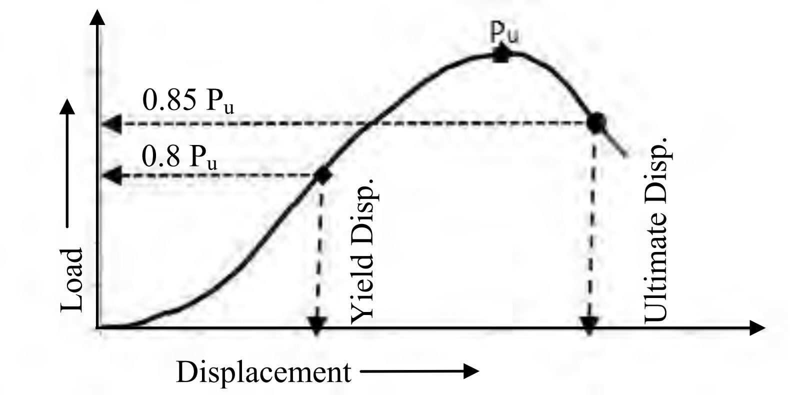
4.1 General

In this study, eight column specimens has been tested in order to investigate the performance of steel angles and strips jacketed strengthened RC columns under concentric axial loads. This chapter gives an outline on observations and failure modes for each test column separately, followed by a summary of the peak test loads and failure modes for strengthened RC columns. Afterwards, a detailed parametric study has been conducted using the test results. The column preloading state before strengthening, variations in steel angle ratio, and spacing of the strips are the main studied parameter in this research. One preloaded column has also been tested accounting the application of partial strengthening method (steel jacketing only at ends). In addition, the experimental results are compared with the capacity prediction models available in the published literatures. Finally, an efficient and simple strengthening method for steel jacketed RC column is proposed based on the results of current study.

4.2 Experimental Results

The outputs results have been extracted from the Universal Testing Machine that records the applied axial loads in Kilo-Newton and the axial shortening distance in millimeter. During the test, the ultimate capacity, load-shortening curves, failure modes of the test specimens have been recorded. It has been found that the strengthened test specimens provide higher compressive strength and improved column ductility than the unstrengthened specimen.

In this study, the column ductility is measured from the displacement ductility index. It is defined as the ratio of the ultimate displacement to the yield displacement as shown in Figure 4.1. Here, the ultimate displacement ($\nabla_{0.85}$) is the displacement corresponding to the 85% of peak axial load located on the post-peak downward branch of the load displacement curve. The yield displacement ($\nabla_{0.80}$) has been used as the displacement related to the 80% of peak axial load on the prior-peak upward branch of that curve. Higher index of displacement ductility indicates higher axial deformation at failure as well as the higher efficient structural performance of the column.



Displacement ductility index, μ = ($\nabla_{0.85}$) / ($\nabla_{0.80}$)

Figure 4.1 Definition of yield and ultimate displacement.

The confined concrete strength of the strengthened column is measured based on the experimental strength of the unstrengthened column. Theoretical angle strength is obtained by the multiplication of angle yield strength and section area. Then, it is deduced from the experimental ultimate strength of the strengthened column. Hence the strength of the confined concrete is calculated. Now the enhancement in confined concrete strength is obtained by dividing the experimental ultimate strength of the unstrengthened column as shown below,

$P_{\text{confined concrete}} = P_u - P_{\text{Angle}}$

Confined concrete strength enhancement $= \dfrac{P_{\text{confined concrete}}}{P_n}$

Where, P_n is the experimental strength of the of the reference column which is not being strengthened.

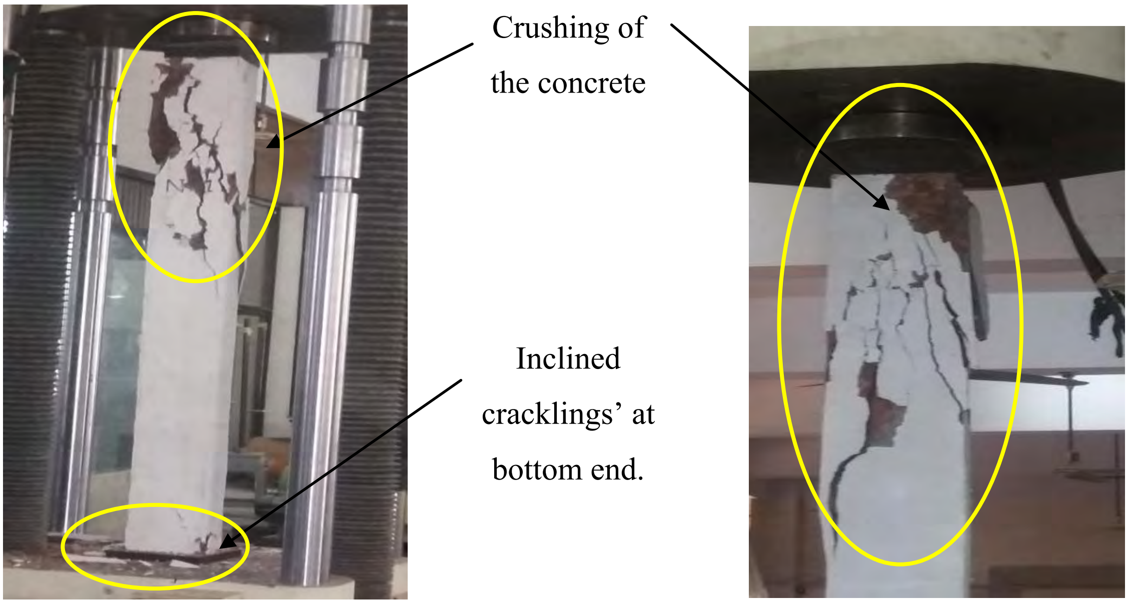
4.3 Observations and Failure Modes

The Universal Testing Machine (UTM) was found to be worked on properly during the pre-test verification of all the column specimens. All the specimens were initially loaded at a stroke rate of 3 mm/min. This rate was maintained throughout the experiment. The photographs were taken following the test. Before the failure load reached, no local failure of the column was observed.

Detailed descriptions of the strength and failure behavior for each test specimens are provided in the following section.

4.3.1 Column NC1

The column NC1 reached its peak load at 457 kN after 185seconds of starting time. By this time, the displacement was 9.2 mm. Following the peak, column's post-peak strength sharply declined to its failure load of 362 kN. The column NC1 test had taken in total 199 seconds till failure. The test stopped at a displacement of 9.9 mm. The failure behavior is illustrated in Figure 4.2.



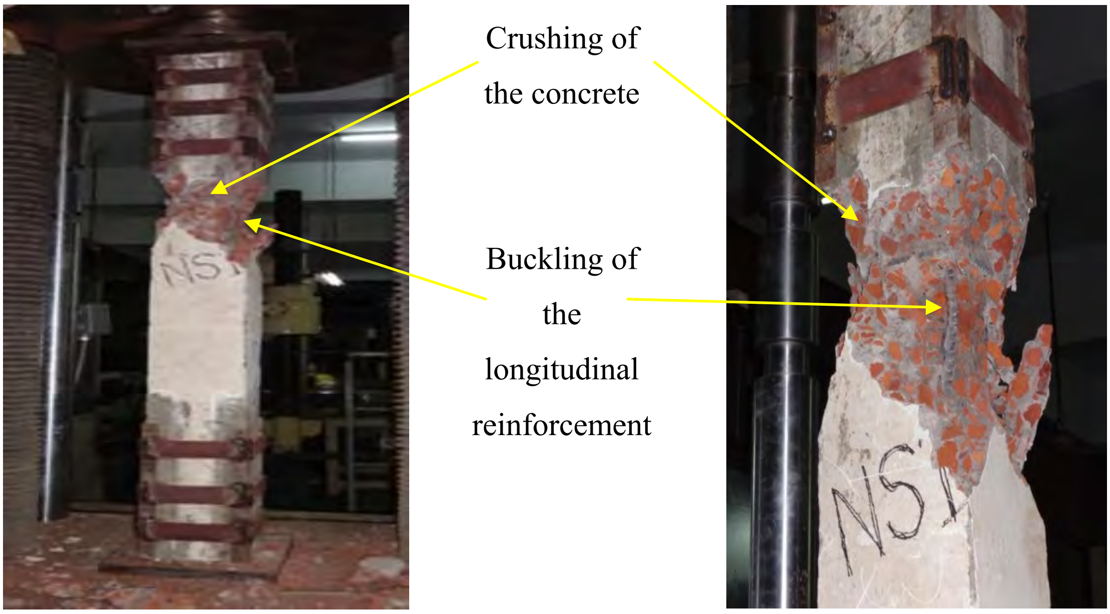
(a) (b)

Figure 4.2 Failure mode of the unstrengthened RC column NC1, (a) Failure shape; and (b) Crushing of the concrete at the back face of the column

Failure of the column was sudden. Failure started with inclined cracks from the upper end of the column. As the load increased, concrete had partially crushed from this end. Then, some thin, vertical cracks were observed to propagate conically along the mid height of the column from this end. As the loading continued, widen of the cracks and spalling of concrete was occurred. At the end of the test, buckling of longitudinal steels was observed followed by the crushing of concrete at the upper end. Some inclined cracks and spalling of concrete was observed at the bottom column end, too.

4.3.2 Column NS1

The column NS1 reached its peak load at 535 kN after 150 seconds of starting time. By this time, the displacement was 7.5 mm. Following the peak load, column's post-peak strength was sharply declined to its failure load 523 kN. The column NS1 test had taken in total 152 seconds till failure. When the test was stopped, the displacement was reached to 7.6 mm. The failure behaviors are illustrated in Figure 4.3.



(a) (b)

Figure 4.3 Failure mode of the partially strengthened column NS1, (a) Failure shape; and (b) Close photograph of the column NS1 shear failure.

Failure of the column was brittle and explosive. The failure was initiated with a large number of cracks at the middle height of the column. Then, they were seen to widen and elongated conically to end of the discontinuous corner angles. As the loads continued, the concrete was crushed, spalled and exploded out with harsh sound. Immediately thereafter, the longitudinal steel bar was observed to buckle very quickly and the column was failed. After the test, the concrete shear planes were visible at the approximately middle height of the column. The fracture of the fillet welds of the horizontal strips did not occurred during testing.

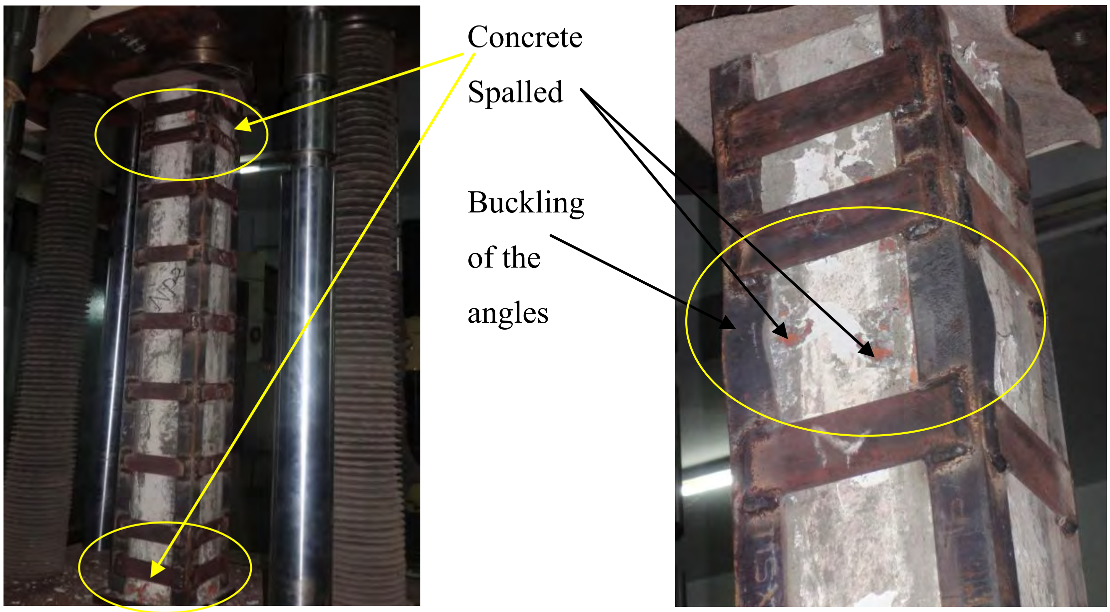
(a) (b)

Figure 4.4 Failure mode of the strengthened RC column NP1.1, (a) Failure shape; and (b) Buckling of the angles and appearance of the longitudinal steel bars

4.3.3 Column NP1.1

The column NP1.1 was almost linear about 72% of its peak load after 184 seconds of starting time. By this time, the displacement was 9.0 mm. After 179 seconds, the column slowly increased to its peak load of 958 kN at 14.6 mm axial displacement. Following the peak, column's post-peak strength was appeared to stabilize up to 956 kN load and after what; it gradually declined to its failure load of 807 kN. The column NP1.1 test had taken in total 362 seconds till failure. When the test was stopped, the displacement was touched to 18.8 mm. The behavior of the column failure is illustrated in Figure 4.4.

Failure of the column was gradual. The failure of the column was started with the local buckling of the angles at the lower end. As the load increased, small cracks were observed to grow and widen at this end. Further increase in loads resulted spalling of concrete cover from this end. At the end of the test, vertical reinforcement bars was observed to expose. A number of the cracks at the upper end were also detected. The splitting out of the welding was continued throughout the test. The failure was found at the bottom end where the repairing method applied.



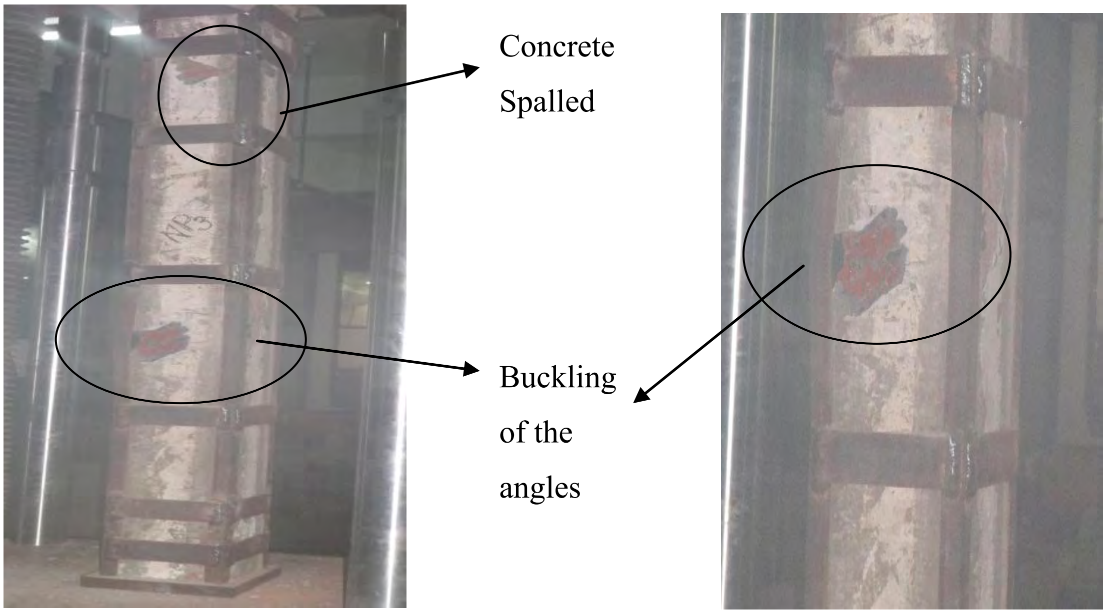
(a) (b)

Figure 4.5 Failure mode of the strengthened column NP2.1, (a) Failure shape; and (b) Buckling of the angles and appearance of concrete cracks.

4.3.4 Column NP2.1

The column NP2.1 was almost linear up to 80% of its peak load. Then the column started to advance non-linearly and reached to its peak load at 1251 kN after 316 seconds of starting time. By this time, the displacement was 16.1 mm. Following the peak, column stabilized up to 14.8 mm axial displacement and at 1246 kN post-peak load, it again started to softened very steadily. At 1140 kN, column prompted to decrease swiftly and finally reached to its failure strength at 955 kN load. The test had taken in total 428 seconds till failure. When the test was stopped, the maximum displacement was achieved at 22 mm. The failure behavior is illustrated in Figure 4.5.

At first, local buckling of vertical steel angles was observed just beyond the upper end zone of the test specimen. Then typical small inclined cracks were appeared and widen as the loads increased gradually. Further increase in loads causing concrete cover to spall off from this part and the specimen was observed to fail afterwards. Splitting of welding was continued all over the test. After the test, concrete spalling was evident at the bottom end of the column specimen. The failure was found at the upper end where the repairing method applied.



(a) (b)

Figure 4.6 Failure mode of the strengthened RC column NP3.2, (a) Failure shape; and (b) Concrete ejection and buckling of the angles.

4.4.5 Column NP3.2

The column NP3.2 was approximately linear up to 55% (470 kN) of its peak load. At this stage, yield was observed and then it started to advance non-linearly. The column reached its peak axial load of 854 kN after 298 seconds of starting time. By this time, the displacement was 15 mm. Following the peak, column's post-peak strength started to softened and rapidly declined. However, column failed at 721 kN load and reached to a displacement of 18.7 mm. This test had taken in total 368 seconds till failure. The locations of interest during the failure are illustrated in Figure 4.6.

Failure of the column was gradual. This failure of the specimen was started with the inclined cracks at upper end and middle of the column, simultaneously. As the load increased, cracks were widened and crushing and spalling of the concrete was occurred. Further increased in load resulted local buckling of the angle followed by the ejection of convex shape concrete piece from the middle of the column. During the test, splitting of the welding was continued. Concrete crushing and spalling was observed beyond the repairing end. During chipping, concrete was slightly damaged accidentally at the mid side of the column. This may be a reason for ejecting of the concrete piece during testing.

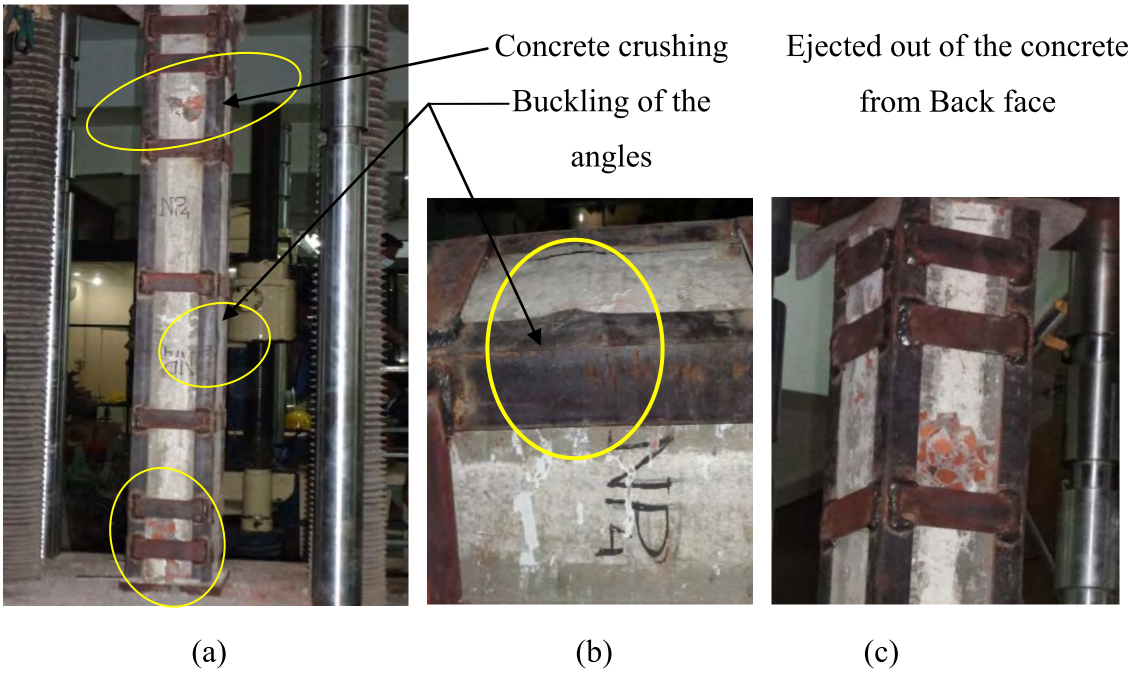


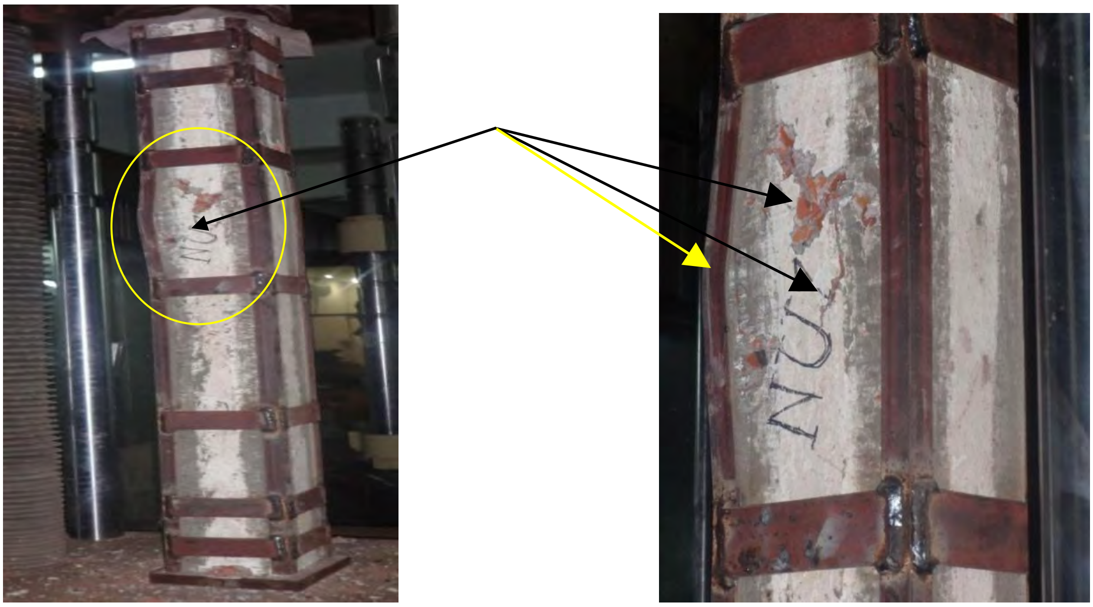
Figure 4.7 Failure mode of the strengthened RC column NP4.2, (a) Failure shape; (b) Buckling of the angle; and (c) Concrete ejection from the back face lower end of the column

4.3.6 Column NP4.2

The column NP4.2 reached its peak axial load of 1072 kN after 336 seconds of starting time. By this time, the displacement was 16.9 mm. Following the peak, column's post-peak strength started to softened and rapidly declined up to 97% of its peak load. Then the column NP4.2 rapidly failed at 876 kN load and reached to displacement of 20.7

mm. This test had taken in total 413 seconds till failure. The locations of interest during the failure are illustrated in Figure 4.7.

Failure of the column was gradual. The failure of this column was initiated with the cracking and spalling of the concrete at both ends. As the load increased, concrete crushed and a large piece of concrete was observed to drive out from the upper end of the back face. Splitting of the welding and light spalling of concrete cover was continued over the test. Following the test, local buckling of the angles was observed. The concrete at the lower end of the column was also observed to crush. In this test, heavy crushing of the concrete was seen to occur.



(a) (b)

Figure 4.8 Failure mode of the strengthened RC column NU3.2, (a) Failure shape; and (b) Crushing and cracking of the concrete and buckling of angles

4.3.7 Column NU3.2

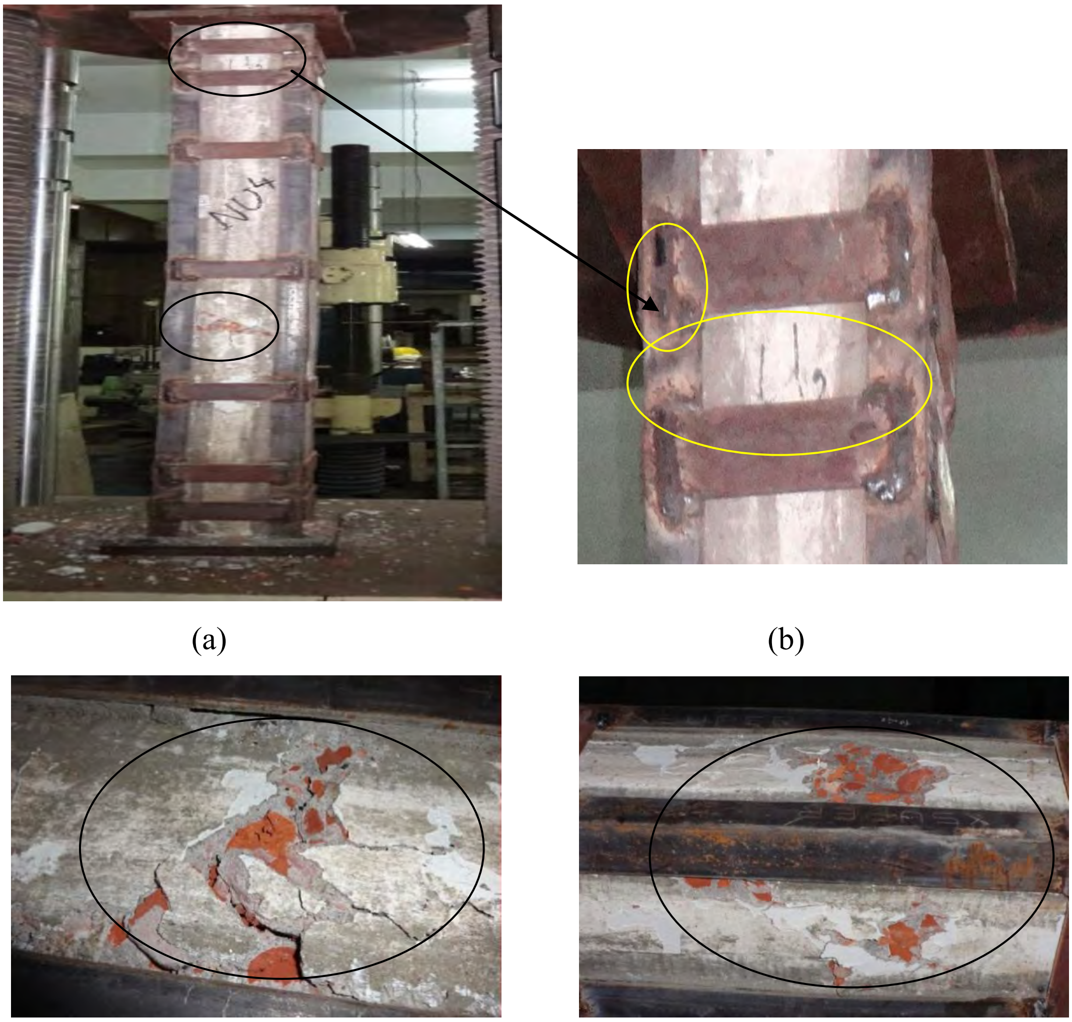
The column NU3.2 reached its peak axial load of 987 kN after 221 seconds of starting time. By this time, the displacement was 11.1 mm. Following the peak, column’s post-peak strength started to soften and a gradual declined observed up to 95% of its peak load. A short stabilization was observed before reaching column at 977 kN load. Afterwards, column rapidly failed at 839 kN load and reached to a displacement of

16 mm. The column NU3.2 test had taken in total 310 seconds till failure. The locations of interest during the failure are illustrated in Figure 4.8. Failure of the column was gradual. This specimen was initiated with the local buckling of the angles at mid height of the column. Then, small cracks were appeared to the middle of the column. As the load increased, cracks were widened and spalling of the concrete appeared which accelerated the column failure. Splitting out of the welding was observed during the testing. Thin vertical crack was also observed at the contact face of the concrete and vertical corner angles after the test.

4.3.8 Column NU4.2

The column NU4.2 was almost linear up to 87% (1138 kN) of its peak axial load. Afterwards, a curvy increasing approach is observed up to peak axial load of 1308 kN. By this time, the test time and the axial shortening was reached at 293 seconds and 14.7 mm, respectively. Following the peak, column's post-peak strength started to softened and rapidly declined to failure. The column failed at 1049 kN load and reached to a displacement of 19.3 mm. The test had taken in total 380 seconds till failure. The locations of interest during the failure are illustrated in Figure 4.9.

Failure of the column was gradual. The failure was started with the typical inclined cracks at the middle of the column. With the increase in load, cracks became widen and causing the concrete cover to spall off. Further increase in load resulted the local buckling in the angles at the upper end and ejection of a piece of concrete from the back face of the column. After the test, fracture of the welding and a thin vertical crack was observed at the contact face between concrete and angles. In this column, the length of the two angles in back side was slightly shorter than the test column. During the cutting of the angles, the workman could not be able to keep all angles equal. For this reason, fracture in the welding connection might take place.



(a) (b)

(c) (d)

Figure 4.9 Failure mode of the strengthened RC column NU4.2, (a) Failure Shape; (b) Welding fracture and local buckling of the angles; (c) Crushing of the concrete at front; and (d) Ejection of the concrete at back.

A summary of the ultimate loads and failure modes for all reinforced concrete columns strengthened using steel jacket is presented in table 4.1. All the columns (NC1, NS1, NP1.1, NP2.1, NP3.2, NP4.2, NU3.2, and NU4.2) were loaded concentrically. All the strengthened columns have been found to change their failure mode from brittle to ductile mode except test column NS1. But all of them are found to increase ultimate compressive strength significantly. Their increases have been varying in range of 17% to 186%. These are certainly a good achievement of this jacketing method. In addition, Table 4.1 shows that the strengthened column reach their ultimate strength in later the unstrengthened column except column NS1.

Table 4.1 Test Load Summary for Concentrically Loaded RC Column

Col.	Failure Mode	Ultimate Load		Displacement at ultimate load		Displacement Ductility Index	
		P_u (kN)	ΔP_u (%)	∇_u (mm)	$\Delta\nabla_u$ (%)	μ	$\Delta\mu$ (%)
NC1	Brittle failure: ➢ Concrete crushing ➢ Buckling of rebars	457	0%	9.2	0%	1.24	-
NS1	Brittle failure: ➢ Concrete crushing ➢ Buckling of rebars	535	17%	7.5	-		-
NP1.1	Ductile failure: ➢ Buckling of angles ➢ Concrete crushing ➢ Visible of rebars	958	110%	14.6	59%	1.90	53%
NP2.1	Ductile failure: ➢ Buckling of angles ➢ Concrete crushing	1251	174%	16.1	75%	2.07	67%
NP3.2	Ductile failure: ➢ Buckling of angles ➢ Concrete crushing and ejection	854	87%	15.0	63%	1.62	31%
NP4.2	Ductile failure: ➢ Buckling of angles ➢ Concrete crushing and ejection	1072	135%	16.9	84%	1.68	35%
NU3.2	Ductile failure: ➢ Buckling of angles ➢ Concrete crushing	987	116%	11.1	21%	1.93	56%
NU4.2	Ductile failure: ➢ Buckling of angles ➢ Concrete crushing ➢ Fracture of welding	1308	186%	14.7	60%	2.02	63%

4.4 Parametric Study

A detailed parametric study has been conducted in this section. The variable parameters used in this research are steel angle ratio, strips spacing, 70% preloading state before strengthening and partial strengthening method at column ends. The dimension of the unstrengthened RC column was 150 ×150 ×1500 mm. The longitudinal rebar ratio was 1.4%. The compressive strength of the concrete used in RC column was 15.2 MPa. The strength of the cement grout used between steel angle and concrete surface was 16.3 MPa.

4.4.1 Effects of Steel Angle Ratio

Two different angle sizes (L25 × 5 and L40 × 5) have been used in the current experimental investigation resulting in 4% and 7% steel angle ratio, respectively. To investigate the effects of the steel angle ratio test columns designated as NP1.1, NP3.2, NP2.1 and NP4.2 has been selected. However, these four columns have two different horizontal strip spacing to column depth ratio. Column NP1.1 and NP2.1 has a strip spacing ratio of 1d whereas column NP3.2 and column NP4.2 has a larger strip spacing ratio of 2d. Based on the strip spacing ratio the columns have been divided in two sets as shown in Table 4.2. These columns were preloaded at 70% of the ultimate capacity before strengthening. Effects of steel angle ratio with respect to axial load shortening behavior, ultimate strength and displacement ductility are discussed below.

Figure 4.10 shows the load deflection curve of column NP1.1, NP3.2, NP2.1 and NP4.2 due to the effect of steel angle ratio. This figure confirmed that the higher ultimate axial strength has been achieved for the column strengthened with higher angle steel ratio (7%) as expected in both sets. Generally, the primary objective of providing angles is to support concrete against the axial compressive loads coming on the column. Again, under axial compressive loads, composite action between angle and concrete occurs in the column resulting in higher load carrying capacity. When the steel ratio of the angle is increased, induction of composite actions became more prominent and showed ability to resists compressive load more efficiently.

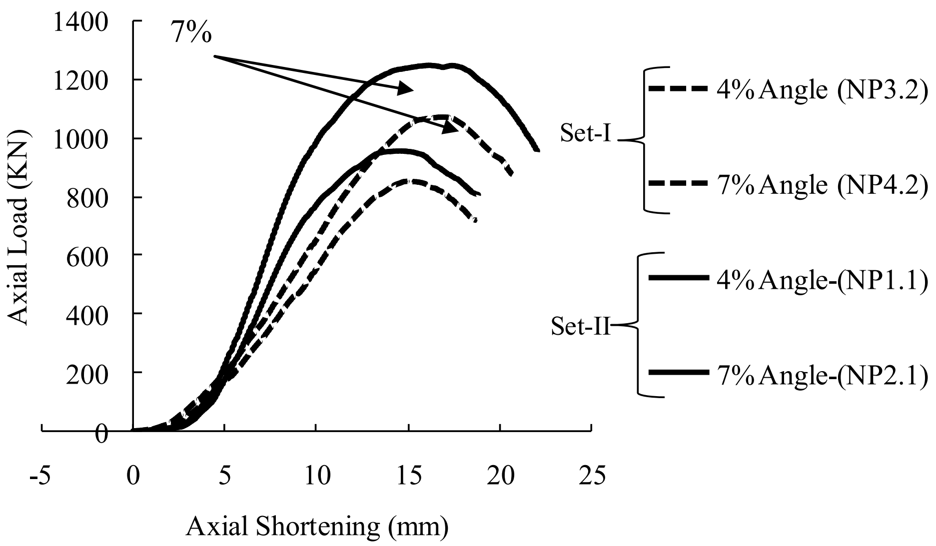


Figure 4.10 Effects of steel angle ratio

Figure 4.10 also shows that columns with 7% steel angle ratio have relatively greater stiffness in both sets. It can be explained from the concept of concrete confinement. The angles provide longitudinal confinement from the corner of the column uniformly. When larger size of angle is used, it exerts somewhat greater confinement pressure from the enlarged portion of the angle area. For this reason, area of shear stress transmission due to the friction is also increased. This frictional shear stress resists the concrete deformation (Poisson's ratio) laterally. The lesser the lateral deformation, the higher is bending moment on the steel angles. For this reason, columns with 7% steel angle ratio is seen to have comparatively higher resistance against concrete lateral deformation plus bending moments and exhibits higher initial stiffness under axial loads. Moreover, columns strengthened with 7% angle ratio have maximum axial shortening and reached their ultimate strength later (as shown in Figure 4.10). Certainly this is an indication of good ductility.

Table 4.2 demonstrates that the effects of angle ratio on ultimate strength, displacement ductility and confined concrete strength enhancement of the strengthened column. It has been seen from Table 4.2 that change in the steel angle ratio from 4% to 7% can enhance the ultimate strength from 87% to 135% and 110% to 174% (with respect to the reference RC column) for S=2d and S=1d strip spacing, respectively. This is undoubtedly a significant strength improvement. In this study, steel angle ratio is found to have the highest contribution on gaining ultimate and confined concrete strength of the reference column among the selected parameters.

Table 4.2: Effects of steel angle ratio

SET	Column	Angle steel ratio	Increment		μ	Δμ(%)	$\frac{P_{angles}}{P_n}$	$\frac{P_{concrete}}{P_n}$
			ΔP_u	$\Delta \nabla_u$				
I	NP3.2	4%	87%	63%	1.62	31%	0.71	1.16
	NP4.2	7%	135%	84%	1.68	35%	1.03	1.31
II	NP1.1	4%	110%	59%	1.90	53%	0.71	1.39
	NP2.1	7%	174%	75%	2.07	67%	1.03	1.70

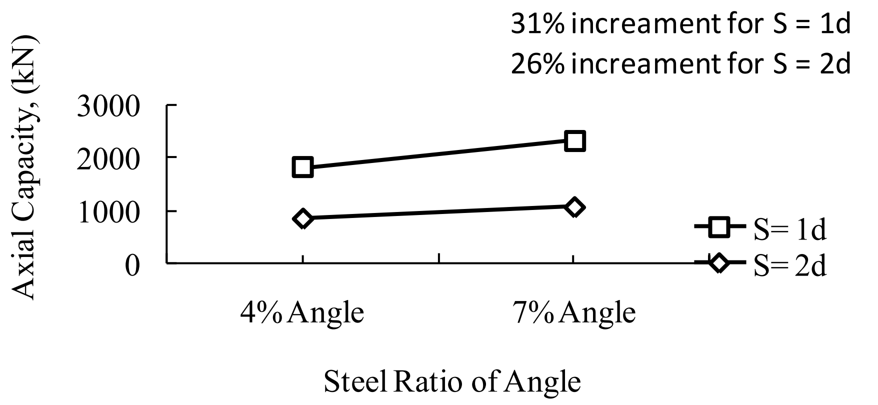


Figure 4.11 Effects of steel angle ratio on ultimate compressive strength

As in Figure 4.11, for S=2d strip spacing, 26% higher ultimate axial strength has been seen to be achieved when steel angle ratio is increased from 4% to 7%.On the other hand, for the same increase in steel angle ratio 31% strength gain was observed for columns with closer strip spacing (S= 1d). Therefore, higher strength gain is observed due to the increase in steel ratio for column with S= 1d strip spacing as compared to the column with relatively wider strip spacing (S=2d).

Figure 4.12 shows the variation of the displacement ductility index due to the effect of steel angle ratio. From this figure, it is evident that column NP4.2 and NP2.1achieved higher displacement ductility when they were strengthened with 7% angle ratio (Table 4.4). For S=2d strip spacing, column ductility index is found to increase from 31% to

35% for the increase in angle ratio from 4% to 7%. However, this increase in ductility index has been found to be higher (53% to 67%) when a closer strip spacing of 1d was used in strengthened column. Increase in the steel angle ratio does not have significant effect on the ductility index of the RC column.

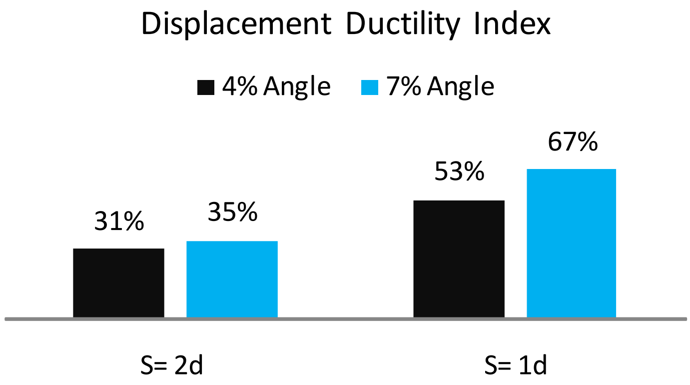


Figure 4.12 Effects of steel angle ratio on displacement ductility

In Figure 4.13, strength of confined concrete was also found to increase from 16% to 31% and 39% to 70% for this variation of steel angle ratio. Greater strength increment of 23% has been noticed in SET-I for closer strip spacing of 1d. In case of wider strip spacing (2d) in SET-II, the strength increment was found to be 13% only.

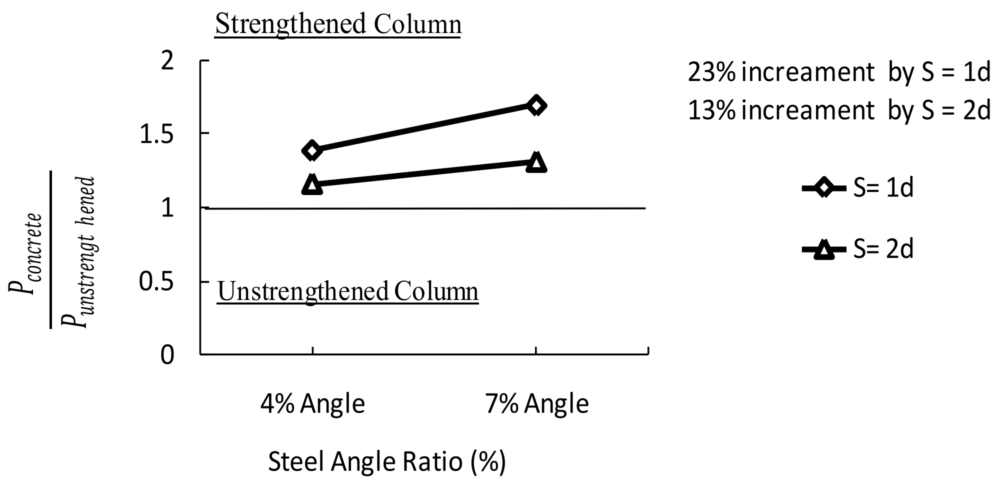


Figure 4.13 Effects of steel angle ratio on confined concrete strength.

4.4.2 Effects of Horizontal Strips Spacing

Two different strip spacing (150 mm and 300 mm) has been used in the current experimental investigation resulting in S=1d and S=2d, respectively, where d is the length of the column. To investigate the effects of this parameter test columns designated as NP1.1, NP2.1, NP3.2 and NP4.2 has been selected. However, these four columns have two different steel angle ratios. Column NP1.1 and NP3.2 has an angle steel ratio of 4% whereas column NP2.1 and column NP4.2 has a larger angle steel ratio of 7%. Based on the steel angle ratio, the columns have been divided in two sets as shown in Table 4.3. Effects of strip spacing with respect to axial load shortening behavior, ultimate strength and displacement ductility of steel jacketed RC columns are presented below.

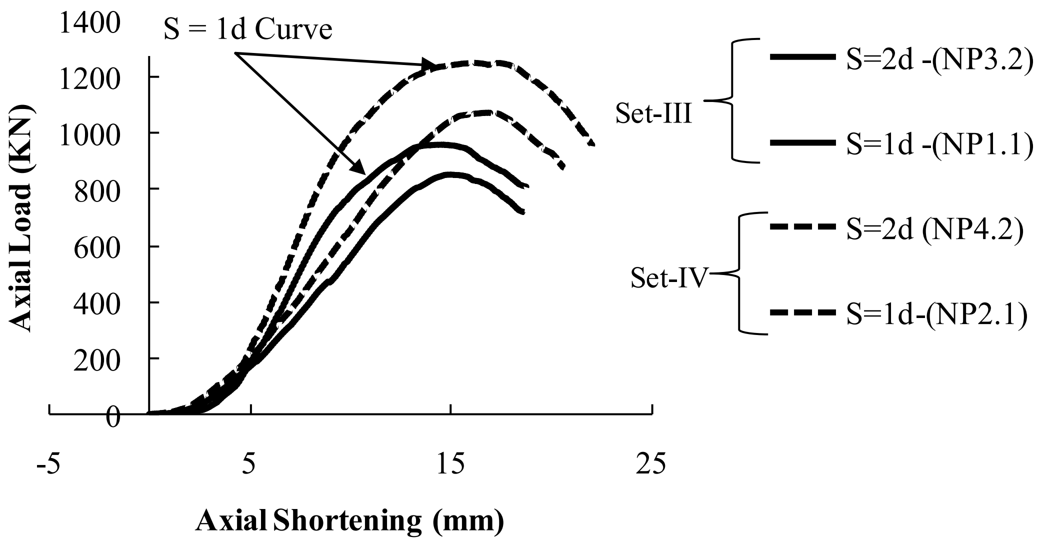


Figure 4.14 Effects of horizontal strips spacing

Figure 4.14 shows that higher ultimate compressive strength with test columns having S=1d strip spacing has been achieved in both sets. Generally, the main objective of providing horizontal strip has been to resist the concrete lateral expansion under compressive axial loads and to increase the overall ductility of the column.

From the confinement concept, the horizontal strips usually apply non-uniform lateral confinement across the length of the column since they have a discrete longitudinal distribution. The place where the strips are located, zone of stiffness is highest in there. Again, this zone of stiffness is increased with the decreased spacing of horizontal steel strips. For this reason, column NP1.1 and NP2.1 which had a closer strip spacing of S=1d are seen to have steeper slope in the load vs. axial shortening curve indicating comparatively higher stiffness than the columns with S=2d strip spacing. They have also seen to reach their ultimate strength earlier.

Table 4.3: Effects of horizontal strips spacing

SET	Column	S	Increment		μ	Δμ(%)	$\frac{P_{angles}}{P_n}$	$\frac{P_{concrete}}{P_n}$
			ΔP_u	$\Delta \nabla_u$				
III	NP3.2	2d	87%	63%	1.62	31%	0.71	1.16
	NP1.1	1d	110%	59%	1.90	53%	0.71	1.39
IV	NP4.2	2d	135%	84%	1.68	35%	1.03	1.31
	NP2.1	1d	174%	75%	2.07	67%	1.03	1.70

Again, from the continuous beam concept, the horizontal strips treat the vertical angle as continuous beam by providing elastic-plastic lateral support throughout the length. When the length between two strips is shortened, the length of the continuous beam is also shortened. As a consequence, comparatively shorter length of continuous beam offers higher resistance against the coupling action of angle buckling and bending between two strips under compressive loads. This is also another reason for achieving higher level of compressive strength with columns NP1.1 and NP2.1 having S=1d strip spacing which is evident in Figure 4.14. In addition, maximum axial shortening is achieved with these columns resulting in higher ductility index as shown in Table 4.3.

Table 4.3 illustrates that the strength of the column has been increased from 87% to 110% for the increase in strip spacing from 2d (300 mm) to 1d (150 mm). This is for the columns that have been strengthened with 4% steel angle ratio. For the column with 7% steel angle ratio, the axial capacity has been found to be enhanced from 135% to 174%. Undoubtedly, a very good strength improvement is obtained from the reduction in the strip spacing. Figure 4.15 also shows the strength enhancements for the decrease in strip spacing from 2d to 1d. It is clear from this figure that columns strengthened with 4% steel angle ratio enhanced the ultimate strength by 12%. This enhancement in column capacity was found to be to 17% when they were strengthened with 7% steel angle ratio.

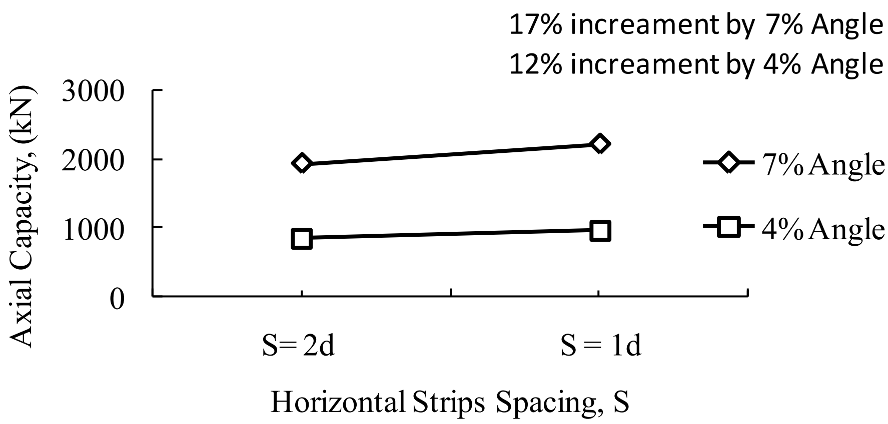


Figure 4.15 Effects of horizontal strips spacing on ultimate strength

In Figure 4.16, displacement ductility index of the columns strengthened with 4% steel angle ratio has been found to be enhanced from 31% to 53% for the decrease in strip spacing from 2d to 1d. With 7% angle ratio, this improvement has been obtained from 35% to 67%. These are certainly a significant improvement of column ductility with the reduction in the strip spacing. In this study, this parameter provides highest contribution in gaining maximum displacement column ductility as expected.

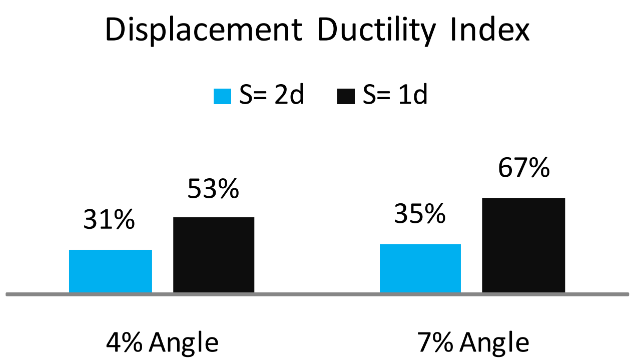


Figure 4.16 Effects of horizontal strips spacing on displacement ductility index

Figure 4.17 shows the graphical representation of confined concrete strength enhancement due to the increase in strip spacing from 2d to 1d. From this curve, confined concrete strength is found to increase from 16% to 39% for 4% and 31% to

70% for 7% steel angle ratio. Figure 4.18 also shows that, from 2d to 1d strip spacing, the confined concrete strength increases by 20% and 30% for 4% and 7% steel angle ratio, respectively.

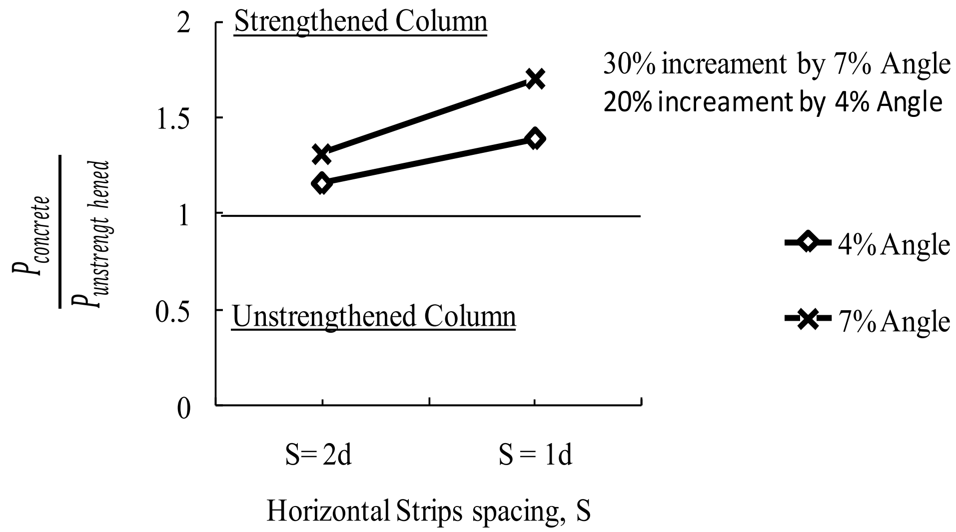


Figure 4.17 Effects of horizontal strips spacing on confined concrete strength

From the above discussion, it can be concluded that, reduction in strip spacing from 2d to 1d with a combination of 4% steel angle ratio, not only increases the ultimate compressive strength but also improves the column ductility and confined concrete strength of the column significantly. These improvements are found to increase as the steel angle ratio is increased to 7%. This is due to the fact that higher confinement of concrete is obtained with higher steel angle ratio as well as with closer strip spacing.

4.4.3 Effects of 70% Preloading State Prior Strengthening

In order to investigate the influence of preloading state, results of four columns (NP3.2, NP4.2, NU3.2, and NU4.2) are presented by organizing them in two sets as SET-V, and VI. The columns NP3.2 and NP4.2 were preloaded up to 70% of their ultimate load and then the resulting damage of the columns were repaired before strengthening. Columns NP3.2 and NU3.2 are kept in SET-V and Columns NP4.2 and NU4.2 are kept in SET-VI as shown in Table 4.5. SET-V and VI has been strengthened with 4% and 7% steel angle ratios, respectively. The strip spacing ratio was 1d for all the four columns. Effects of 70% preloading state (prior strengthening) on the axial load shortening behavior, ultimate strength and displacement ductility of strengthened column with respect to bare concrete column is studied and presented below.

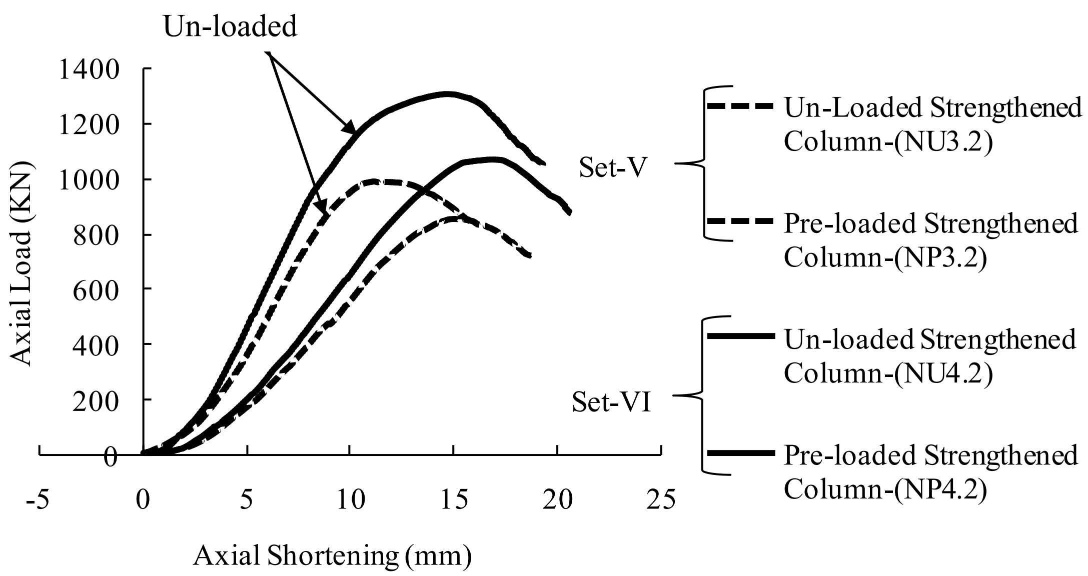


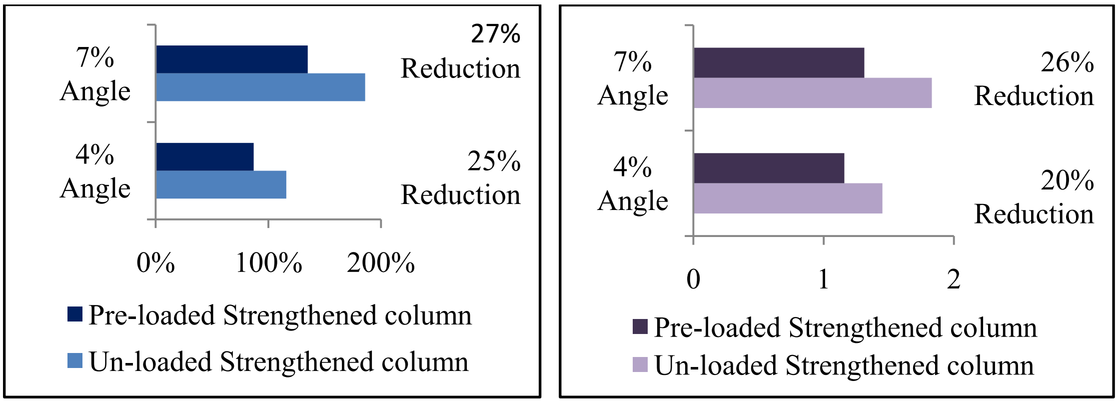
Figure 4.18 Effects of columns with 70% Preloading State

It can be demonstrated from the above axial load vs. displacement curves of Figure 4.18 that the columns with 70% preloading state results in a relatively lower ultimate strength and lesser initial stiffness of the column. Two reasons can be attributed to this behavior of preloaded strengthened column. First, improper load transmission across the column cross section resulting from the damage accumulation during the preloading phase. This damage accumulation results in reduced cross sectional area due to the production of micro cracks along the outer surface of the column. Second, loss of column concrete intactness due to preloading. Since the preloaded columns are axially shortened under axial loads, as per Poisons ratio, they simultaneously induce concrete lateral expansion as well as transverse deformation inside of each fiber in them. This further results in degradation of the strength and stiffness of the strengthened column.

Table 4.4: Effects of 70% Pre-loading State

SET	Column	Load state	Increment		μ	Δμ(%)	$\frac{P_{angles}}{P_n}$	$\frac{P_{concrete}}{P_n}$
			ΔP_u	$\Delta \nabla_u$				
V	NU3.2	Unloaded	116%	21%	1.93	56%	0.71	1.45
	NP3.2	Preloaded	87%	63%	1.62	31%	0.71	1.16
VI	NU4.2	Unloaded	186%	60%	2.02	63%	1.03	1.83
	NP4.2	Preloaded	135%	84%	1.68	35%	1.03	1.31

Again, the earlier deformed concrete section will not be able to resist deformation further as much as undeformed concrete section under same axial loads. For this reason, deformed concrete section goes for higher axial shortening and lower compressive strength which is evident in Figure 4.18 for both column sets. Also, their peak points are also noticed to shift from right to left of the curve. It indicates that preloaded columns reach their ultimate strengths comparatively earlier.



(a) Ultimate strength reduction (b) Confined concrete strength reduction

Figure 4.19 Column strength reductions due to the effects of 70% preloading state

However, SET-VI column are found to show relatively higher strength with higher initial stiffness than SET-V. This is due to the fact that SET-VI columns have been strengthened with 75% higher steel angle ratio which provides basically higher resistance against the applied compressive axial loads directly.

Table 4.4 presents the percent increase in axial capacity, corresponding deflection and ductility index of the preloaded strengthened column with respect to bare concrete column. From this table, it has been found that for 4% and 7% steel angle ratios, the capacity of the unloaded columns decreased from 116% to 87% and 186% to 135%, respectively due to 70% preloading effect. In Figure 4.19(a), this decrease in the ultimate strength has been found 25% and 27%for steel angle ratio of 4% and 7%, respectively. These are certainly a significant strength reduction of the strengthened column due to preloading effect. In this study, the strength reduction has been found to be higher for column strengthened with higher steel angle ratio.

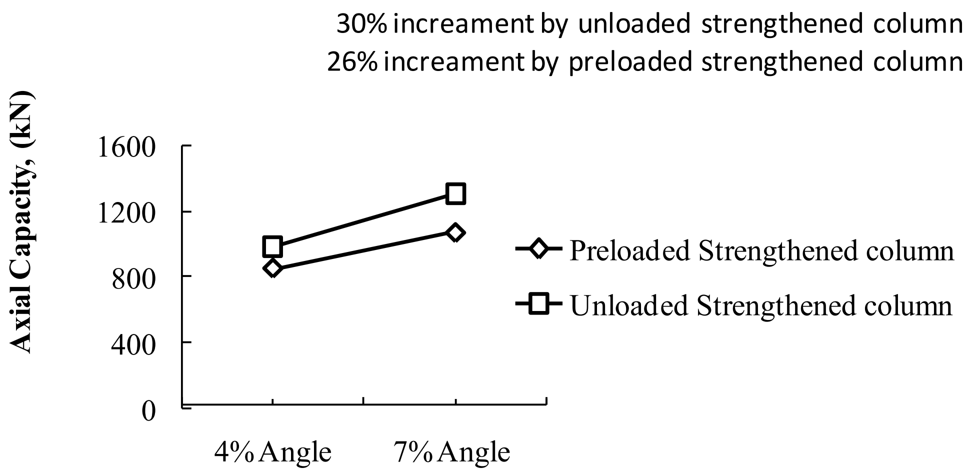


Figure 4.20 Effects 70% preloading state on ultimate strength

Figure 4.20 shows the effects of steel angle ratio on the axial capacity of unloaded and preloaded columns. The columns with no preloading increased the capacity by 30% for the increase in steel angle ratio from 4% to 7% whereas for column with 70% preloading state enhanced the capacity by 26% for the same variation in steel angle ratio. Therefore, effect of steel angle ratio is reduced by 4% due to the column preloading condition. The effect of steel angle ratio on concrete confinement for the both preloaded and unloaded column is shown in Figure 4.21. In this figure it is clear that for the preloaded column increase in concrete confinement is found to be 13% as the steel angle ratio is increased (from 4% to 7%) whereas this increment is obtained to be 23% for the unloaded columns. Therefore, preloading to the RC column has significant effect on concrete confinement of the strengthened column.

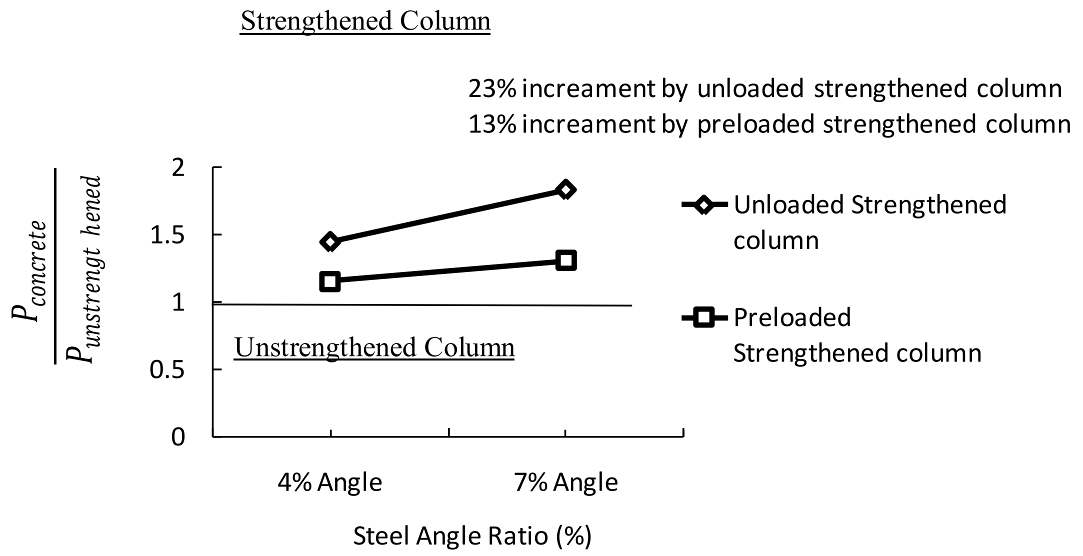


Figure 4.21 Effects of 70% preloading state on confined concrete strength

Ductility of the column preloaded columns has been investigated from displacement ductility index. The values of column displacement ductility are presented in Table 4.3. Unloaded strengthened RC columns has been found to show improved displacement ductility in this study although these columns showed lower axial deformation at peak point as compared to the preloaded columns. Due to 70% preloading condition the displacement ductility of unloaded strengthened RC is found to be reduced by 17%.

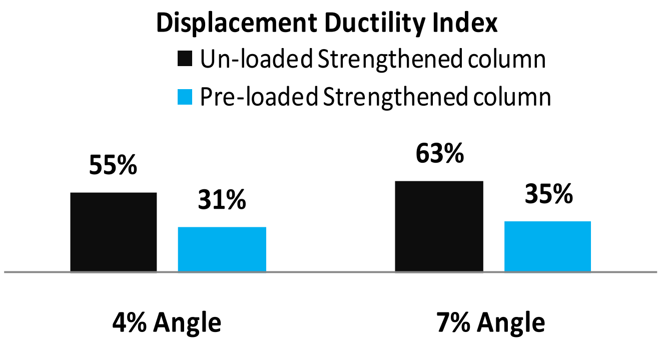


Figure 4.22 Effects 70% preloading state on displacement ductility index

4.4.4 Effects of Partial Strengthening Method at Column Ends

Test column NS1 has been allotted for weighing the effects of partial strengthening at column ends on preloaded columns. This column has been strengthened by keeping the length of the steel angles up to one fourth of column length from both ends. Three discrete horizontal strips have been fixed at a spacing of 75 mm at each face of the column by welding. This column has been tested to study the effects of steel jacketing only at ends at the column to satisfy the seismic tie spacing requirements. The results and analysis of this column based on axial load shortening behavior, strength and displacement ductility are presented below.

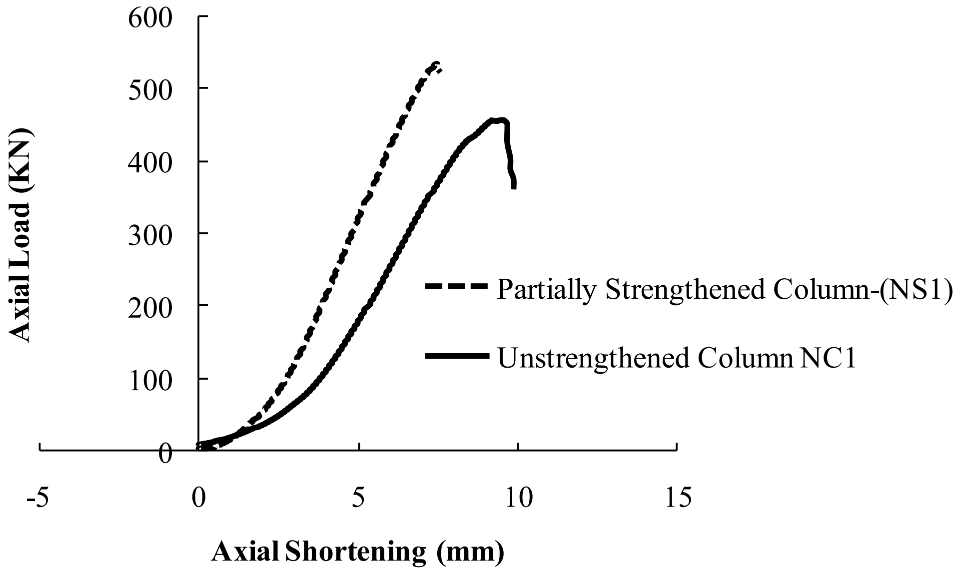


Figure 4.23 Effects of partial strengthening method at column ends

The load vs. axial shortening curve for unstrengthened column NC1 and partially strengthened column NS1 is shown in Figure 4.23. From this figure it is clear that partially strengthening method, i.e. providing steel jackets only at the column ends (quarter of the length of the column) increases the capacity and initial stiffness of the column. However, the displacement at the peak load for column NS1 is observed to be lower (18%) than NC1. Moreover, during the test column NS1 showed brittle failure behavior as compared to column NC1. It is to be mentioned that column NS1 has been preloaded approximately 70% of its capacity before the application of the strengthening scheme. The absent of the angle at the middle half of the strengthened column accelerated the sudden failure of column NS1 as compared the failure of column NC1. For this reason, it can be said that this partial strengthening method is not efficient since it does not improve the ductility of the unstrengthened column at all. However, application of partial strengthening method at ends can enhance the compressive strength of the unstrengthened column by 17% as shown in Table 4.5.

Table 4.5: Effects of partial strengthening method at column ends

Col. Set	Col.	Length of the Angle (each face)	No of Strips (each face)	Increase in Ultimate load	Displacement at Peak Axial Load, (mm)	μ
SET-VII	NC1	-	-	0%	9.2	1.24
	NS1	750 mm	3 at top 3 at bottom	17%	7.5	-

4.5 Performance of the Capacity Prediction Models

The theoretical capacities of the strengthened RC test columns has been predicted using the capacity prediction models proposed by Calderon et al (2009), Tarabia and Albakry (2014), Campoine (2012), ACI 318 M-99 (1999) and Euro Code (1994) as described in chapter two. Then, the predicted axial capacity has been compared with experimental results as shown in Tables 4.6 and 4.7. The experimentally obtained axial capacity for the strengthened columns has been found to be higher than the predicted capacities.

The capacity of the test columns predicted by ACI 318 M-99 (1999) code is found to be 711 kN and 860 kN for 4% and 7% angle ratio, respectively. ACI 318 M-99 (1999) code does not take into account the effect of strip spacing and preloading state of the column in predicting the axial capacity of steel jacketed RC columns. As shown in Table 4.7, as the steel ratio increase, the difference between codes predicted value and experimental value increases. Moreover, increase in the strip spacing results in reduction in the difference between code predicted value and the experimental value. This is due to the fact that as the strip spacing increases the effect of confinement reduces which is however neglected in the code. For preloaded column experimental values are still much higher (around 30%) for 4% angle ratio and 40% for 7% angle ratio than the code predicted values.

Table 4.6: Theoretical capacities from different capacity prediction models

References		Ultimate Capacity of Strengthened Column (kN)					
		Angle Steel Ratio = 4%			Angle Steel Ratio = 7%		
		NP1.1	NP3.2	NU3.2	NP2.1	NP4.2	NU4.2
1.	ACI -318-M-(99) (1999)	711	711	711	860	860	860
2.	Euro-Code 4 (1994)	751	751	751	895	895	895
3.	Calderon*et al.*(2009)	515	485	485	646	589	589
4.	Campione (2012)	671	653	653	746	725	725
5.	Tarabia and Albakry(2014)	785	769	769	929	913	913

Table 4.7: Comparison between experimental results and capacity prediction model

Angle steel ratio	Column	Exp.(kN)	Capacity prediction models				
			$\frac{P_{EXP}}{P_{ACI}}$	$\frac{P_{EXP}}{P_{EURO}}$	$\frac{P_{EXP}}{P_{Cald}}$	$\frac{P_{EXP}}{P_{Cam}}$	$\frac{P_{EXP}}{P_{Tara}}$
4%	NP1.1	958	1.35	1.28	1.86	1.43	1.22
	NP3.2	854	1.2	1.14	1.76	1.30	1.11
	NU3.2	987	1.39	1.31	2.04	1.51	1.28
7%	NP2.1	1251	1.45	1.4	1.94	1.67	1.34
	NP4.2	1072	1.25	1.2	1.82	1.48	1.17
	NU4.2	1308	1.52	1.46	2.22	1.80	1.43

The capacity prediction model used in Euro code 4 (1994) is similar to ACI 318 M-99 (1999) except that Euro code 4 (1994) uses A_g (gross concrete area) instead of A_c (net concrete area) used in ACI 318 M-99 (1999). The details of the capacity prediction models are included in Section 2.6 of Chapter 2. The capacities of the test column predicted by Euro code 4 (1994) are observed to be higher than ACI code predicted values. The load carrying capacity of steel jacketed RC strengthened column is calculated here by the sum of the contribution of unconfined concrete core and area of the steel angles. The effect of strip spacing and preloading state of the column is not considered in the code. The confinement effect is explicitly ignored in this code. As the steel ratio increases, the difference between experimental and code predicted values increases. This is due to the increase in confinement of concrete due to the higher angle size. However, Euro code 4 (1994) neglects the effect of confinement in capacity prediction for jacketed RC columns.

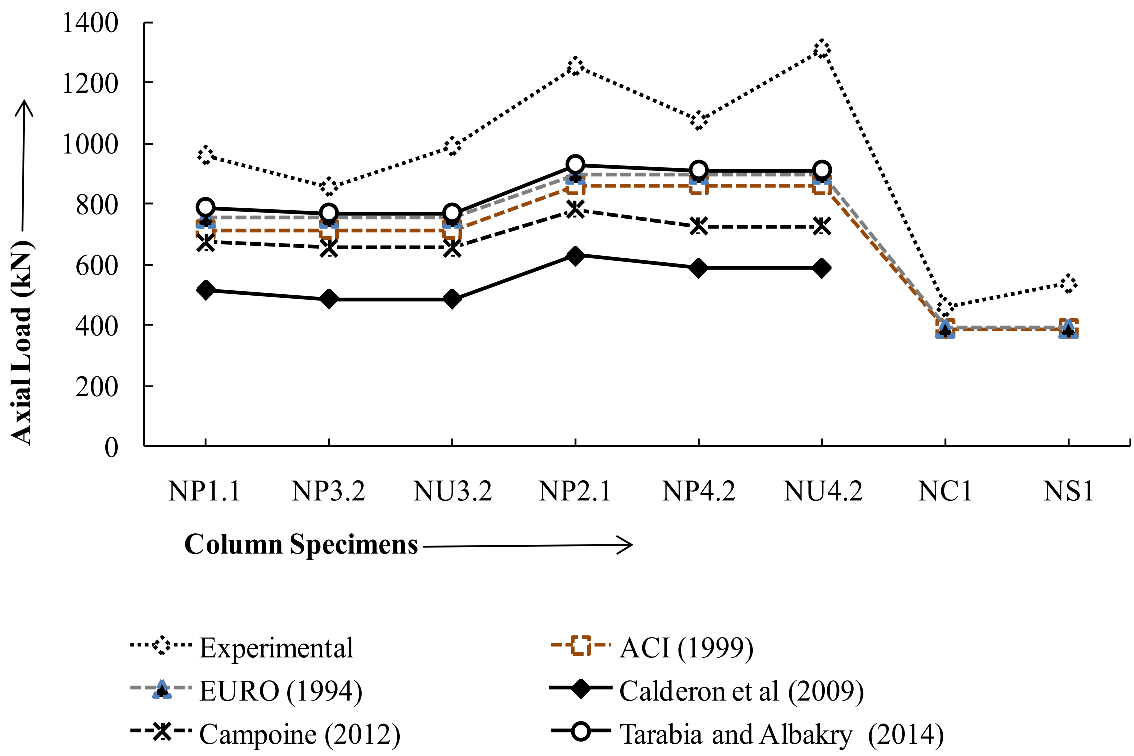


Figure 4.24 Comparison between capacity prediction models and experimental capacities.

Tarabia and Albakry (2014), Campoine (2012) and Calderon et al (2009) proposed capacity prediction models show good agreements with test results as compared to other models. They accounted the confining effects of the outer steel case in their models. The confinements due to the stirrups and deformation due to the bending are neglected in these models. Among these models, the model proposed by Tarabia and Albakry (2014) has been found to be less conservative as compared to other models.

Figure 4.24 shows that, this model has the lowest test to predicted load ratio for column NP3.2 and NP4.2. These columns have higher strip spacing as compared to other columns and have been preloaded. Figure 4.24 also show that the least results have been confirmed with Calderon (2009) Model. This model evaluates the contribution of the vertical angles through strain compatibility and friction but it does not consider the direct or indirect loading case as discussed in Chapter 2. The Campione (2012) Model also presents quite closer and safer values of ultimate strength to the experiment than Calderon since the proposed model accounted the effects of both direct loading and indirect loading case.

4.6 Efficient Strengthening Scheme

Column strengthening is a technical invention that improves the global behavior as well as the structural integrity by optimizing the strength, stiffness and ductility. Strength of a column is generated from its dimension, shape, materials and degree of axial loads resistant etc. Ductility of a column is generated from good detailing, materials used, degree of lateral loads resistant etc. Due to the variety of column deficiencies, it’s hard to develop typical rules for strengthening. For this purpose, an attempt is carried out for approaching an efficient strengthening method which will fulfill the satisfied level of minimum requirements such as strength, stiffness and ductility. The chronological order that has been followed in this section in selecting of a better strengthening method is described below.

At first, a comparison is made by unstrengthened column NC1 with selected configuration of strengthened column NU3.2 (Figure 4.25). No initial loading was applied to these columns. However, an expected improvement in strength and ductility is observed from the curve in Figure 4.27 under compressive axial loads. The strengthening configuration of this column consists of 4% steel angle ratio and 2d horizontal steel strips spacing. It is the bearing column on which a number of comparisons will be made in after for justifying which strengthening method results most similar behavior when the column being preloaded.

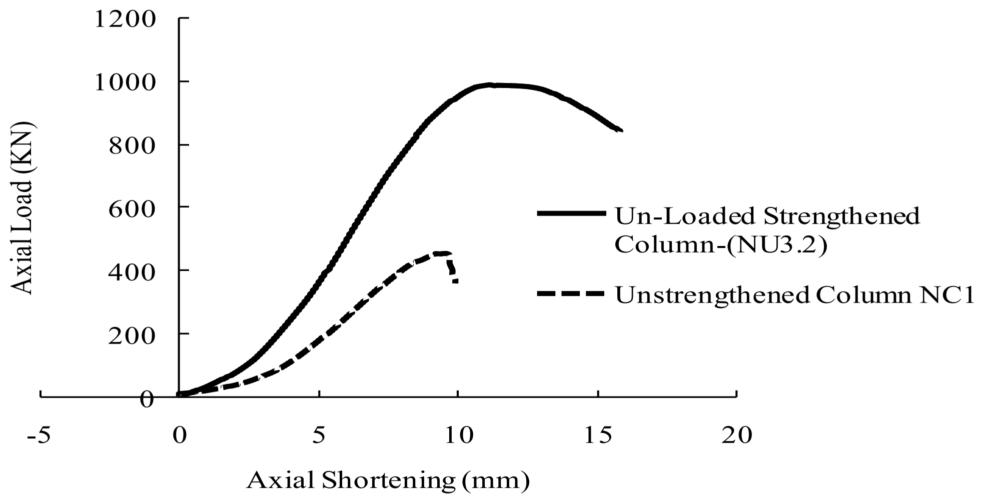


Figure 4.25 Axial Load – Shortening curve of unstrengthened and strengthened column.

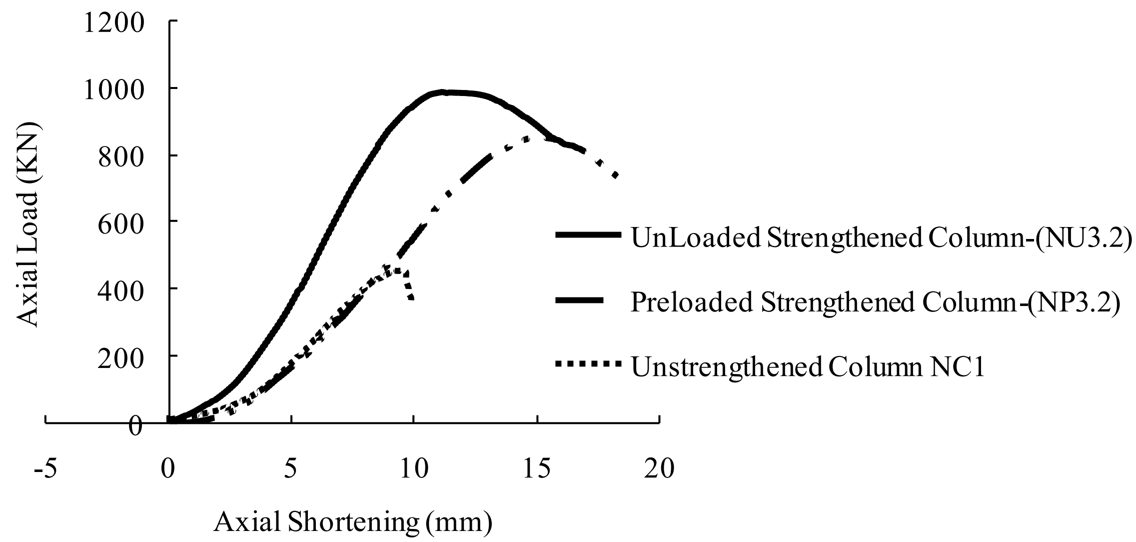


Figure 4.26 Degradation of the strengthened column behavior due to preloading before strengthening

Second, the load-shortening behavior of the initially unloaded bearing column NU3.2 is scrutinized (Figure 4.26) when the column is initially preloaded NP3.2 by 70% of ultimate load under similar configuration. It is seen that the strengthening of the initially preloaded column provides very good strength and ductility as compared to the unstrengthened column. However, the preloaded strengthened column has lower strength, stiffness and ductility as compared to the column with no preloading. Test column NP3.2 had a steel angle ratio of 4% and strip spacing of 2d. Now in this specimen when the steel angle ratio is increased to 7% with the similar strip spacing (which is column NP4.2) significant improvement in the load versus axial shortening curve is obtained (Figure 4.27). The strength and ductility of column NP4.2 is close to the strength and ductility of the unloaded strengthened column (NU3.2). The strengthening scheme adopted in column NP4.2 that is relatively higher steel angle ratio (7%) with wider strip spacing (2d) is denoted as Method-A. Again, another column designated as NP1.1 was tested with steel angle ratio of 4% and closer strip spacing of 1d.The strengthening scheme followed in this column is named as Method-B. Comparisons of the column behavior for these two strengthening methods are shown in Figure 4.28.

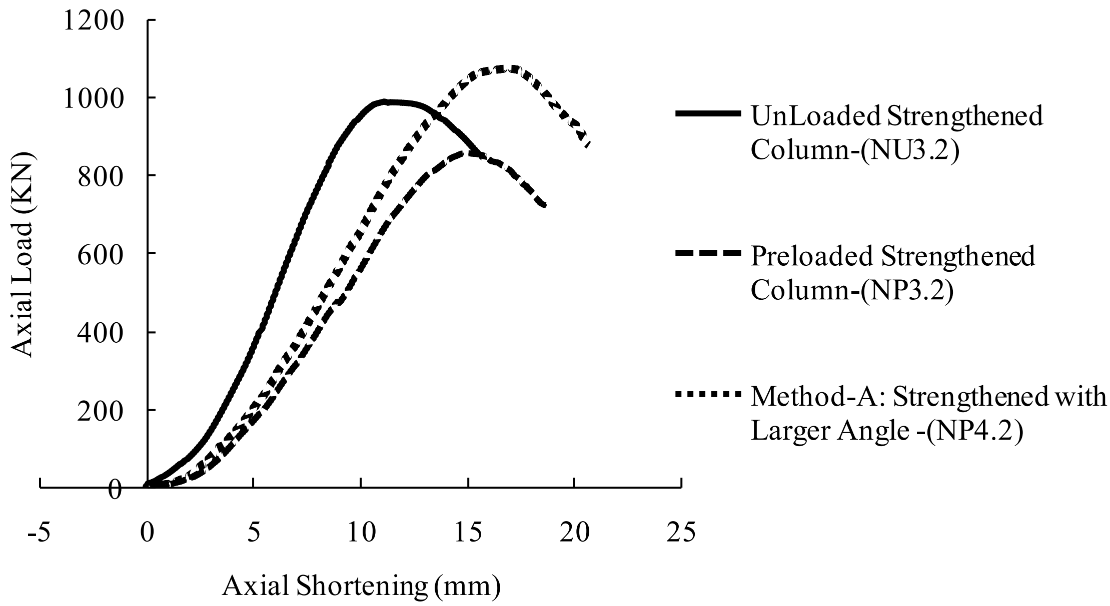


Figure 4.27 Improvement of preloaded strengthened column behavior using Method-A.

In Method-B, the pitches of the horizontal strips are reduced to half but the angle size kept similar to column NU3.2. From the study, this method is found to display similar behavior along with greater displacement and higher initial stiffness as like as column NU3.2. Although, the strength is not improved as much as NU3.2, overall column ductility is increased with this method. On the other hand, column strengthened with Method-A demonstrated higher strength (23%) as compared to that obtained from

Method-B. However, Method-B which uses closer strip spacing provides relatively higher ductility (51%) as compared to that obtained from Method-A.

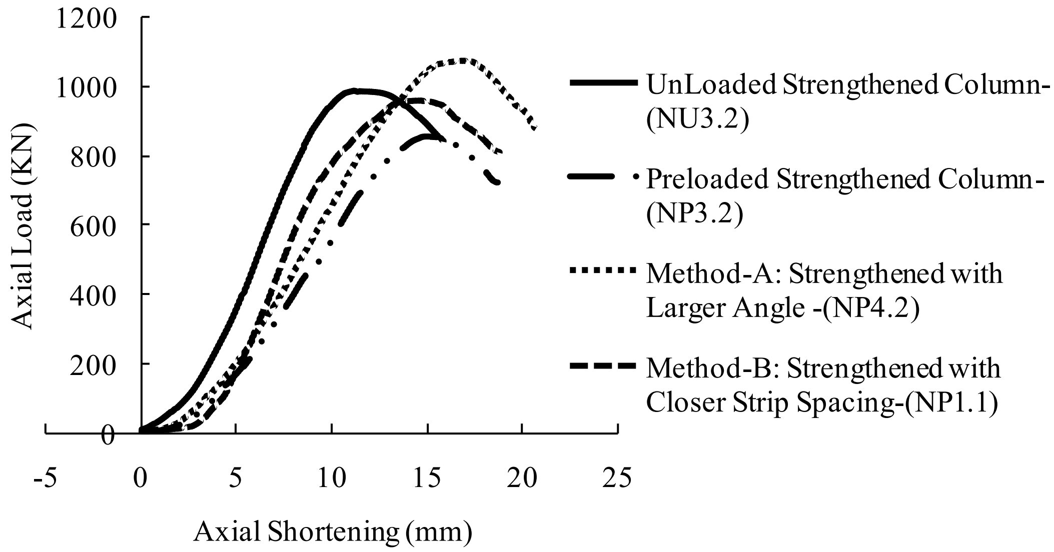


Figure 4.28 Better improvement of preloaded strengthened column behavior using Method-B.

Since, strength enhancement is the primary objective of column strengthening, Method-A is concluded as the most efficient strengthening scheme in the study. Although Method-A does not enhance ductility index as much as Method-B, ductility increment offered by this procedure is quite safe and satisfactory as compared to the unstrengthened column NC1.This method will also be economical since it might have reduced labor, time, and weight and hence might help to reduce the overall cost of the project.

4.7 Summary

This study was under taken in order to explore the behavior and strength of the steel angles and strips jacketed deficient RC columns under concentric axial loads. To achieve this objective, an experimental attempt has been made on seven strengthened and one unstrengthened RC columns. Column 70% preloading stage, angle size and strips spacing, partial strengthening at column ends has been the fundamental studied parameter in this study. From the test results, it has been found that the strengthened columns improve both of their load carrying capacity (ranging from 87% to 186%) and ductility (ranging from 31% to 67%) than the unstrengthen RC column. The highest strength improvement has been obtained with the columns having no preloading condition. But this enhancement has been found to be reduced (with an average value of 27%) due to the application of column 70% preloading condition before strengthening. This is certainly a considerable influence of strength degradation due to initial loading before strengthening. The preloading condition also significantly affects the failure behavior and ductility of the strengthened columns with no preloading.

Among the preloaded column groups, maximum strength enhancement has been obtained due to the variation of angle size parameter (26% to 31% increment). A satisfactory level of strength improvement is also achieved due to the effects of horizontal steel strips spacing parameter (12% to 17% increment). But this parameter mainly enhanced the columns displacement ductility (71% to 91%) as well as confined concrete strength. It has also been found that column strength and the spacing of the horizontal steel strips are inversely proportional. However, both of the unloaded and preloaded columns conformed to the similar behavior and ductile failure pattern under concentric axial load. But the partial strengthening configurations did not prove itself efficient as much as other strengthening method as it did not improve the column ductility at all.

This study finally provided an efficient steel angles and strip jacketing method based on the limited test data of current research. It was observed from this study that 7% steel angle ratio along with 2d (d is the smaller dimension of the RC column) strip spacing enhanced the ultimate strength by 135% and ductility by 35% which is very much appreciable. Therefore, steel jacketing scheme with 7% steel angle ratio along with 2d strip spacing can be practiced for deficient RC columns. However, more test data are required to propose an optimum strengthening scheme for steel jacketing method for RC columns. The existing capacity prediction models are found to agree well with the experiment results. Their performance is found quite safer for preloaded columns also. The capacity prediction model proposed by Tarabia and Albakry (2014) is found to show better agreement with the experiment results.

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 General

In this study, an experimental program was undertaken in order to investigate the behavior and strength of the steel jacketed reinforced concrete columns under concentric axial loads. The steel jacketing technique adopted in this study consists of four equal leg angles positioned at four corners of the column and discrete horizontal strips distributed at a uniform spacing along the length of the angles. The main variables of this study were column's preloading state, steel angle ratio (4% to 7%), strip spacing (from 2d to 1d), and method of partial strengthening at column ends. Since insufficient study was found on preloaded RC columns in literature review, this study mainly focused the behavior and strength enhancement of the initially loaded, damaged and repaired RC columns. For this study, eight RC columns were constructed with a dimension of 150 × 150 × 1500 mm; a longitudinal rebar ratio of 1.4% and a low concrete strength of 15.2 MPa. All the columns were cast by placing them horizontal position in contrast to the actual situation where columns are cast in vertical position. However among the eight columns, five specimens were preloaded up to 70% of their ultimate strength. Then the damage accumulations during the preloading stage were repaired and later the columns were strengthened by steel angles and strips jackets. Total seven columns were tested as strengthened test specimen and one was tested without strengthening which was used as reference column. During the test the steel angles were subjected to direct loading. The axial load shortening behavior, failure mode, ultimate strength and displacement ductility were obtained through the laboratory experiment and the effects of the selected the parameters were analyzed for preloaded RC columns. The validity of the capacity prediction models in predicting the ultimate capacities of preloaded strengthen RC columns were also investigated in this study.

5.2 Conclusions

In view of the results and load-shortening curves of each specimen, the following conclusions are deduced tentatively in this study.

- The performance of the steel jacketing method using angles and strips is found to be very efficient. Significant improvement in ultimate load and ductility is obtained. In this study, the enhancement in ultimate axial capacity of deficient RC columns due to steel jacketing ranged from 87% to 186% as compared to the unstrengthen test column. This gain in capacity is mainly due to the confinement effect of the external steel cage. The displacement ductility of the strengthened column is also found to be amplified ranging from 31% to 67% with respect to the reference test specimen.

- The failure mode of the unstrengthened RC column is brittle while jacketing the column with steel angles and strips changed the failure mode to ductile behavior except the column strengthened with discontinuous steel angles at the ends (i.e. partial strengthening scheme). The failure in most of the strengthened specimens is due to the buckling of the steel angle followed by yielding of steel angles and crushing of concrete.

- The steel jacketing method is found to perform well for RC columns with preloading (70% of their ultimate strength, resembling damage due to over load) before strengthening. The improvement in ultimate axial capacity of the preloaded strengthened RC column is found to vary from 87% to 135% of the standard specified. This variation is due to the various angle size and strips spacing adopted in this study.

- However, strengthened columns with no preloading showed approximately 27% higher capacity as compared to the columns with 70% initial loading. Test columns with no preloading showed a ductility enhancement of around 60% whereas preloaded columns demonstrate about 30% increase in ductility.

- The effects of steel angle ratio and strips spacing is studied on preloaded strengthened columns. Increase in angle steel ratio from 4% to 7% proportionally increases the axial strength, displacement ductility and confined concrete strength of the preloaded column.

- The strength of the column is proportionally increased with the increase of steel angle ratio. The increase in steel angle ratio from 4% to 7% results in an enhancement in ultimate strength by 26% for 150 (d) mm strip spacing and 31% for 300 (2d) mm strip spacing. This parameter provides the highest influence in achieving the ultimate strength of strengthen columns in current study. The displacement ductility index is improved ranging from 13% (2d) to 23% (d) for the same steel angle ratio variation.

- Decreasing the strip spacing from 300 (2d) to 150 (d) mm increases the capacity of the preloaded columns by an average value of 15% with respect to the column with 2d strip spacing. However, the ductility of the column with reduced strip spacing (d) is found 60% higher as compared to the column with 2d strip spacing. This is due to the increase in the concrete confinement resulting from closer strip spacing.

- It is highly recommended not to practice the partial strengthening with steel jacketing at column ends only for deficient RC column since it renders brittle and explosive failure mode.

- All the capacity prediction models analyzed in this study are found to be safe for predicting the axial capacity of steel jacketed RC columns with 70% preloading condition. But the model proposed by Tarabia and Albakry (2014) showed better agreement with the experiment results.

5.3 Suggestions for Future Research

The following suggestions are provided for future research.

- This study performed experimental investigation on RC columns with 70% preloading condition only. Further research work is required performed to determine the efficiency of steel angles and strips jacketing method for different preloading stages since the number of available test data for strengthening of the preloaded column is very limited.

- Nonlinear finite element models need to be developed for verifying the obtained test results and failure behavior of the specimens under concentric axial load.

- Behaviour of eccentrically loaded RC columns with steel jacketing must be studied for various levels of preloading stages.

- Effects of concrete compressive strength on the behavior of steel jacketed RC columns also need to be addressed in future research.

- This study investigated the behavior of strengthen RC columns for monotonic loading conditions only. Further research is required to study the behavior of steel jacketed RC columns under cyclic and dynamic loadings.

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