TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING
M .Sc. Program in Structural Engineering

Thesis no: SS00118

EFFECT OF HOOK ANGLE AND SPACING OF TRANSVERSE TIES OF RC COLUMN IN AXIAL CAPACITY

SHEKHAR CHANDRA KC

IOE, Pulchowk, February 2010

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A Thesis Report Submitted By
SHEKHAR CHANDRA KC
(2062/MSS/r/108)

In partial fulfillment of the requirement for the degree of

MASTER OF SCIENCE
IN
STRUCTURAL ENGINEERING

IOE, Pulchowk, February 2010

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CERTIFICATE

This is to certify that the work contained in this thesis entitled "**EFFECT OF HOOK ANGLE AND SPACING OF TRANSVERSE TIES OF RC COLUMN IN AXIAL CAPACITY**", in partial fulfillment of the requirement for the degree of Master of Science in Structural Engineering, as a record of research work, has been carried out by **Mr. Shekhar Chandra KC** (2062/MSS/r/108) under my supervision and guidance in the Institute of Engineering, Pulchowk Campus, Lalitpur, Nepal. The work embodied in this thesis has not been submitted elsewhere for a degree.

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ACKNOWLEDGEMENTS

I express my sincere thanks and gratitude to my supervisor Dr. Roshan Tuladhar, Department of Civil Engineering, Institute of Engineering, Pulchowk Campus for his proficient guidance, kind support, rewarding technical suggestions and the persistent encouragement for the betterment of work throughout this thesis. Honestly speaking, I used to refill my energy with the very active and friendly discussion with him every time I went to consult him.

I would like to express my profound gratitude to the M. Sc. Program in Structural Engineering, Department of Civil Engineering, Pulchowk Campus for the financial assistance provided for the tour to the IIT-Kanpur at the initial stage of the thesis work. It was really a golden opportunity to get exposed to the precious literatures at IIT-K and the inspirational environment thereof including the interaction with respected professors: C.V.R. Murty, D. C. Rai. Had such very important literatures been not collected from, the thesis would not have come to present standard. I am grateful to the respected program coordinator Dr. Roshan Tuladhar for the technical and managerial advice during the same tour. I am grateful to the Program and the Department also for creating the studious and research environment by making the arrangement of a separate room with computers for the thesis work

I would like to express my sincere thanks to all my teachers who have brought me to this level. I can never forget the contributions of the respected expertise of structural engineering, Pulchowk Campus; especially Prof. Dr. P. N. Maskey, Prof. Dr. M. P. Aryal, Prof. Prajwal Lal Pradhan and Assistant Dean Mr. Jagat Kumar Shrestha, for their very valuable suggestions and encouragement at different stages of this thesis work.

I am thankful to my seniors and colleagues and all other helping hands for their kind cooperation during this thesis work especially Ravi Lal Sharma, Anil Kumar Shrestha, Rajendra BC, Nabin Shrestha, Rajendra Sapkota, Deepak Gupta and Chudamani Aryal.

Shekhar Chandra KC

062/MSS/r/108

ABSTRACT

Effect of different hook angle ties and the spacing of ties is investigated on axial load carrying capacity of RC columns. Axial capacities of sixteen small column and four medium columns are experimentally obtained. The progressions of damage and failure modes are found to depend upon the hook angle of ties and the spacing of transverse reinforcement. The columns with 135° ties bear the more axial load than the columns with 90° ties. Column with 90° ties opened and the longitudinal bars started buckled even in small spacing of ties. Column with 135° ties are found to be more grip to longitudinal reinforcement and they are not opened. Small ties spacing column show the high axial load carrying capacity than the column with large spacing of transverse ties.

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Chapter 1

Introduction

1.1 General

Columns of reinforced concrete frame buildings are generally designed to resist axial and biaxial bending moment induce by frame action under gravity and lateral load. If they are properly designed, detailed and constructed, the performance of the building improved substantially. Earthquake resistant structures need to be design to dissipate seismic energy through post-elastic energy dissipation in the members. This is achieved by designing certain structural members to possess large ductility and usually these select members in the frame structures are beams. Past earthquake have shown that columns possess poor strength and ductility and result in major structural damage or even complete collapse of the entire building.

The capacity design concept allows acceptable distribution of damage in structures, by ordering the strengths of structural members in an acceptable manner. As a result, strong column-weak beam approach was introduced in frames which encourage formation of plastic hinges in beams, rather than in columns. This design approach minimizes the possibility of formation of plastic hinges in columns. However, during strong earthquake shaking, plastic hinges may occur in columns also. Therefore, design codes require ductile detailing of RC columns so that they posses ductility in these eventualities. One of the important requirements of RC columns is confinement of concrete in the columns through transverse reinforcements.

The reinforcement cage of RC columns comprises of longitudinal bars and transverse ties (with or without cross ties). The transverse steel reinforcement confines the compressed concrete, provides shear resistance and prevents bond splitting failure as well as buckling of longitudinal reinforcement after damage of cover concrete. Thus, transverse reinforcement effectively mitigates the brittle nature of concrete and leads to inelastic deformation of longitudinal bars and delay of adverse, permanent failure modes. In other words, structure strength and ductility is largely depends on the amount and configuration of the transverse reinforcement in RC column.

1.2 Problem and statement

In Nepal, the construction practice is very poor. Due to highly unskilled manpower, the hook angle and the spacing of the transverse ties are not done as per design. Hook angle of 90° or less than 90° ties are used in construction due to ease of bend. Similarly, the spacing of the ties are also the one of the careless part in the column during construction.

Due to such practice in construction, the load carrying capacity of the column could be reduced. Hence during earthquake the entire structure could be vulnerable.

1.3 Aims and Objective

The main objective of the research work is to study the effect of transverse ties hook angle and the various spacing of ties in axial load carrying capacity and ductility of the column.

The confinement requirements given in the Indian Standard Code [IS 13920:1983 and IS 456:2000] are not sufficient to describe the effect of hook angle and spacing of transverse reinforcement in confinement of column. To equip the designers and builders with realistic specifications to create earthquake-resistant columns, the effect of ties hook and the spacing of ties to confine the core concrete on axial capacity of columns is the aim of the present study.

1.4 Scope of the Study

In this study, the axial capacity of the column for different angle ties and spacing are considered. Only square columns are prepared for study. Hook angle 90° and 135° are considered in this study. Spacing of 50mm, 100mm, 150mm and 200mm for small size column (100 x 100 x 6000 mm) and 75mm and 150mm for medium size column (150 x 150 x 1000 mm) is considered in this study. The columns are tested in monotonic axial compression.

Longitudinal bars of 8mm are used for small columns and 10mm are used for medium size columns. Transverse ties of 4.76 mm steel are used for all specimens.

1.5 Methodology

This study deals with an experimental investigation of sixteen small size columns and four medium size columns. The entire specimens are tested under monotonic axial compression on UTM. The progression of damage and the load carrying capacities were determined.

Chapter 2

Literature Review

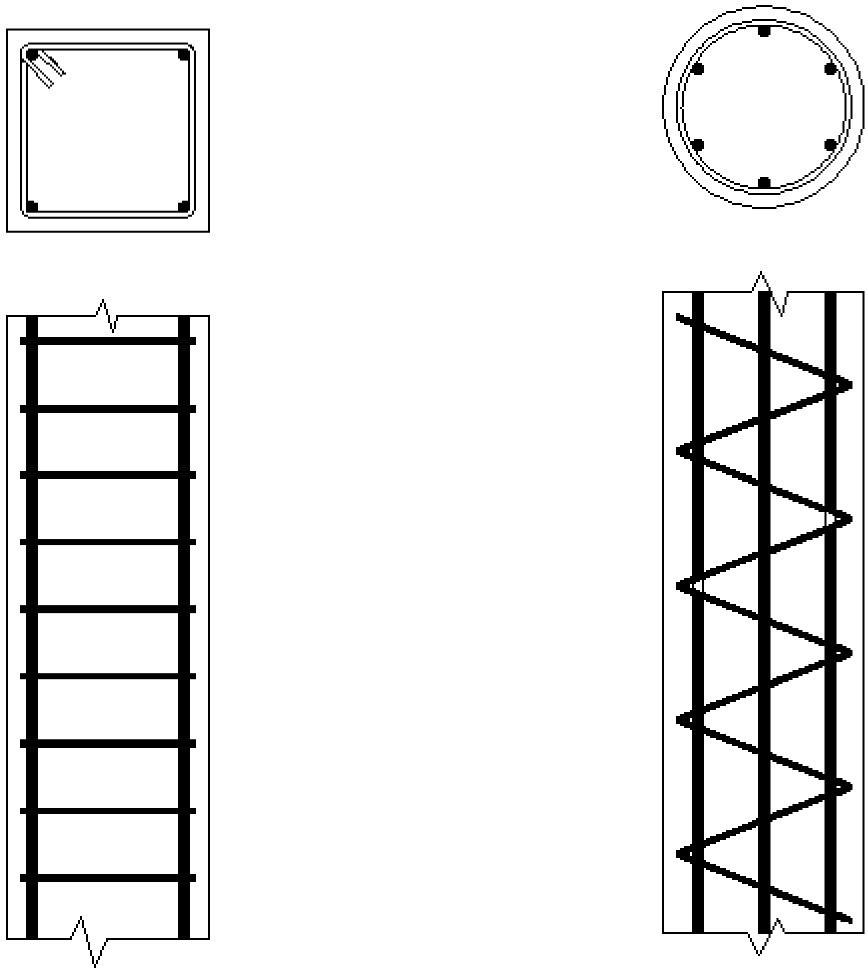
2.1 General

The safety of an RC structure during a major earthquake depends on its ability to deform inelastically while maintaining near maximum load carrying capacity. In earthquake-resistance design, behavior of concrete reinforced structural elements subjected to large inelastic deformations is highly influenced by the behavior of the confined concrete. For plain concrete, a generally recognizable relation exists; it provides an acceptable approximation of the properties of concrete in the structural elements. With the introduction of lateral reinforcement as confinement, the behavior of concrete changes and is influenced by a number of factors that comprise the lateral reinforcement. Longitudinal reinforcement in columns further complicates the phenomenon of confinement. The confinement associated with hoop reinforcement and transverse ties increased the ductility and energy absorption capacity of RC columns. Therefore, it is important to incorporate the effects of transverse ties and ties spacing into seismic design when evaluating strength and ductility.

2.2 Classification of RC columns

2.2.1 Based on type reinforcement

RC columns are classified into two types based on the reinforcement provided, namely (a) *Tied columns*, where longitudinal bars are enclosed within closely spaced transverse reinforcement, and (b) *Spiral columns*, where the main longitudinal bars are enclosed within closely spaced and continuously wound spiral reinforced. Spiral columns are common for circular cross-sections, sometimes used in square and octagonal sections also whereas tied columns are applicable to all cross-sections. (Figure 2.1)



(a) (b)

Figure 2.1: Column based on the type of reinforcement (a) tied, and (b) spiral.

2.2.2 Based on Type of Flexural Loading

Columns are classified into three types, based on the nature of loading, namely (a) Columns with axial loading (applied concentrically), (b) Columns with uniaxial eccentric loading, and (c) Columns with biaxial eccentric loading. The occurrence of pure axial compression in a column (due to concentric loads) is relatively rare. Generally, flexure (and, sometimes shear) accompanies axial compression – due to rigid frame action, lateral loading and/or actual eccentricities in loading. Column in RC framed buildings, in general fall into the third category.

2.2.3. Classification of columns based on slenderness ratios

Slenderness is a geometrical property of a compression member which is related to the ratio of its effective length to its lateral dimension. Columns are classified into two types depending on weather slenderness effect are considered insignificant or significant, namely (a) Short columns, and (b) Slender (or long) columns. The Indian concrete code (IS 456, 2000) specifies the limiting slenderness ratio as 12. Short columns often fail in strength with concrete and steel reaching their ultimate strength, whereas long columns often fail by buckling i.e., by large lateral deflection under compressive loads smaller than axial compression.

2.2.4 Based on Ductility

From earthquake resistant point of view, RC columns are classified based on ductility. Ductility allows use of lower design forces for buildings in regions of high seismicity than that for buildings without ductility. Ductility is achieved as a direct consequence of confinement of concrete. Confinement of concrete refers to holding concrete as a unified mass without allowing it to dilate due to Poisson's effect.

2.3 Effect of confinement on Concrete

Columns with effective transverse reinforcement consist of cover concrete (which is unconfined) and core concrete (which is confined). Confinement enhances ductility as well as strength of concrete sections through the development of passive lateral confining pressure on the core concrete. The extent of confinement achieved depends on how well lateral expansion of concrete is prevented; the lateral dilation of concrete is directly related to the axial compressive stress applied on it and to the Poisson's ratio of concrete. At low levels of axial compressive stress, the transverse reinforcement is hardly stressed and so the concrete remains as unconfined. As the axial stress in the column increases, the concrete starts dilating perpendicular to the axial direction due to progressive internal fracturing, and eventually bears out against transverse reinforcement, which in turn applies a confining reaction to the concrete. In this way, the transverse reinforcement (spirals, rectangular hoops and cross-ties) provides passive confinement, preventing the unstable propagation of internal cracking, the dilation of the core concrete thereby delaying the formation of longitudinal cracks. Thus the transverse reinforcement enables the concrete section to sustain large axial strain, which results in improved ductility and enhanced strength of the core concrete and therefore of the column.

The extent of confinement offered by transverse reinforcement and its effect on the behavior of circular column has been well documented. But, such exhaustive studied are not reported in literature on rectangular columns ties hook angle and spacing, while the effectiveness of rectangular ties is broadly understood to be less than that of circular ties, and its quantitative effect on the strength and ultimate axial strain of columns has been difficult to evaluate. The transverse confining pressure varies along the length of the hoop because the transverse hoop bends outwards and allows different amounts of lateral expansion of core concrete along its length. Therefore, unlike the circular columns confined by spiral or closed hoop reinforcement, the confining pressure in rectangular columns is non-uniform throughout the volume of core concrete both in cross-section as well as along the height (Figure 2.2). Notwithstanding this, rectangular columns and rectangular hoops are used, because of the ease of designing, detailing and fabricating them. At high axial strain levels, when the cover concrete spalls off, there is a reduction in the load carrying capacity of the core concrete also. The effective confinement of the core concrete at the tie level is essentially based on the distribution of longitudinal bars supported by transverse reinforcement. Thus, the effective confined concrete area is less than the core area. At a section midway between two adjacent transverse hoops, the core area is least confined (Figure 2.3). Lateral confinement pressure distribution in different types of transverse reinforcement varies along the column cross-section (Figure 2.4) and along the height (Figure 2.5).

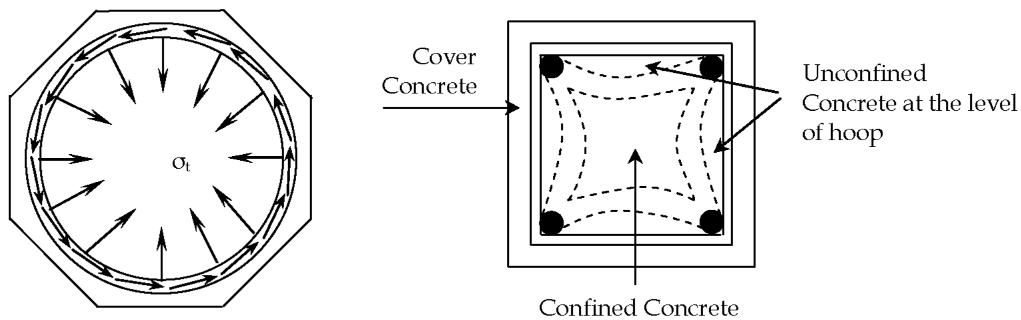


Figure 2.2: Types of transverse hoops (a) spiral hoop (b) square hoop [Penelis and Kappos, 1997]

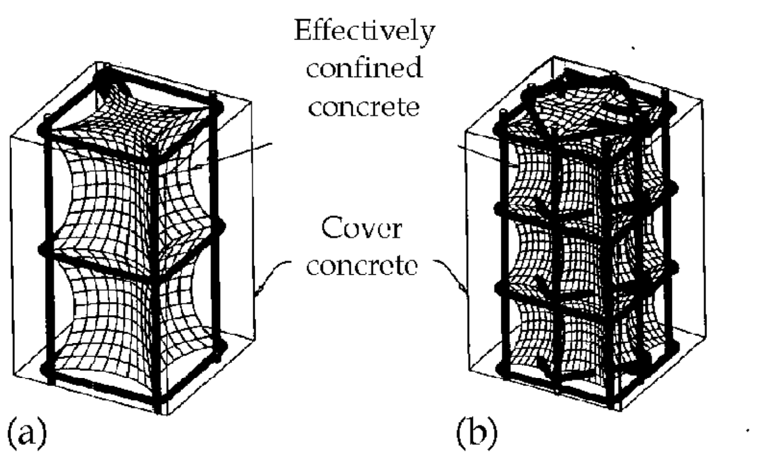


Figure 2.3: Arching action in confined core concrete : (a) poor tie configuration with large spacing, and (b) good tie configuration with small spacing [Cusson and Paultre, 1994].

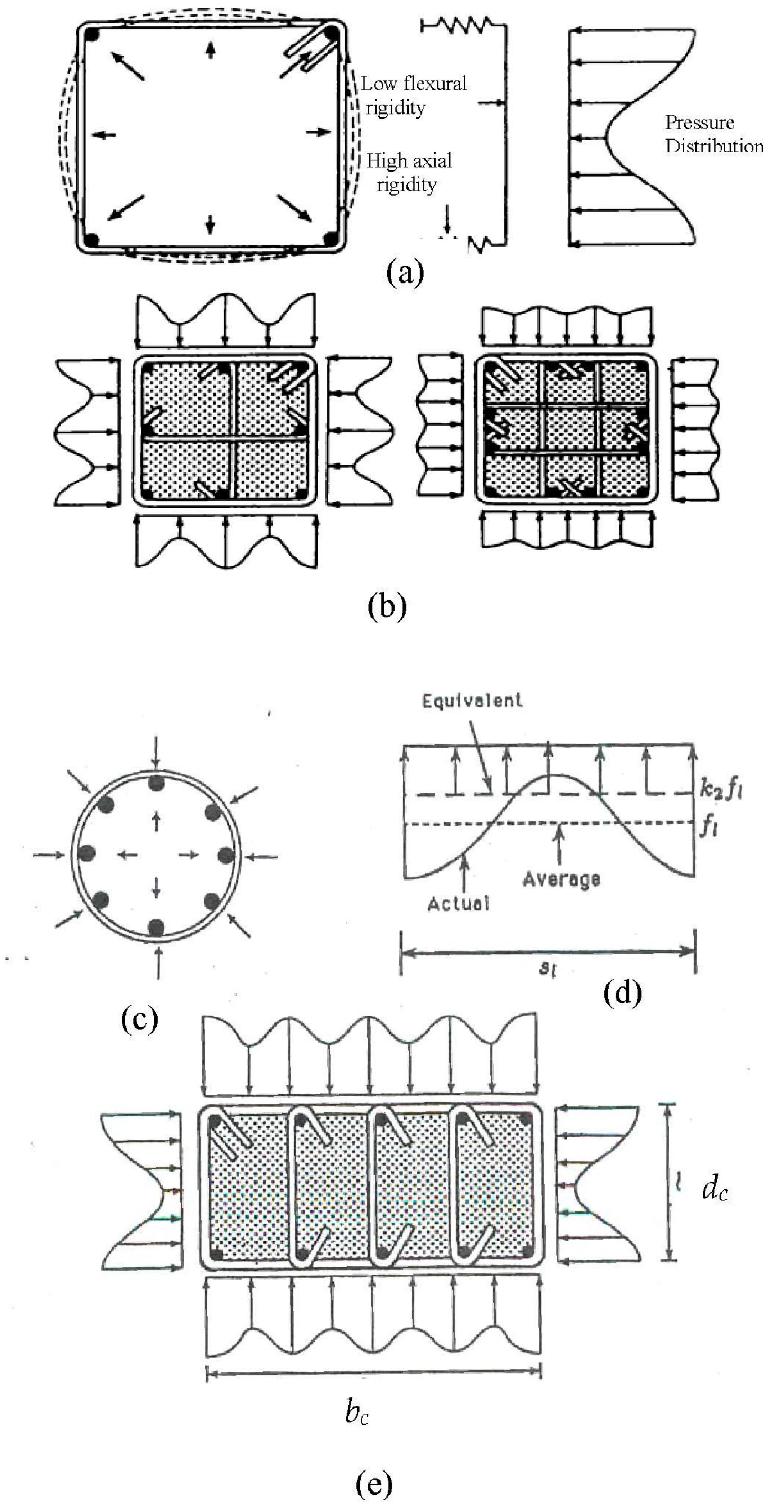


Figure 2.4: Variation of lateral confinement pressure in columns with different configuration (of geometry and arrangement of transverse reinforcement); (a) square column without transverse ties, (b) square column with transverse ties, (c)

circular column without radial ties, (d) Equivalent uniform pressure in square column (e) rectangular column with transverse ties [Saatcioglu and Razvi, 1999]

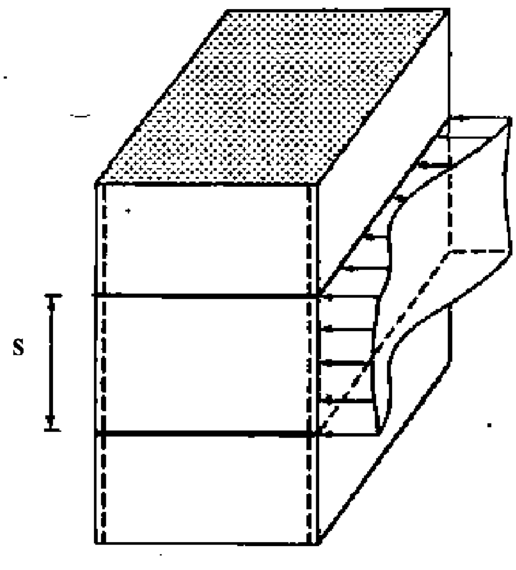


Figure 2.5: Distribution of lateral pressure along the column height [Saatcioglu and Razvi, 1992]

2.4 Failure Modes of Columns

In general, RC columns sustain two types of damages, namely axial-flexural failure and shear failure. In the long term gradual damage is also caused to RC columns due to creep and shrinkage, the time dependent effects on columns resulting in volume change of concrete leading to reduction in strength. Creep is affected largely by age of concrete, environmental humidity, size of member and water & cement contents etc. [MacGregor, 1997; Nilson, 1997; Branson, 1997]. Shrinkage also depends on the same factors, but is more likely to be affected by relative surrounding humidity and the amount of water in the concrete mix. [MacGregor, 1997; Nilson, 1997]. Under sustained compressive stresses, creep and shrinkage cause the load to transfer from the concrete to the longitudinal reinforcement bars. If the column has small amount of reinforcement the longitudinal steel bars may reach of the yield stress and consequently the column may fail. However, under seismic actions, RC columns are subjected to cyclic lateral loads with co-existing axial loads. Thus, failure in these columns is induced by inadequacy in shear strength, flexural strength of ductility. Considering these inadequacies, three typical failure modes are identified [Priestley and Seible, 1995]: (a) shear failure, (b) confinement failure in flexural plastic hinge region and (c) lap splices failure in longitudinal steel reinforcement.

2.4.1 Shear Failure

Most catastrophic and sudden mode of failure is shear failure. In a typical shear failure, first diagonal cracks in the concrete appear, followed by rupture of opening-up of transverse steel reinforcement and then by the bulking of longitudinal steel reinforcement. Due to inadequate arrangement of transverse reinforcement, column subjected to axial force sustain diagonal tension failure well before concrete reaches its compressive strength; this leads to brittle failure of columns. In seismic areas, where shear force induced in columns is high, large spacing of transverse reinforcement or 90° hook at the end of transverse reinforcement (Figure 2.6) are responsible for poor performance. This undesirable diagonal tension failure can be prevented by providing transverse reinforcement at close spacing and with 135° hook ends.

The presence of shear reinforcement distributes the shear stress on the column between the transverse reinforcement and the concrete, and prevents the premature appearance of the diagonal shear cracks in the concrete. Moreover, the longitudinal bars are prevented against bulking when the spacing between adjacent bars are prevented against bulking when the spacing between adjacent hoops is small.

(a) **(b)**

Figure 2.6: Brittle shear failure of columns during (a) 2001 Bhuj (India) Earthquake and, (b) 2001 Attico (Peru) Earthquake

2.4.2 Flexural Failure

Columns are usually under compression, and their ductility is strongly influenced by the level compressive axial load on them; the lesser the compressive load, the better is the ductility. Also, the ductile flexural behavior of columns subjected to axial forces largely depends upon the design of transverse reinforcement. When the axial load is close to the pure axial compressive capacity of the column, the failure of the column is due to the compression of the concrete. And, when this ratio is small, the failure is due to tensile yielding of longitudinal reinforcement. In the latter case, as the compressive axial load increases, compressive axial strain increases in the columns. The cover concrete, being unconfined, starts spalling off. At this point, if the core concrete is effectively held by the transverse reinforcement and subjected to lateral confining pressure, the column continues to carry higher compressive load; this increase the ductility of the column.

2.4.2.1 Flexural Plastic Hinge Failure

This type of failure occurs at column end and is limited to a small region. It is characterized by spalling of cover concrete, failure of transverse reinforcement and bulking of longitudinal reinforcement. It is accompanied by large inelastic flexural deformation, and thus is more ductile and desirable mode of failure than brittle shear failure.

2.4.2.2 Lap Splice Failure

This is another type is seismic flexure failure that occurs when longitudinal reinforcement is lap spliced at maximum moment regions near the column ends. During strong earthquake shaking, these lap splices often break down leading to loss of structural integrity of the column for sustaining large inelastic deformations to achieve the required energy absorption capacity (Figure 2.7). Splices should be located in regions with low flexural stresses.

Figure 2.7: (a) Plastic hinge formation at column end during Turkey Earthquake, 2003, and (b) Lap splice failure during Sumatra Earthquake, 2004

2.4.3 Other Failure Modes

There are other modes of failure in a column, including anchorage failure, bond splitting failure and joint failure. *Bond splitting* failure is generated due to lack of closely spaced ties, low concrete strength of thin cover to longitudinal bars. Under these circumstances, splitting cracks develop along the longitudinal bars resulting in loss of bond stress, weakening the column and potentially resulting in various failures which can be associated with weak columns. *Anchorage failure* is a direct sequel of inefficient anchorage of longitudinal reinforcement in columns within a beam column joint. When column reinforcements are anchored in plain concrete, high anchorage bond stress occur, which in turn cause bond splitting failure. *Joint failure* is observed in moment-resisting frames designed for weak beam-strong column behavior. The beam-column joints in such frames are heavily stressed after yielding of the beam and diagonal cracks appear in the joint panal zone. It the degradation of the joint is sufficiently sever, the concrete spalls off resulting in loss of column support and thus finally leading to the failure of joint. Another mixed mode of failure termed as *flexural-shear failure* occurs in slender columns with low axial loads; it starts as a flexural crack but progresses into becoming a diagonal shear crack. *Axial compression failure* is common in stocky columns bearing high axial loads and is characterized by complete detoriation of the concrete in the column core prior to the formation of any diagonal shear of flexural cracks.

2.5 Past Studies Related to Confinements

2.5.1 Experimental studies

Extensive experiments have been performed on the effect of confinement in concrete. Tests have been shown that confinement of concrete by suitable arrangement of transverse reinforcement results in a significant increase in strength and ductility of compressed concrete. The extent of enhancement of the column performance is affected by the shapes of cross-section of the columns. Circular, square and rectangular columns have been studied.

2.5.1.1 Circular Columns

Normal strength concrete:

The first information on modeling of concrete was obtained from research on concrete cylinders, confined by spiral reinforcements or hydrostatic pressure (Richart et al., 1928). Concrete strength was found to increase proportional to the lateral confining pressure.

Spirally reinforced small sized cylindrical columns were experimentally tested (Ahmed and Shah, 1982) to quantify the influence of yield strength and longitudinal spacing of transverse steel reinforcement on concrete properties. Increase in volume of spiral reinforcement significantly was found to enhance the ductility and strength of confined concrete. How ever, the effect on ductility was more than that on strength. Only a marginal effect was seen on the strength and ductility of columns.

Monotonic compression tests were performed on twenty-seven short columns (Sheikh and Toclucu, 1993) to compare the relative contributions of circular hoops and spiral ties on the strength and ductility of circular columns. The diameter or the test specimens were 203, 254 and 356 mm and the volumetric ratio of the transverse reinforcement ranged from 0.0058 to 0.023. The mean compressive strength of concrete was 35Mpa. Circular hoops and spiral ties offered similar performance. Strength and ductility increased in the confining steel, but the effect on ductility was higher. Varying the size of the similarly confined specimen did not result in appreciable difference in column performance.

Twelve cylindrical specimens (250mm diameter and 1000mm length) were tested under monotonic axial compression (Ilki et al., 1997). To study the effects of number of parameters on the column performance, including volumetric ratio, diameter, spacing and types (hoops or spirals) of the confining steel. The mean compressive strength of the concrete used was 18Mpa. Keeping the volumetric ratio constant, decreasing the longitudinal spacing of transverse reinforcement resulted in increased by using higher diameter bars as transverse reinforcement, the strength enhancement was not much due to buckling of longitudinal bars. However, strength enhancement was appreciable when volumetric was increased by using higher grade steel for transverse reinforcement. Again there was no significant difference in the efficiency of spiral ties and circular hoops.

Tests on concentrically loaded circular columns (500 mm diameter columns with 1500 mm height) confined by spirals were performed at quasi-static and high strained rates to study the arrangement of longitudinal and transverse reinforcement (Mander et al, 1988). The volumetric ration of the confining steel varies from 0.006 to 0.025. A direct proportional relation was observed between the volumetric ration of confining steel and strength development in columns. Also, with the increase in the volumetric ration, the longitudinal strain at which the hoop fracture occurs increase and the falling branch of the stress-strain curve of confined concrete flattened

Stress-strain curve of confined concrete was proposed based on results of test on 200-500 mm circular concrete columns (Hoshikuma, 1994). Uniaxial monotonic compressive load was applied in displacement-controlled mode at rate of 1mm per min. mainly three parameters, peak stress, strain at peak stress and detoriation rate were studied along with the effect of cover concrete on the columns. When the concrete stress exceeded the peak stress, large spacing of hoops resulted in development of crack around the longitudinal bars following by buckling of the longitudinal bars and by crushing of core concrete within hoops. On the other hand, specimen with closely spaced hoops failed when some of the hoops ruptured and outward locally buckling of longitudinal followed. It was also noticed that as the volumetric ration of confining steel increase, the peak stress and corresponding strain also increased preventing severe detoriation of concrete after the peak stress.

High Strength Concrete

Effect of high strength confining steel was studied on the axial behavior of columns (Sun and Sakino, 1993). Forty eight columns were tested which were reinforced with high strength circular hoops having yield strength of 1130Mpa. The concrete used for the test had compressive strength ranging from 32.3MPa to 134.3MPa. Strength and ductility of high strength concrete is improved by use of high strength confining steel.

High strength concrete (HSC) under compression was studied to obtain the stress-strain behavior of concrete with strength above 69MPa with or without ties confinements (Hsu and Hsu, 1994). Cylindrical specimens (76.2mm x 152.4 mm) were tested for this purpose. In general, high strength concretes have higher strain corresponding to peak stress than normal strength concretes. The ascending portion of the stress-strain curve is more linear and steeper. Also, the maximum compressive strength and strain corresponding to peak stress increase with increasing lateral tie configuration.

The axial load behavior of high strength concrete was studied by testing eight full-scale spirally reinforced columns (Pessiki and Pieroni, 1997). These columns with concrete compressive strength ranging from 34.5MPa to 69Mpa and diameter 559mm were subjected to concentric axial compressive loading. The test variables were longitudinal reinforcement ratio, concrete strength, size and pitch of the spiral reinforcement. Lower strength columns were more ductile than higher strength counterparts. Columns possessing lower longitudinal reinforcement ratio fared better than those with observed in lower strength columns in contrast to higher strength columns which exhibited shear failure mode.

An experimental study was conducted wherefrom it was inferred that lateral confinement pressure required for HSC may be significantly higher than that for normal strength concrete (Razvi and Saatcioglu, 1994). Extensive test data were generated to established the characteristics of high-strength confined concrete whit circular geometry with concrete strength ranging from 60-124MPa. (Saatcioglu and Razvi, 1999). These test data were used to generate an analytical stress-strain model valid for both normal and high strength concrete.

2.5.1.2 Square columns

Normal Strength Concrete

Initial studies on rectilinear hoop confinement were performed intending to establish a formula for ultimate strength of RC columns with single square hoops (king, 1946), but ductility was not addressed. Much later in time, twenty four short columns were tested with 305mm square cross-section and 1960mm height (Sheikh and Uzumeri, 1979). The variables of the study were distribution of longitudinal steel around the column perimeter, tie configuration, spacing of ties and amount of longitudinal steel. The columns were subjected to axial monotonic compression. An increase in number of well distributed longitudinal bars that are effectively increase the confined concrete area, resulting in improved ductile behavior along with enhanced strength. Also, the strength gain was found to be not linearly proportional to volumetric ratio of lateral steel.

Short RC columns subjected to monotonically increasing compressive load at different strain rates ranging from 3.3×10^{-6}/s (static loading) to 16.7×10^{-3}/s (seismic loading) were tested [Scott, Park and Priestley, 1982]. Nine specimens (450mm square cross-section and 1200mm height) were studied with different arrangement of transverse and longitudinal steel. A control specimen with same dimension but without reinforcement was also tested for comparison. Confined concrete showed larger compressive strength and ductility than unconfined concrete. Confined concrete gained about 3.5-4% more strain than unconfined concrete (Figure 2.8).

In an extensive experimental program, sixty-three specimens (124 x 125 x 510 mm) were tested under concentric and eccentric loads [Sargin et al., 1971]. No longitudinal reinforcement was used in any of them; twenty-two were plain concrete and rest forty-one laterally reinforced. Eighteen plain and thirty-one reinforced specimens were tested under concentric loads. The test variables were concrete strength, type of lateral reinforcement, bar diameter, tie spacing, cover thickness and casting position. The force-deformation characteristics of concrete was found to depend upon volumetric ratio of lateral steel, types of lateral steel, spacing and grade of lateral reinforcement, and the quality of confined concrete. In particular, the effect of transverse reinforcement was found to decrease drastically with increasing spacing, and become negligible for spacing lager than the core dimension. Though the lateral reinforcement improves strength and ductility of the column, it has detrimental effect on the cover.

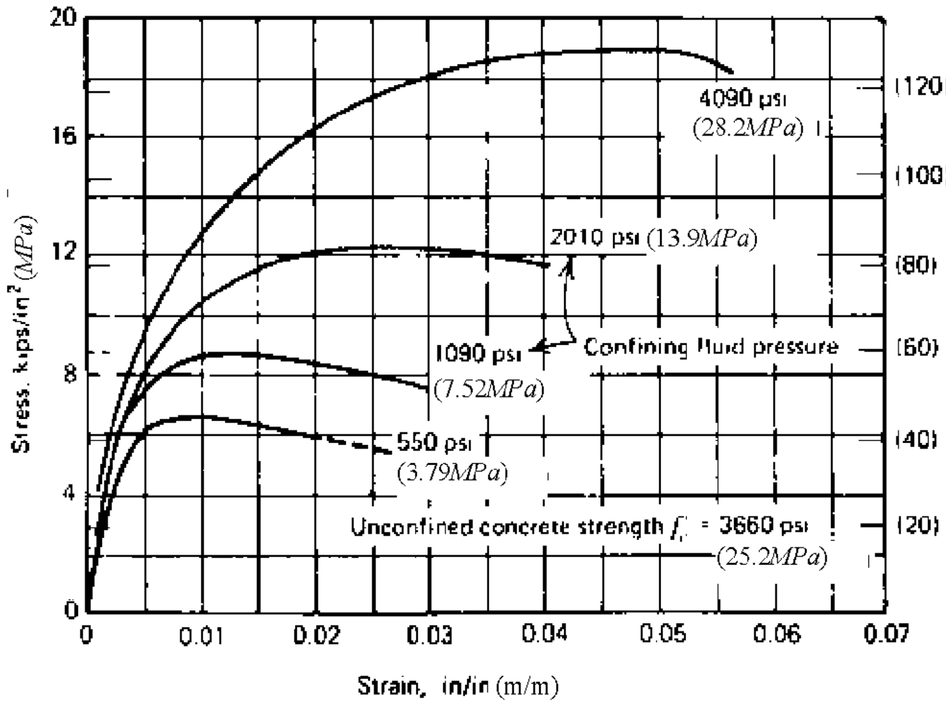
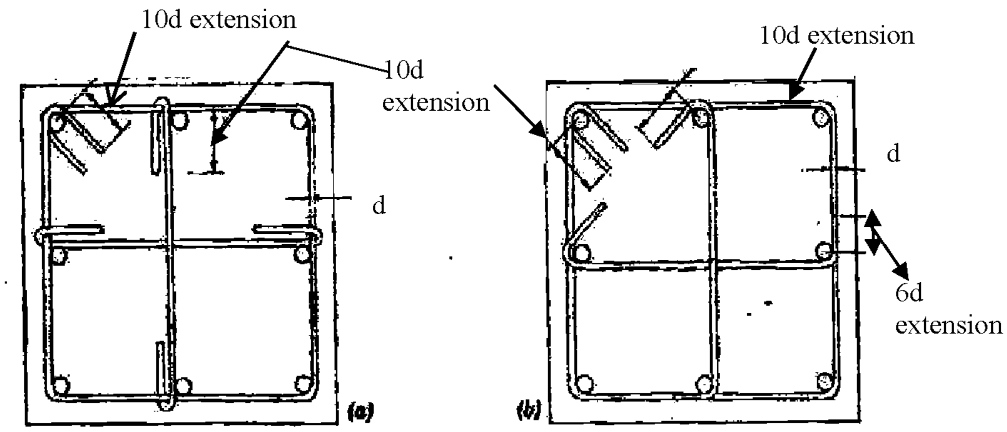


Figure 2.8: Experimental stress-strain curves obtained under various confining pressure [Scott *el al.*, 1982]

The confinement effectiveness of cross-ties was experimentally studied [Moehle and Cavanagh, 1984]. Ten columns were tested to failure under monotonically increasing axial compression. Each column was 914mm tall with 305mm cross-section. Arrangement of longitudinal bars was same in all the columns and spacing of transverse ties were also same; the configuration of transverse ties alone varied (Figure 2.9). Concrete had a mean compressive strength of 38Mpa. Out of the ten columns, two were plain concrete columns. RC columns performed better than plain concrete columns. Among the RC columns, those having intermediate hoops or cross-ties performed better than those without them. Columns having 90° and 135° hooked crossties were slightly less ductile than those 180° hooks, which in turn were slightly less ductile than sections with intermediate hoops. Failure of columns was initiated by longitudinal bars buckling followed by fracture of transverse ties.



(a) (b)

Figure 2.9: Recommended details of Hoops and Cross-ties (a) Permitted by Uniform Building Code and (b) Permitted by ACI 318-89 [Moehle and Cavangh, 1984]

Five columns (450mm square cross-section and 1200 mm height) with different hoop arrangement and two longitudinal bars configuration, namely, 12 and 8 longitudinal bars were axially loaded [Mande et al., 1988]. Strain rate of loading was 0.167/s. Two sets of columns were tested; one was 67 days old and another 940 days old. No significant ageing effect was observed. The stress-strain curve of confined square concrete columns was similar to those circular columns. For same value of volumetric confinement ratio, spiral reinforcements clearly shows better performance over the square hoops from the point of view of strength enhancement and ultimate compressive strain.

Five full sized square columns were tested for varying hoop reinforcement and spacing [Hoshikuma, 1994]. Two sets of square cross-section were considered (200mm and 500mm) with volumetric ratio ranging from 0.0019 to 0.0466. The column height was 1500mm, and the compressive strength of concrete ranged from 18.5Mpa to 24.3Mpa. Square columns showed trends similar to circular columns insofar as the effect of hoop spacing and volumetric ratio are concerned.

However, the strength development and ductility enhancement in case of circular columns was higher than those in square columns.

High Strength Concrete

A test program was carried out [Yong et al., 1987] on twenty-four columns made of high strength concrete with strengths varying from 83.6 MPa to 93.5MPa. The rectilinearly confined columns were tested to study the effects of volumetric ratio of transverse steel, unconfined concrete cover and longitudinal steel distribution around core perimeter. The square columns were 457mm tall and 153mm in cross-section, and were subjected to axial compressive loading. Peak stress and strain as well as ductility improved with increase in volumetric ratio, though not proportionally. Lateral steel confinement was less effective in HSC, and it did not yield at the first maximum compressive load. It was specified that for effective rectilinear confinement in HSC, the lateral hoop spacing must be kept less than the lateral dimension of the specimen.

Twenty-seven large scale high strength concrete columns with ties (235*235*1400mm) were studied under axial compressive concentric loading [Cusson and Paultre, 1993] to understand the complex mechanism of confinement of high strength concrete columns. Behaviors of these columns were characterized by sudden separation of cover concrete at an early stage leading to loss in axial load carrying capacity. After spalling of the cover concrete, well confined core concrete specimens gained strength, ductility and toughness. The failure of columns was characterized by formation of inclined shear sliding surfaces. In well confined specimens, improvement in strength was 50% and 100% and ductility as 10 and 20 times larger than the corresponding values of unconfined specimens made of 99.9MPa and 52.6MPa, respectively.

Ten square (250mm cross-sectional dimension and 1200mm height) large size square columns were studied for strength and deformability [Saatcioglu and Razvi, 1998]. The transverse steel had different arrangements. (a) single peripheral hoop, (b) hoop with one tie at the centre in both directions, and (c) hoop with two ties in both directions at equal spacing. The columns failed in a brittle mode implying that ductility requirements for high strength concrete columns need to be more stringent than normal strength concrete columns.

Another experimental study inversigating flexural capacity and ductility of high strength concrete [Azizinamini and Kuska, 1994] concluded that columns subjected to axial load of 20% of the design load capacity as per ACI 318-89 seismic provisions show sufficient amount of ductility. Nine 2/3rd scale square column specimens (305mm cross-section and 2440mm height) were subjected to an initial constant axial load applied followed by application of repeated lateral loads. Three levels of initial constant axial loads were applied, namely 20%,30% and 40% of design axial load capacity of column.

Square columns (200mm sections) were tested under axial compression [Sun et al., 1994] with different configurations of lateral steel reinforcement. Thew strength of concrete was 60 MPa. Specimens with nearly same lateral reinforcement and bars with smaller diameter showed better ductility effects.

2.5.1.3 Rectangular columns

Compared to the widespread study and analysis of circular and square columns, experimental research in the field of rectangular columns is still an unexplored area. Experimental investigations regarding rectangular hoops were extended by other researchers subsequently [Chan, 1955; Roy and Sozen, 1963; Soliman and Yu, 1967] and the stress-strain curves were proposed for confined concrete. Tests on columns with spiral ties and rectangular hoops proved spiral to be more effective than rectangular hoops from the point of view of ductility and strength ductility and strength [Aoyama and Noguchi, 1979].

The confinement effect was studied on sixteen rectangular columns (150*700 mm cross-section and 1200mm height) [Mander et al., 1988] under axial compressive loading applied either quasi-statically and at high strain rate. A relation was obtained between volumetric ratio of lateral steel and the stress-strain curve of confined concrete. As the volumetric ratio was increased, the peak stress increased and the descending branch of the curve became flatter. Also, longitudinal strain at which hoop fracture occurred increased with an increase in the volumetric ratio of lateral steel. Still, rectangular columns performed poorly in comparison to circular columns, because confining pressure varies in rectangular columns unlike the uniform pressure in spirally reinforced circular columns.

In another study, peak stress, strains corresponding to peak stress and strength detoriation were studied in relation to different configuration of transverse steel through four full-sized rectangular column specimens [Hoshikuma, 1997]. Two of them had same cross-sections (250mm*1000mm) and rests two were 300mm*900mm and 350mm*700mm; the column height in each case was 1500mm. The volumetric ratio of transverse steel varied from 0.0172 to 0.0245. The behavior of columns was significantly influenced by aspect ratio of column cross-section and use of cross-ties. As the aspect ratio increased, the peak stress and ductility detoriated appreciably. Also, columns without cross-ties resulted in dilation of the core concrete on application of axial compressive load, and those having the cross-ties were safeguarded against this dilation, and thus resulting in a more ductile performance.

2.6 Code Provision for Confinements

Ductile detailing for RC columns have been codified in some codes based test results performed on the column specimens.

2.6.1 ACI Building Code (ACI 318-05)

ACI Code requirements for minimum transverse reinforcement are based on the strength enhancement of concrete due to confinements. The underlying concept is that the load carrying capacity of a column should be maintained at the instance of spalling of cover concrete. The minimum transverse reinforcement requirements of the ACI code are:

$$(\ \ )\quad = \begin{cases} 0.45\ \frac{}{} - 1\ \frac{}{} \qquad \geq 0.12\frac{}{} \\ \\ 0.3\ \frac{}{} - 1\ \frac{}{} \qquad \geq 0.12\frac{}{} \end{cases} \tag{2.1}$$

Thus, Equation (2.1) suggests that rectangular transverse reinforcement is taken to be 75% as efficient as circular hoops in confining the same volume of concrete. This design axial load does not consider accidental eccentricities in the axial loads. Also, there is no provision on maximum allowable for transverse reinforcement; practical considerations determine this.

Maximum allowable spacing of the transverse reinforcement is minimum of

(a) one-quarter of the minimum member dimension

(b) six times the diameter of the longitudinal reinforcement or

(c) $\quad = \begin{matrix} 4 + \frac{}{} \\ \leq 150 \end{matrix}$

Where h_x is the maximum center to center spacing of cross-ties or hoop legs

2.6.2 New Zealand Code (NZS 3101: 2006 Part- I)

The NZS Code recognizes that axial load is detrimental to ensuring ductility. The expression for the minimum transverse steel in rectangular column is:

$$(\quad) \quad = \begin{cases} 0.3 \quad h \; — \quad — - 1 \quad 0.5 + 1.25 \frac{\quad}{\emptyset} & (2.2) \\ \geq 0.12 \quad h \; — \quad — - 1 \quad 0.5 + 1.25 \frac{\quad}{\emptyset} & (2.3) \end{cases}$$

And in circular column equation (2.1) is similar to Equations (2.2) and (2.3) with the exception of the penalty factor for higher axial loads.

Diameter of rectangular hoops or tie reinforcement is required to be at least *5mm* when longitudinal bar diameter is less than *20mm, 10mm* when longitudinal bar diameter varies from *20 to 32mm* and *12mm* when longitudinal bar diameter is more than or equal to the smaller of *(a)* one third least lateral dimension of the cross-section, and *(b)* 10 times the diameter of longitudinal bars being restrained.

2.6.3 Japanese Code (AIJ 1994)

The Architectural Institute of Japan (AIJ) recommends the minimum diameter of hoops to be *9mm* and a spacing of hoops of at least *(a) 150mm, (b)* one-half of the smaller section dimensions, and *(c)* 7.5 times longitudinal bar diameter. This recommendation incorporates the special consideration of lateral reinforcement necessary to provide sufficient confinement effect to obtain the desired deflection capacity. Further, the code requires the design axial load to satisfy the following constrain:

$$\frac{1}{3} \qquad < \quad \leq \frac{2}{3} \qquad (2.4)$$

The code specifies that when spacing between longitudinal bars exceeds *200mm,* additional lateral reinforcement ties should be used for increasing confinement.

2.6.4 European Code (EC 8, 2004)

The minimum required confining reinforcement is specified for both circular and rectangular cross-sections. For rectangular columns,

$$(\quad) \quad \geq \qquad , \quad , \frac{2}{3} \quad , \qquad (2.5)$$

Where

$$, \quad = \frac{}{} \quad + 0.13 \frac{}{} (\quad - 0.01) \qquad (2.6)$$

In which is the normalized axial force and the mechanical ratio of confining reinforcement. Depending upon the desired seismic behavior, and ( ) are specified (Table 2.1):

Table 2.1: Minimum Values of the Modification Factors [EC8, 2004]

Desired Seismic Behavior		( )
Ductile Behavior	0.37	0.18
Limited Ductile Behavior	0.28	0.12

2.6.5 Australian Code (AS 3600, 2001)

The lateral restrain in the form of circular or non-circular hoops and ties are required to have a spacing not exceeding the smaller of least lateral cross-sectional dimension and $15d_b$, where d_b is the diameter of the smallest bar in the column. The specified diameter for the transverse reinforcement in coherence with the longitudinal diameter is given in Table 2.2

Table 2.2: Minimum Bar diameter for Ties and Helices [AS 3600-2001]

Longitudinal bar diameter (mm)	**Minimum bar diameter of ties or helix (mm)**
≤ 20	6
24 ≤ diameter ≤ 32	10
≥ 32	12

2.6.6 Indian Standard Code (IS 13920:1983)

Special confining reinforcement requirements similar to those in the ACI Code are available. The minimum transverse steel required is given as by:

$$(\quad)\quad \geq \begin{cases} 0.9 \quad — \quad — - 1 \\ \\ 0.18 \quad — \quad — - 1 \end{cases} \qquad (2.7)$$

Equation (2.7) suggests that efficiency of rectangular hoops is 50% of that of circular hoops in confining the same volume of core concrete. Minimum diameter for transverse reinforcement is recommended as *8mm* when longitudinal bar diameter is less than or equal to *25mm.* for longitudinal bars with higher diameter, the transverse reinforcement is required to have a minimum of *10mm*.

Chapter 3

Experimental Study

3.1 General

Square cross-section RC columns with different configuration of transverse ties are subjected to monotonic axial load. The details of experimental study (including fabrication, loading, instrumentation etc.) along with the results are presented in this chapter.

3.2 Test specimens

Twenty RC columns were constructed. They are divided into two groups, namely group 'A' and group 'B'. In group 'A' types specimen, sixteen columns with cross-section area 100 mm x 100 mm and overall length 700 mm, and in group B types, four columns with cross-section area 150 mm x 150 mm and overall length 1500 mm with test region of 1000 mm. Transverse ties spacing and hook angle are the variable for both group of columns. 4 numbers of 8mm diameter longitudinal reinforcement were used for all columns in group 'A' and 4 numbers of 10 mm diameter longitudinal bars were used for all columns in group 'B'. Group 'A' columns were constructed without column head whereas group 'B' columns were with column head. Reinforcement details of the specimens are presented below.

Table 3.1: Reinforcement detail of Group 'A' columns

Column	Longitudinal bars		Transverse ties		
	Bar diameter (mm)	Number	Bar diameter (mm)	Spacing (mm)	Hooks
C1	8	4	4.76	50	90˚
C2	8	4	4.76	50	90˚

C3	8	4	4.76	50	135˚
C4	8	4	4.76	50	135˚
C5	8	4	4.76	100	90˚
C6	8	4	4.76	100	90˚
C7	8	4	4.76	100	135˚
C8	8	4	4.76	100	135˚
C9	8	4	4.76	150	90˚
C10	8	4	4.76	150	90˚
C11	8	4	4.76	150	135˚
C12	8	4	4.76	150	135˚
C13	8	4	4.76	200	90˚
C14	8	4	4.76	200	90˚
C15	8	4	4.76	200	135˚
C16	8	4	4.76	200	135˚

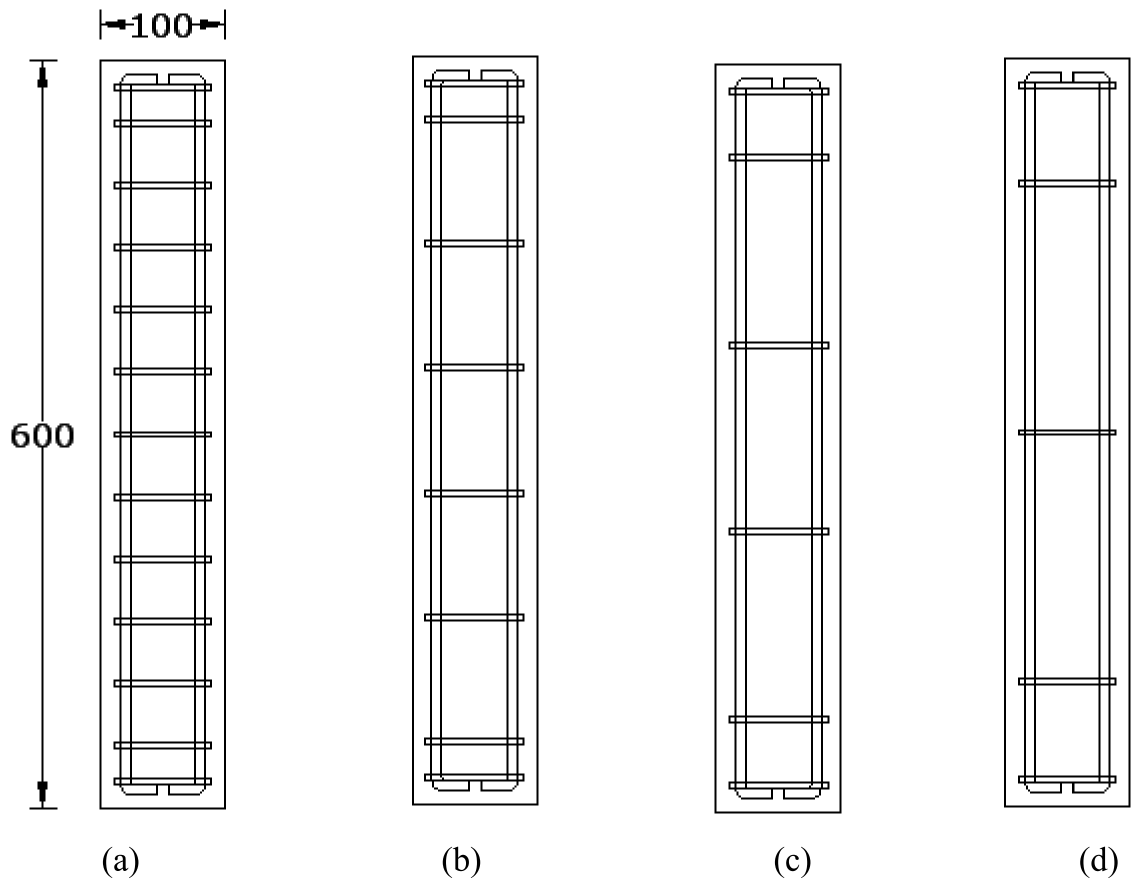
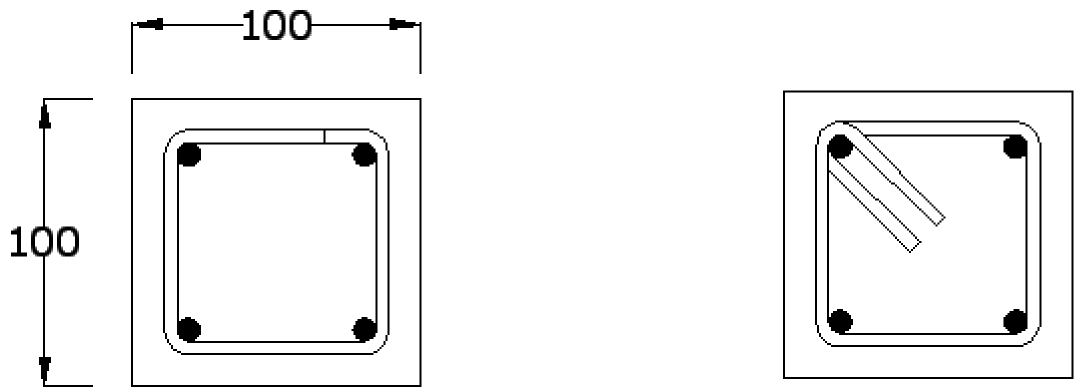


Figure 3.1: Elevation and Side-view of columns (a) C1, C2, C3, C4. (b) C5, C6, C7, C8. (c) C9, C10, C11, C12, and (d) C13, C14, C15, C16 *(all dimensions are in mm)*

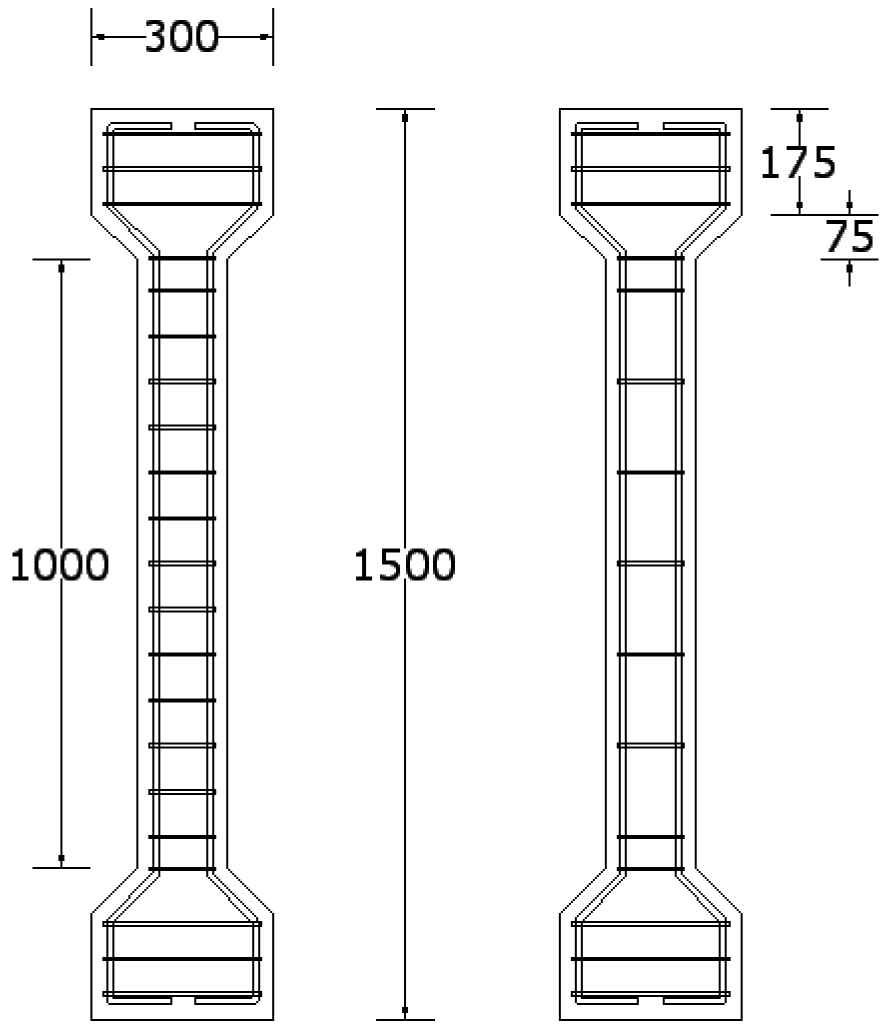


(a) (b)

Figure 3.2: Cross-sectional view of specimen (a) C3, C4, C7, C8, C11, C12, C15, C16 and (b) C1, C2, C5, C6, C9, C10, C13 C14 *(all dimensions are in mm)*

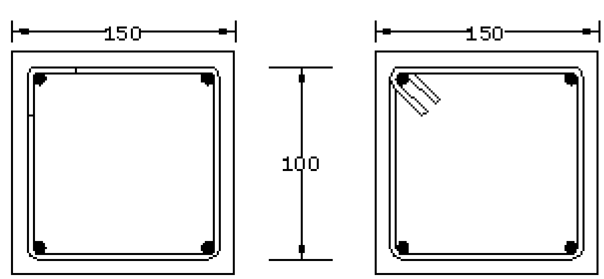
Table 3.2: Reinforcement detail of Group 'B' columns

Column	Longitudinal bars		Transverse ties		
	Bar diameter (mm)	Number	Bar diameter (mm)	Spacing (mm)	Hooks
B1	10	4	4.76	75	90˚
B2	10	4	4.76	150	90˚
B3	10	4	4.76	75	135˚
B4	10	4	4.76	150	135˚



(a) (b)

Figure 3.3: Elevation and Side-view of columns (a) B1, B3; and (b) B2, B4



(a) (b)

Figure 3.4: Cross-section of columns (a) B1, B2; and (b) B3, B4

3.3 Preparation of Test specimens

All sixteen columns of group ‘A’ specimens with different hoop angle and ties spacing were cast in steel moulds in horizontal position while four specimen of group ‘B’ were casted in *35mm* particle board in horizontal position. All the specimen were cured for 28 days.

3.3.1 Material

HYSD steel bars of 8mm diameter were used for group ‘A’ columns and 10 mm diameter were used for group ‘B’ columns as longitudinal reinforcement and 4.76 mm diameter reinforcement were used as transverse ties for all specimen. The target 28-days cube compressive strength of concrete was 20MPa.

3.3.1.1 Reinforcement

The reinforcement steel used in all specimens confirm to Fe415 grade. The results of tensile test of sample are shown in table 3.3.

Table 3.3: Tensile strength of steel bar

Bar diameter (mm)	Strength (MPa)	
	Yield (kg/cm2)	Ultimate(kg/cm2)
4.76	4953.82	5645.15
8	4915.87	5603.03
10	5013.73	5708.75

3.3.1.2 Cement

For the entire specimen, OPC cement, 53 grade, verified by Nepal Standard (NS) was used from a single lot.

3.3.1.3 Concrete

The specimens were made using concrete of target compressive strength (f_{ck}) of *20 MPa*.

Sand

Sand was washed to removed clay and other soluble particle (Figure 3.5 a). The sand grading was done using sieve analysis. After analysis the sand is found to lies on zone II. The sand grading results are shown in table 3.4.

Table 3.4: Grading of sand

Sieve	Retain	%Retain	Cumulative Passing%
4.75mm	2gm	0.2%	99.8%
2.36mm	15gm	1.5%	98.3%
600μ	480gm	48%	50.3%
300 μ	347gm	34.7%	15.3%
Pan	156gm	15.6%	0%

Aggregate

Crushed aggregate was used. Aggregate was washed and dried (Figure 3.5 b). For group 'A' specimen 10mm crushed aggregate was used whereas for group 'B' specimen 20mm crushed aggregate was used.

(a) (b)

Figure 3.5: Preparation of material (a) washing of sand (b) washing of aggregate

Mix design

Mix design was done as per IS 10262:1982.

Mix design for10mm crushed aggregate and sand of zone II.

1 : 1.445 : 2.356 : 05

Mix design for 20mm crushed aggregate and sand of zone II.

1 : 1.509 : 3.041 : 0.5

Cube

For every batch of concrete mix, six numbers of cubes were prepared. The cubes are compacted on vibrator machine (Figure 3.6 a) and it was cured on curing tank for 28 days. After 28 days the cubes are tested on UTM (Figure 3.6 b). The cube test result are presented in table 3.5 and 3.6.

(a)

(b)

Figure 3.6: (a) Preparation of Cubes and (b) Cube test.

Table 3.5: Compressive strength of concrete cubes of Group 'A' specimen

Cube	Cube Compresssive Strength at 28 days (day of test) Mpa
1	27.7
2	25.3
3	26.8
4	26.2
5	24.4
6	26.3

Table 3.6: Compressive strength of concrete cubes of Group 'B' specimen

Cube	Cube Compresssive Strength at 28 days (day of test) Mpa
Specimen B1 and B2	
1	25.6
2	23.5
3	23.8
4	24.4
5	23.6
6	24.3
Specimen B3 and B4	
1	23.4
2	23.6
3	22.4
4	24.3
5	24.3
6	23.9

3.3.1.4 Casting

Two different groups of specimens are casted in different mould.

Group 'A' Specimens

In group 'A' sixteen columns are prepared for cast. All the specimens were cast in horizontal position in a steel mould (Figure 3.7c). The reinforcement case (Figure 3.7a) was put inside the steel mould (Figure 3.7b) maintaining a 25mm cover on all sides of cage. The mould was prepared as per Figure 3.1 and Figure 3.2. The casting of specimens was done in single batch and the needle vibrator was used for compaction. After 24 hours of casting, the mould was open and it is cured to the curing tank in normal room temperature for next 28 days. The cubes were cured in curing tank.

(a)

(b) (c)

(d) (e)

Figure 3.7: Detail of Casting Group ‘A’ Specimens (a) Reinforcement cage, (b) Steel Mould, (c) Preparing specimen (d) specimen of group ‘A’ and (e) Curing of specimens

Group ‘B’ Specimens

In group ‘B’ four columns with column head are prepared for cast. All the specimens in this group were cast in horizontal position in a wooden mould (Figure 3.8 a) The reinforcement case (Figure 3.8 c) was put inside the wooden mould maintaining a 25 mm cover on all sides of cage. The mould was prepared as per Figure 3.3 and Figure 3.4. The casting of specimens was done in two batches and the needle vibrator (Figure 3.8 b) was used for compaction. After 24 hours of casting, the side moulds were opened (Figure 3.8 d), the column covered with jute bags and moist curing was performed for next 28 days. The cubes were cured in water tank.

(a)

(b)

(c) (d)

Figure: 3.8 Detail of Casting Group 'B' specimens (a) Wooden Mould, (b) Preparation of specimen, (c) Reinforcement cage (d) Specimen of group 'B'

3.4 Analytical Estimates

Before conducting the experiments, analytical estimates were obtained to know the level of loads and deformations to be applied on the specimen.

3.4.1 Axial Load

The axial load capacity of a column depends upon the axial capacities its longitudinal reinforcement and concrete. The reference axial capacity of an RC column under axial load is calculated (Table 3.7)

Table 3.7: Reference axial capacities of specimens

Column Specimens	**Pcal (KN)**
Group 'A' Specimens	250.02
Group 'B' Specimens	507.53

3.5 Test Procedure

In the current study, the column specimens were tested in Universal Testing Machine (UTM). The test specimen was place inside the test frame and a thin layer of Plaster of Paris was coated on the top surface to ensure uniform contact with the loading head. Column was centered using spirit level. All the specimen was subjected to concentric monotonically increasing axial load. Dial gauge were placed in both sides of column, to measure displacement. The results are discussed in detail in Chapter 4.

Chapter 4

Result and Discussion

4.1 General

The overall load displacement curve for the column specimens are obtained experimentally following the test procedure discussed in Chapter 3. The behavior of column in the elastic and post elastic range is the focus of study. In post elastic range of response, the ductility becomes important and the effectiveness of the transverse reinforcement plays an important role.

4.2 Experimental Results

The RC columns were loaded axially and concentrically up to the point of total fracture of each specimen. The response quantities that were studied consisted of the maximum load carrying capacity, displacement ductility and the behavior of column in failure. These parameters consequently paved the way for a comparative study of the effectiveness of the hook angle and the spacing of the ties as a confining element in RC column.

4.3 Group 'A' specimens

The detail of the Group 'A' specimens are presented in Chapter 3. Overall sixteen columns are tested in this group. For each variation in hook angle and spacing two columns are casted. Hence the behaviors of columns are almost same for each two columns.

4.3.1 Specimen C1and C2

These columns are casted with hook angle 90° and ties spacing 50mm.

4.3.1.1 Progression of Damage

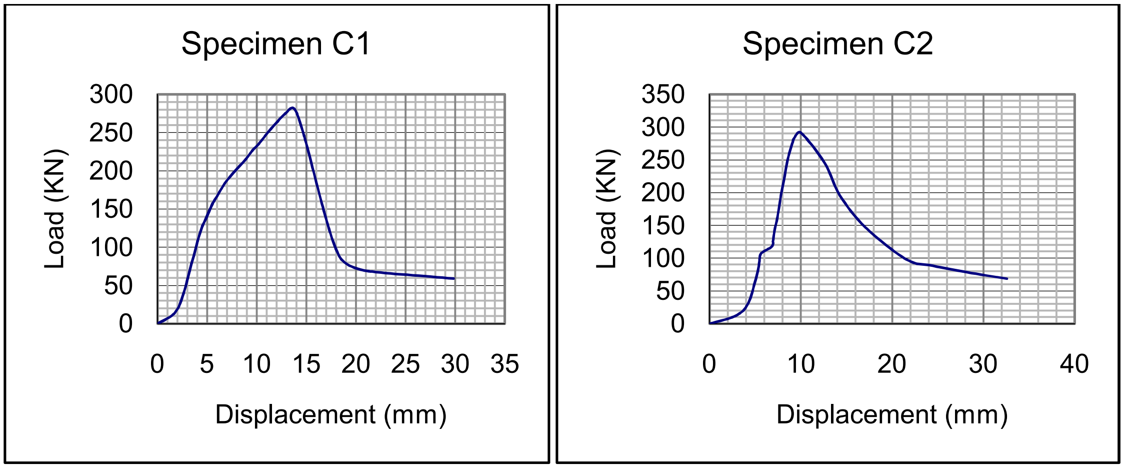
The progression of damage in the specimen (Figure 4.1 a&c) was initiated in top of the column. The concrete cover spalled off and the transverse ties are opened (Figure 4.1 b &c). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C1 and C2 were 280.56 KN and 291.35 KN and the corresponding displacement were 13.8 mm and 10 mm respectively.

(a) (b) (c) (d)

Figure 4.1: (a) Test setup Specimen C1, (b) Pulling out of 90° ties in specimen C1, (c) Test setup specimen C2 and (d) Progression of damage in specimen C2

4.3.1.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure 4.2 a and b).



(a) (b)

Figure 4.2: Load displace curve of (a) specimen C1 and (b) specimen C2

4.3.2 Specimen C3and C4

These columns are casted with hook angle 135° and ties spacing 50mm.

4.3.2.1 Progression of Damage

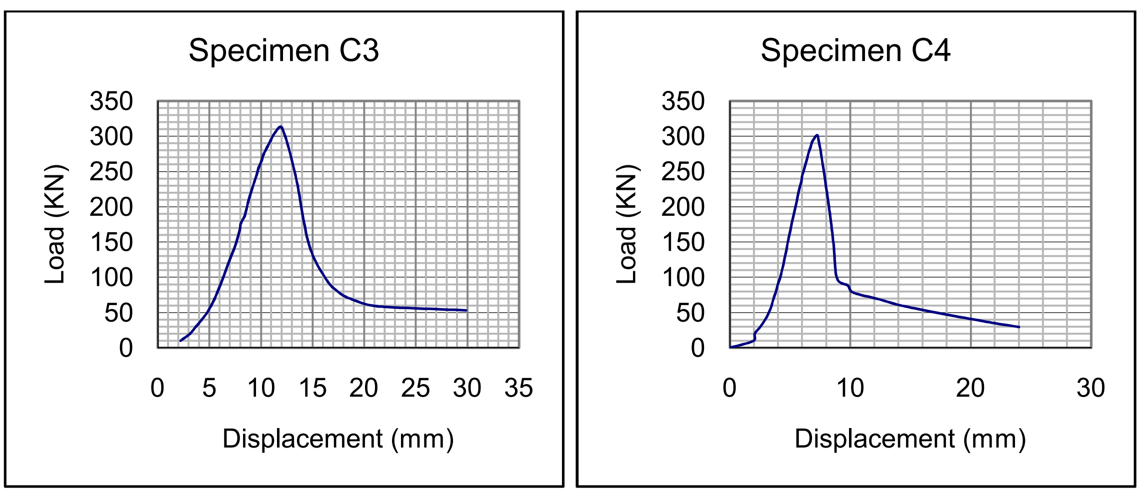
The progression of damage in the specimen (Figure 4.3 a&c) was initiated in top of the column. The concrete cover spalled off and the longitudinal bars are buckled (Figure 4.3 b &c). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C3 and C4 were 311.95 KN and 300.18 KN and the corresponding displacement were 12.05 mm and 7.3 mm respectively.

(a) (b) (c) (d)

Figure 4.3: (a) Test setup Specimen C3, (b) Progression of damage in specimen C3, (c) Test setup specimen C2 and (d) Progression of damage in specimen C4

4.3.2.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure 4.4 a and b).



(a) (b)

Figure 4.4: Load displace curve of (a) specimen C3 and (b) specimen C4

4.3.3 Specimen C5and C6

These columns are casted with hook angle 90° and ties spacing 100mm.

4.3.3.1 Progression of Damage

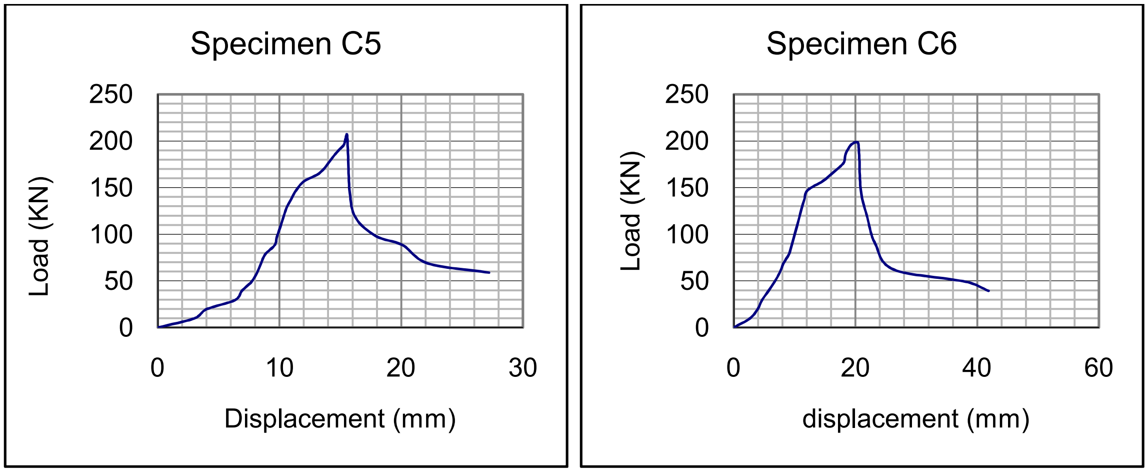
The progression of damage in the specimen (Figure 4.5 a&c) was initiated in top of the column. The concrete cover spalled off, transverse ties hoop opened and the longitudinal bars are buckled (Figure 4.5 b &c). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C5 and C6 were 206.01 KN and 197.18 KN and the corresponding displacement were 15.55 mm and 20.45 mm respectively.

(a) (b) (c) (d)

Figure 4.5: (a) Test setup Specimen C5, (b) Progression of damage in specimen C5, (c) Test setup specimen C6 and (d) Progression of damage in specimen C6

4.3.2.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure).



(a) (b)

Figure 4.6: Load displace curve of (a) specimen C5 and (b) specimen C6

4.3.4 Specimen C7and C8

These columns are casted with hook angle 135° and ties spacing 100mm.

4.3.4.1 Progression of Damage

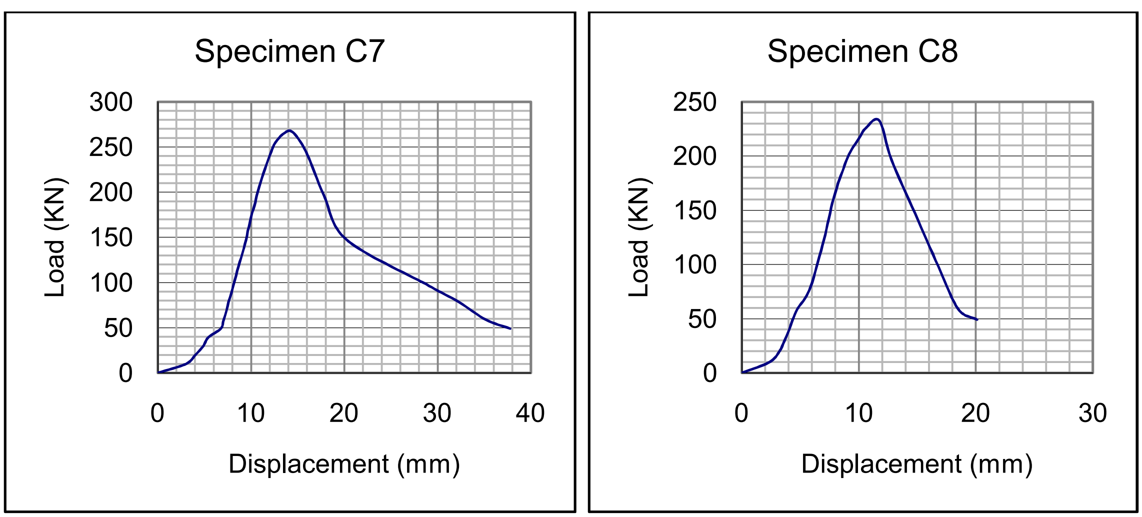
The progression of damage in the specimen (Figure 4.7 a&c) was initiated in bottom of the column in specimen C7 and at the top of the column in specimen C8. The concrete cover spalled off and the longitudinal bars are buckled (Figure 4.7 b&d). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C7 and C8 were 266.83 KN and 232.49 KN and the corresponding displacement were 14.4 mm and 11.75 mm respectively.

(a) (b) (c) (d)

Figure 4.7: (a) Test setup Specimen C7, (b) Progression of damage in specimen C7, (c) Test setup specimen C8 and (d) Progression of damage in specimen C8

4.3.4.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure 4.8).



(a) (b)

Figure 4.8: Load displace curve of (a) specimen C7 and (b) specimen C8

4.3.5 Specimen C9and C10

These columns are casted with hook angle 90° and ties spacing 150mm.

4.3.5.1 Progression of Damage

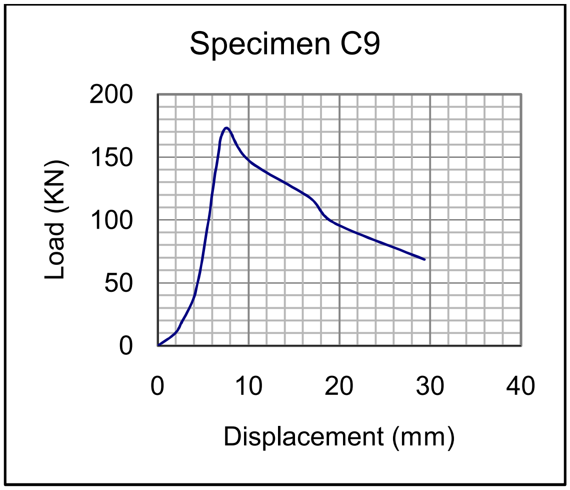
The progression of damage in the specimen (Figure 4.9 a&c) was initiated in top of the column. The concrete cover spalled off, ties were opened and the longitudinal bars are buckled (Figure 4.9 b&d). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C9 and C10 were 172.65 KN and 175.59 KN and the corresponding displacement were 7.75 mm and 7.61 mm respectively.

(a) (b) (c) (d)

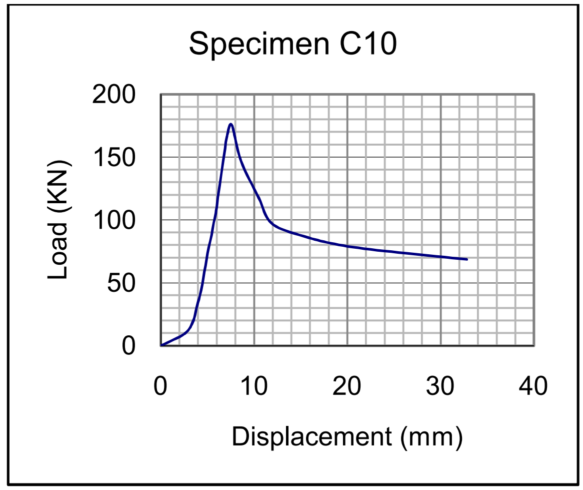
Figure 4.9: (a) Test setup Specimen C9, (b) Progression of damage in specimen C9, (c) Test setup specimen C10 and (d) Progression of damage in specimen C10

4.3.5.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure 4.10).



(a)



(b)

Figure 4.10: Load displace curve of (a) specimen C9 and (b) specimen C10

4.3.6 Specimen C11and C12

These columns are casted with hook angle 135° and ties spacing 150mm.

4.3.6.1 Progression of Damage

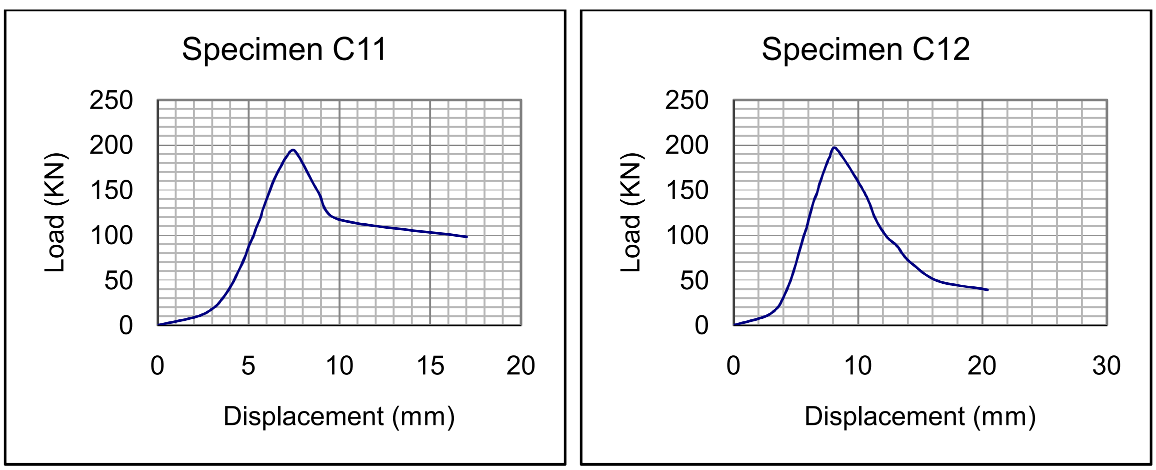
The progression of damage in the specimen (Figure 4.11 a&c) was initiated at the top of the column in specimen C11 and the bottom of the column in specimen C12. The concrete cover spalled off and the longitudinal bars are buckled (Figure 4.11 b&d). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C11 and C12 were 193.25 KN and 196.20 KN and the corresponding displacement were 7.55 mm and 8.22 mm respectively.

(a) (b) (c) (d)

Figure 4.11: (a) Test setup Specimen C11, (b) Progression of damage in specimen C11, (c) Test setup specimen C12 and (d) Progression of damage in specimen C12

4.3.6.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure 4.12).



(a) (b)

Figure 4.12: Load displace curve of (a) specimen C11 and (b) specimen C12

4.3.7 Specimen C13and C14

These columns are casted with hook angle 90° and ties spacing 200mm.

4.3.7.1 Progression of Damage

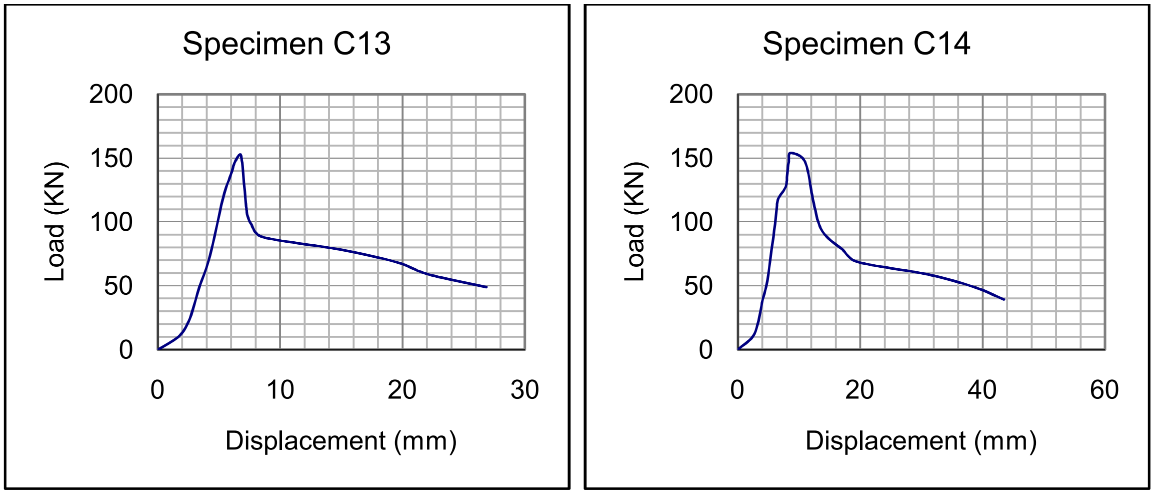
The progression of damage in the specimen (Figure 4.13 a&c) was initiated in top of the column. The concrete cover spalled off and the diagonal cracks are developed. These cracks slowly widened and eventually developed into a well define diagonal share failure. The longitudinal bars are buckled (Figure 4.13 b&d). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C13 and C14 were 152.05 KN and 154.01 KN and the corresponding displacement were 6.8 mm and 8.6 mm respectively.

(a) (b) (c) (d)

Figure 4.13: (a) Test setup Specimen C13, (b) Progression of damage in specimen C13, (c) Test setup specimen C14 and (d) Progression of damage in specimen C14

4.3.7.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure 4.14).



(a) (b)

Figure 4.14: Load displace curve of (a) specimen C13 and (b) specimen C14

4.3.8 Specimen C15 and C16

These columns are casted with hook angle 135° and ties spacing 200mm.

4.3.8.1 Progression of Damage

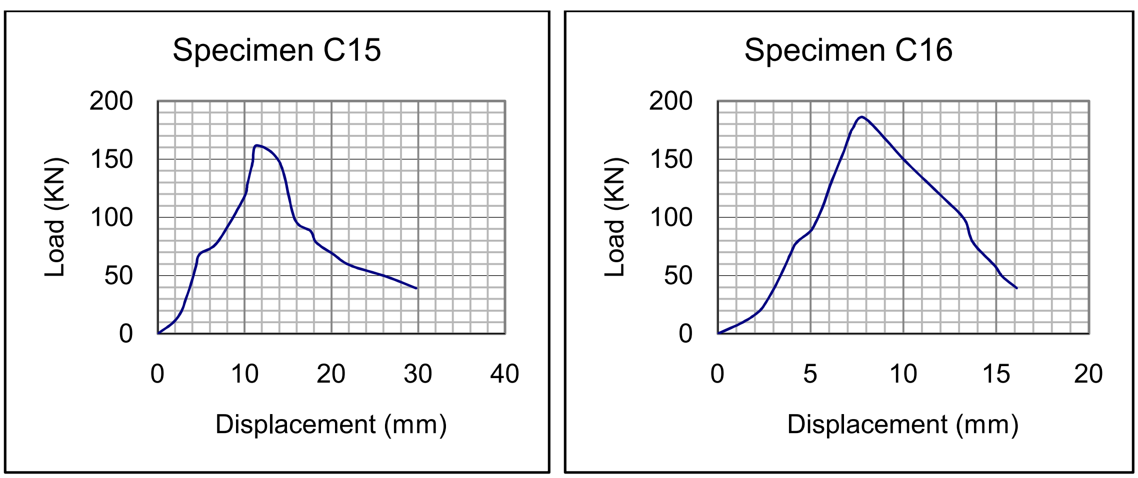
The progression of damage in the specimen (Figure 4.15 a&c) was initiated in top of the column. The concrete cover spalled off and the diagonal cracks are developed. These cracks slowly widened and eventually developed into a well define diagonal share failure. The longitudinal bars are buckled (Figure 4.15 b&d). This resulted in a sudden drop of load carrying capacity in the specimen. The peak load attained by the specimen C15 and C16 were 161.86 KN and 185.40 KN and the corresponding displacement were 11.4 mm and 7.9 mm respectively.

(a) (b) (c) (d)

Figure 4.15: (a) Test setup Specimen C15, (b) Progression of damage in specimen C15, (c) Test setup specimen C16 and (d) Progression of damage in specimen C16

4.3.8.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load-deformation curve (Figure 4.16).



(a) (b)

Figure 4.16: Load displace curve of (a) specimen C15 and (b) specimen C16

4.4 Comparison of Result and Discussion of Group 'A' column

The sixteen specimens tested showed different behavior it terms of load carrying capacity and the displacement. Since the number, diameter and the spacing of the longitudinal bars were same in each of the columns, the longitudinal reinforcement ratio had no role to play in the observed modes of differences. Therefore, the transverse reinforcement was key factor for such varied performance of RC columns. Hence the effect of hook angle and the spacing of transverse reinforcement as confining elements are now discussed.

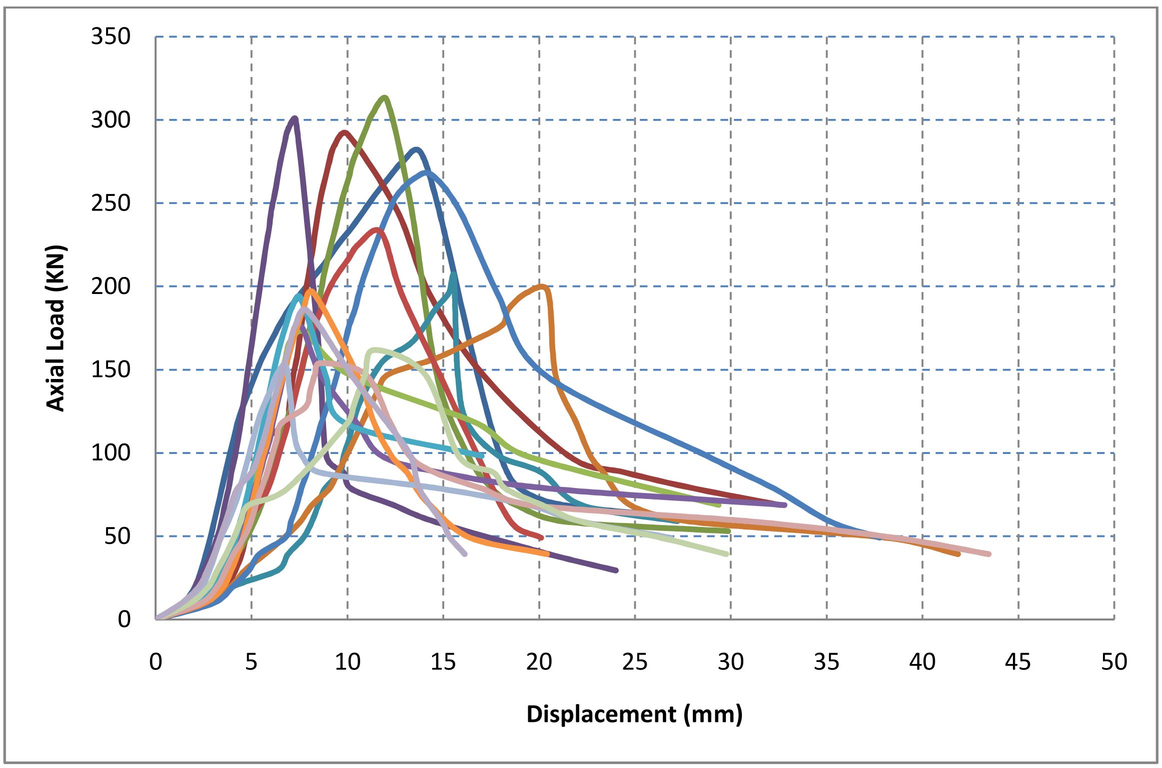


Figure 4.17: comparison of the experimental load-displacement behavior of the test specimen of group 'A'

The load carrying capacity of columns of small spacing of transverse ties and the hook angle of 135° ties is high where as the columns of large spacing of transverse ties and the hook angle of 90° ties have relatively low load carrying capacity. The displacement of the different specimens are not shows the relevant result. This is because the columns are casted without column head and the particle boards are used at top and bottom of the column at test. All the specimens are failed either at the top or bottom of the specimen. This is caused due to local crushing of the column. Also the particle boards are crushed and it causes the unwanted data for displacement. Although the displacement data are not relevant the load carrying capacity of the column can predicted from the experiment.

Mean peak load of the column having 90° hook angle and 50 mm spacing was 285.95 KN where as the column having 135° hook angle and 50 mm spacing was 306.06 KN. This shows the column having 135° hook angle have more load carrying capacity than the column of 90° hook angle ties. This is 7.03% increment. Similarly the mean load carrying capacity of the column of 90° hook and 100 mm spacing was 201.59 KN and the column of same spacing and 135° ties was 249.66 KN. This is 23% increment in load carrying capacity. The column of 90° ties and 150 mm spacing carry the peak load of 174.62 KN and 135° ties and 150 mm spacing carry the peak load of 194.72 KN. This is 11.51% of increment of load carrying capacity. Likewise the load carrying capacity of column of 90° ties and 200 mm spacing of ties is 153.03 KN and the load carrying capacity of column of 135° ties and 200 mm spacing of ties is 173.63KN. This is 13.46% increment of load carrying capacity.

The column of same hook angle ties and different spacing shows the different behavior. After experiment, it was observed that the small spacing have the higher load carrying capacity and the large spacing have relatively low load carrying capacity. The load carrying capacity of column of 90° ties and 50mm, 100mm, 150mm and 200mm spacing are 285.95, 201.59, 174.62 and 153.03 respectively. Decreasing the spacing to 200 to 150mm, 14.10% load carrying capacity was increased, similarly decreasing the spacing 150 to 100 mm, 15.44%

load carrying capacity increased, likewise decreasing the spacing 100 to 50 mm, 41.94% load carrying capacity increase in same 90° ties.

Similarly, the load carrying capacity of column of 135° ties and 50mm, 100mm, 150mm and 200mm spacing are 306.06, 249.66, 194.72 and 173.63 respectively. Decreasing the spacing to 200 to 150mm, 12.14% load carrying capacity was increased, similarly decreasing the spacing 150 to 100 mm, 28.21% load carrying capacity increased, likewise decreasing the spacing 100 to 50 mm, 22.59% load carrying capacity increase in same 135° ties.

4.5 Group ‘B’ specimens

The detail of the Group ‘B’ specimens are presented in Chapter 3. Four columns are tested in this group. Column head are provided to minimize the probable local crushing as seen group ‘A’ columns.

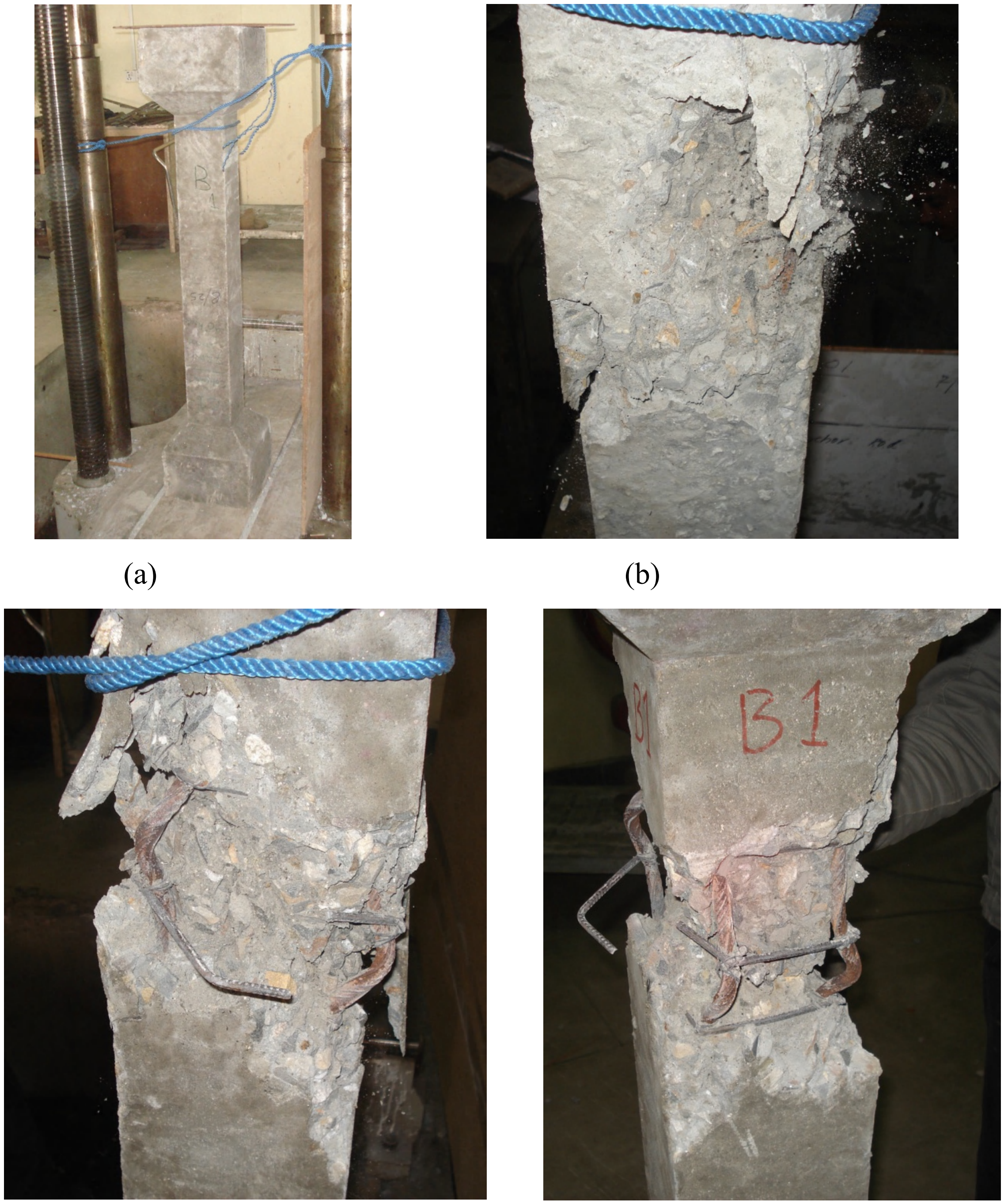
4.5.1 Specimen B1

This column is casted with hook angle 90° and ties spacing 75mm.

4.5.1.1 Progression of Damage

The progression of damage in the specimen (Figure 4.18 a) was initiated by horizontal crack, followed by vertical cracks as well as diagonal cracks. The concrete cover spalled off (Figure 4.18 b) was very short. After the cover spalled off, the transverse ties were unable to confine the core concrete. The ties opened (Figure 4.18 c) and hence sudden drop of the load carrying capacity in the specimen. The peak load attained by the specimen was 372.78 KN and corresponding displacement was 11.58 mm.

At failure, it was also observed that the longitudinal bars had buckled (Figure 4.18 d) and the ties slipped against the longitudinal bars.



(a) (b)

(c) (d)

Figure 4.18: (a) Specimen B1 (b) Progression of damage with concrete cover spalled off (c) 90° ties opened and (d) damage of specimen with buckling of longitudinal bars and ties totally opened.

4.5.1.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load – axial displacement curve (Figure 4.19).

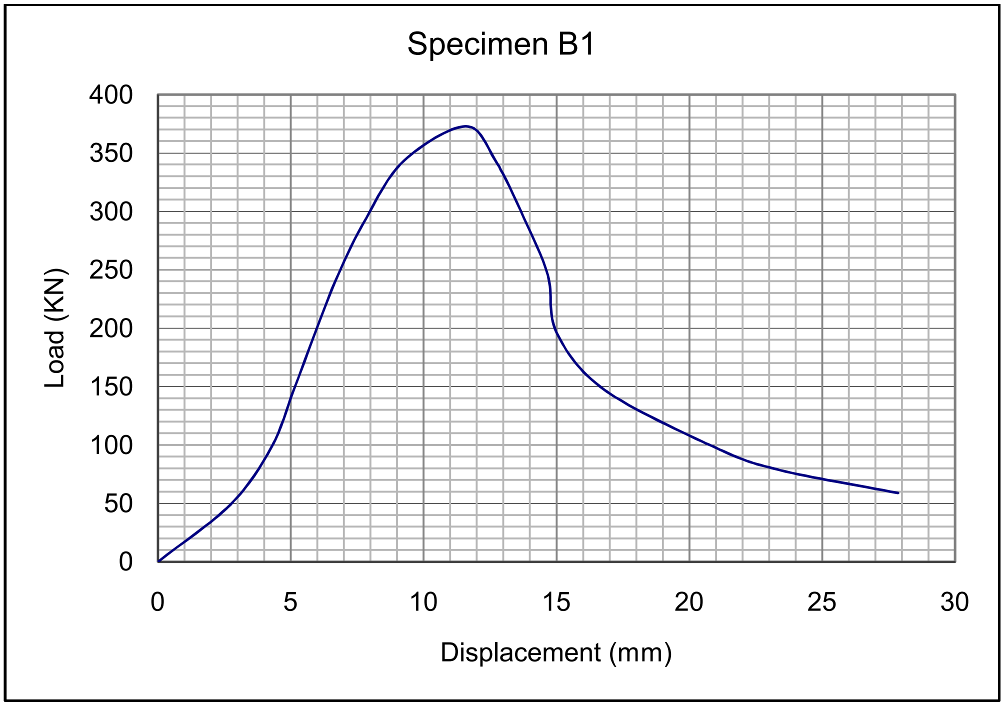


Figure 4.19: Load displace curve of Specimen B1

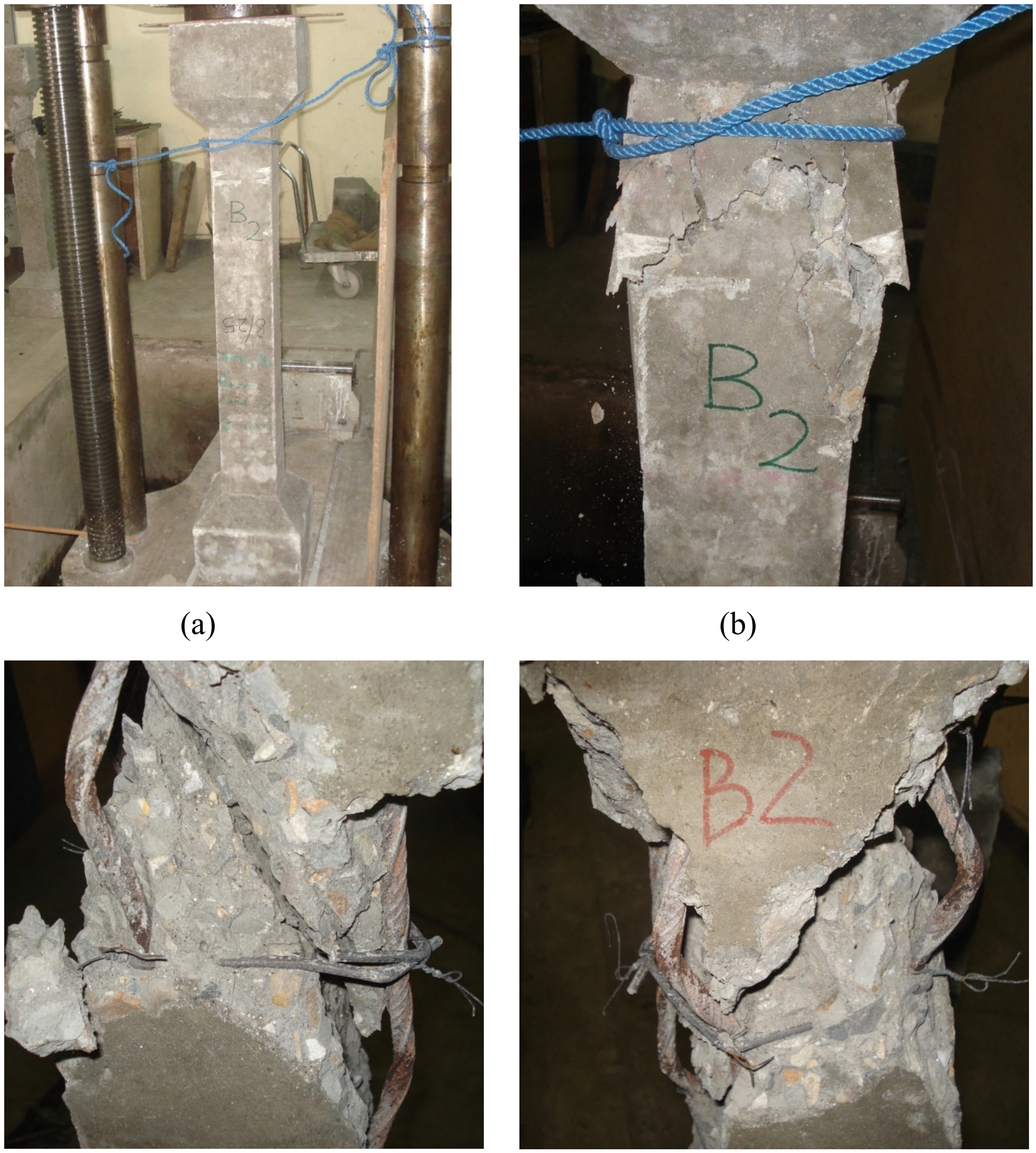
4.5.1 Specimen B2

This column is casted with hook angle 90° and ties spacing 150mm.

4.5.1.1 Progression of Damage

The progression of damage in the specimen (Figure 4.20 a) was initiated by horizontal crack, followed by vertical cracks (Figure 4.20 b) as well as diagonal cracks. The cracks slowly widened and eventually developed into a well defined diagonal shear failure plane (Figure 4.20 c). The gap between the appearance of first crack and spalling of the cover concrete was very short. After the cover spalled off, the transverse ties were unable to confine the core concrete. The ties opened (Figure 4.20 d) and the longitudinal bars started to buckled. This result, sudden drop of the load carrying capacity in the specimen. The peak load attained by the specimen was 309.015 KN and corresponding displacement was 9.05 mm.

The longitudinal reinforcement bar buckled was large and the ties opened at the failure.



(a) (b)

(c) (d)

Figure 4.20: (a) Specimen B2 (b) Progression of damage with concrete cover spalled off (c) diagonal crack widening and ties opened (d) damage of specimen with buckling of longitudinal bars and ties totally opened

4.5.2.2 Load Displacement Behavior

The load displacement behavior of the specimen is presented as axial load – axial displacement curve (Figure 4.21).

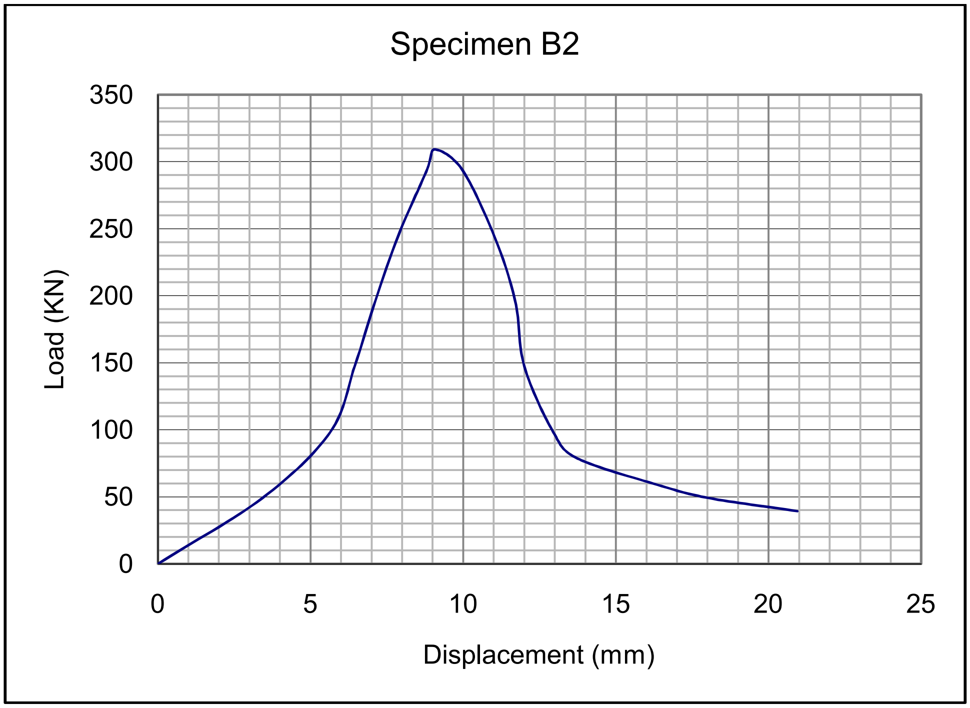


Figure 4.21: Load displace curve of Specimen B2

4.5.3 Specimen B3

This column is casted with hook angle 135° and ties spacing 75mm.

4.5.3.1 Progression of Damage

The damage started with similar cracks as in specimen B1. The difference was the ties are not opened like in specimen B1. The ties are not opened and the longitudinal reinforcement started to buckled. This proves that after the cover concrete spalled off, the ties of 135° bend confined the core concrete and provide better restrain to the column against sudden failure. The maximum load carried was 433.60 KN and the corresponding displacement was 7.4 mm.

The longitudinal bars buckled in this column too, but the extent of the bulking was less. When the load was very high, the core concrete tended to dilate outward, and the ties with 135° hook prevented this type of behavior.

(a)

(b)

(c) (d)

Figure 4.22: (a) Specimen B3 (b) Progression of damage with concrete cover spalled off and diagonal shear crack (c) Buckling of longitudinal bars (d) damage of specimen with buckling of longitudinal bars and ties tend to open

4.5.2.2 Load Displacement Behavior

The load displacement behaviour of the specimen is presented as axial load – axial displacement curve (Figure 4.23).

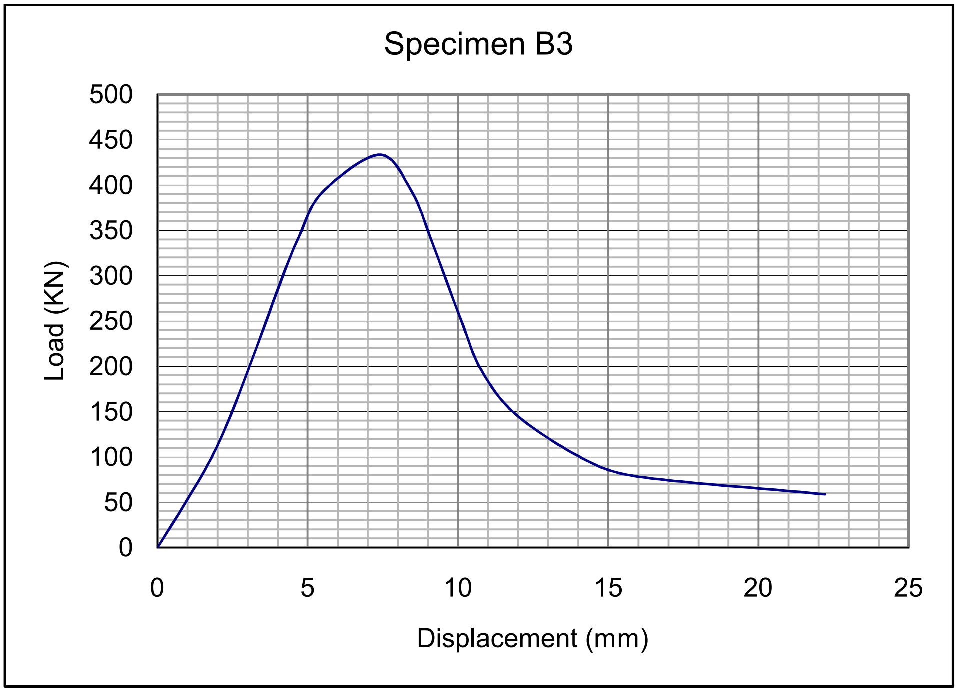


Figure 4.23: Load displace curve of Specimen B3

4.5.3 Specimen B4

This column is casted with hook angle 135° and ties spacing 150mm.

4.5.3.1 Progression of Damage

The horizontal and the vertical cracks appear in this column as the load was reaching the peak value. This column display nearly same behavior as specimen B2. A band of cracks appear and finally a diagonal shearing failure plane was formed. Longitudinal bars start buckled. Ties are not opened in this specimen. Finally column damage, buckling of longitudinal bars.(Figure 4.24)

(a)

(b)

(c)

(d)

Figure 4.24: (a) Specimen B4 (b) Progression of damage with concrete cover spalled off (c) Diagonal shear crack and longitudinal bars start buckled (d) damage of specimen with buckling of longitudinal bars and shear failure

4.5.2.2 Load Displacement Behavior

The load displacement behaviour of the specimen is presented as axial load – axial displacement curve (Figure 4.25).

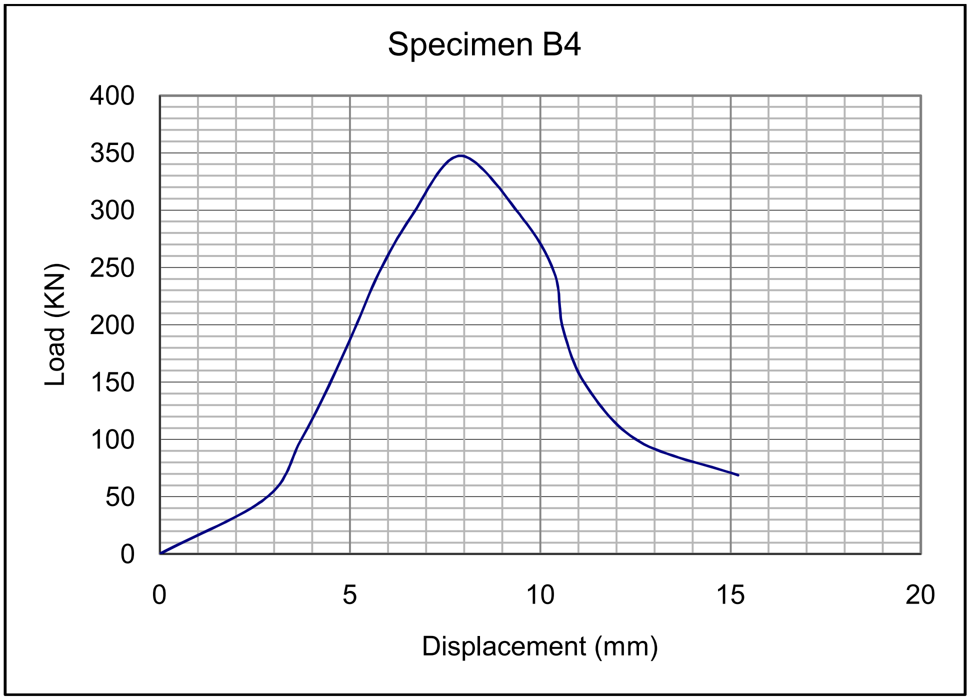


Figure 4.25: Load displace curve of Specimen B4

4.6 Comparison of Result and Discussion of Group 'B' column

Four specimens tested showed different behavior it terms of load carrying capacity and the displacement. Since the number, diameter and the spacing of the longitudinal bars were same in each of the columns, the longitudinal reinforcement ratio had no role to play in the observed modes of differences. Therefore, the transverse reinforcement was key factor for such varied performance of RC columns. Hence the effect of hook angle of ties and the spacing of transverse reinforcement as confining elements are now discussed (Figure 4.26)

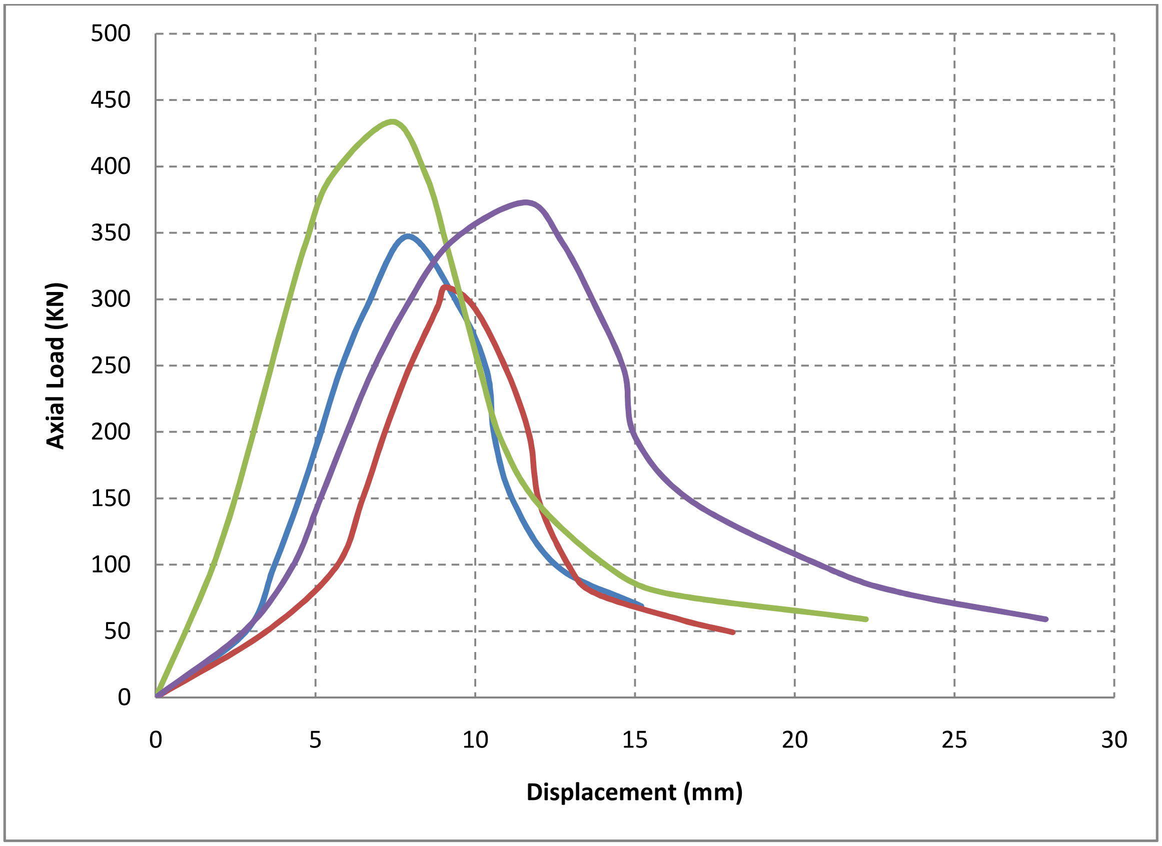


Figure 4.26: Comparison of experimental load – displacement behavior of the test specimens of group 'B'

The load carrying capacity of the column B1 is 372.78 KN and B2 is 309.01 KN. This specimen have same hook angle of ties but the spacing of ties are different. For the same 90° ties, spacing from 150 to 75, mm the load carrying capacity increases 20.63%. Similarly for the 135° ties, spacing from 150 to 75 mm, the load carrying capacity increase 24.85%.

Likewise the load carrying capacity of column of same spacing, but different in spacing of transverse reinforcement also differs. The load carrying capacity at 75 mm spacing and 90° ties is 372.78KN and 135° ties is 433.60KN. This is 16.31% increment. Similarly the load carrying capacity at 150mm spacing and 90° ties is 309.01KN and 135° ties is 347.27 KN. This is 12.38% increment in load carrying capacity.

Chapter 5

Summary and Conclusion

5.1 Summary

Sixteen small size column (100mm x 100mm x 600mm) and four medium size column (150mm x 150mm x 1000mm) were tested experimentally in the UTM under monotonic axial compression. Test variables were hook angle of ties and the spacing of the ties. The following aspects were addressed in the study:

- Final strength of columns to determine the axial capacity enhancement due to confinement
- Effectiveness of ties angle in the post elastic range of behavior of the specimen
- Effectiveness of spacing of the transverse ties

5.2 Conclusion

The following salient conclusions are drawn from this experimental study:

a) Column with 135° hook ties shows the better performance in axial capacity than the column with 90° hook ties.

b) In small columns, the axial capacity of the column increase 7.03% in 135° ties and 50mm spacing than in the column with 90° ties and same spacing. Similarly, the axial capacity of the column increase 23%, 11.51% and 13.46% in 135° ties as compare to the 90° ties at the same spacing of 100 mm, 150mm and 200mm respectively.

c) Decreasing the spacing of the ties, the axial capacity increase. In small columns with same 90° ties, the axial capacity increase 14.10% in decreasing the spacing from 200 to 150mm, increase 15.44% in decreasing the spacing from 150 to 100mm and increase 41.84% in decreasing the spacing from 100 to 50mm.

d) In small columns with same 135° ties, the axial capacity increase 12.14% in decreasing the spacing from 200 to 150mm, increase 28.21% in decreasing the spacing from 150 to 100mm and increase 22.59% in decreasing the spacing from 100 to 50mm.
e) Column without column head starts damage from machine head
f) RC column with large spacing could not restrain the longitudinal bars from buckling.
g) In the medium size column the axial capacity of the column increase 16.31% in 135° ties and 75mm spacing than in the column with 90° ties and same spacing. And the axial capacity increase 12.27% in 135° ties and 150mm spacing than in the column with 90° ties and same spacing
h) In the medium size column with same 135° ties, the axial capacity increase 24.85% in decreasing the spacing from 150mm to 75mm. And the axial capacity increase 20.51% in the same decrement of spacing and the 90° ties.

5.3 Scope of Future Work

In the present study, test variables included hook angle of transverse ties and the spacing of ties. The effectiveness of the intermediate hoops and cross ties may be studied to provide a broader spectrum of information regarding ductility and axial capacity demand of RC columns. The longitudinal reinforcement ratio as well as effect of spacing of longitudinal bars may be studied. Effect of thickness of concrete cover may be studied.

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