EXPERIMENTAL INVESTIGATION AND NUMERICAL SIMULATION ON EFFECT OF DRILLED CORES ON AXIAL CAPACITY OF COLUMNS

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by

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The thesis titled "EXPERIMENTAL INVESTIGATION AND NUMERICAL SIMULATION ON EFFECT OF DRILLED CORES ON AXIAL CAPACITY OF COLUMNS" submitted by Md. Ayatullah Khomeni, Student number-1014042322F and Session: October/2014 has been accepted as a satisfactory in partial fulfillment of the requirement of the degree of Master of Science in Civil Engineering (Structural) on September 29, 2018.

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ABSTRACT

Drilling out of core from columns now a days is one of the most established and reliable practice to assess the in-situ concrete strength of structural members, especially columns, which is one of the most important structural member of any RCC structure. The tragic collapse of Rana Plaza in 2013 and some other structural failures due to poor construction has triggered structural integrity assessment of existing RCC structures in Bangladesh. Besides, it is quite common in this country, a significant number of structures have been constructed without following any code provision. In addition, recent earthquakes are also causing concerns among engineers to check the adequacy of the existing structures. Besides, this test is performed when doubt exists about the in-place concrete quality due to either low strength test results during construction or signs of distress in the structure and also used to assess strength information on older structures. In this regards, it is now quite common that buildings are being recommended for evaluation of concrete in-built strength through core extraction. There are codes that provide guideline regarding the procedure of core drilling. However, core drilling has the potential to impose a negative effect on the structural capacity of the element being drilled for cores. The relevant ASTM standard also advises that concerned engineers must be aware of the associated impact of core drilling. Despite such potential negative impact, there have been few studies available on the effect of core extraction on the structural capacity of RC columns. In these circumstances, a comprehensive study has been undertaken to investigate the effect of core extraction on capacity of RC columns. The effect of compressive strength of concrete, column size, lateral tie spacing, core size and locations were investigated on structural capacity of column after core removal.

The study was divided into two phases. In the first phase, sixteen lab-scale columns were made and their ultimate load capacity and crack failure pattern have been observed for two core locations and compared with columns with no cores drilled. The experimental columns were made with stone aggregate for target strength of 27.6 MPa to represent a ubiquitous concrete mix. In the second phase, finite element analysis (FEA) of tested column specimens has been conducted using ABAQUS environment and compared with the experimental outcomes for validation. After satisfactory validation of the FEA models, further FEA have been performed on columns having real scale dimensions for different core sizes, compressive strengths, column sizes and different lateral confinement in order to develop a quantitative guideline for safe core extraction.

It was observed that core drilled from one third height from support of the column resulted in significantly higher reduction in column capacity than that of column with core at mid height. The behavior of core drilled columns shows a significant dependency on core diameter particularly when column size is small. The effect of core drilling was found to be more pronounced for low strength columns and higher tie spacing. A noteworthy observation was obtained from the study that columns having dimension smaller than 300mm x 450mm would require special consideration for core extraction regardless of tie bar spacing, core size and concrete strength. Finally, graphical guidelines have been prepared based on combined interrelation of all the parameters for the entire range of column dimensions, tie bar spacing and compressive strength. The parameters required to utilize the graphs can be obtained from design drawings or from on-site investigations.

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List of Abbreviations and Symbols

fc'	Compressive strength of concrete
NC	Normal Column
CMD	Core mid height
COD	Core one third height
CLN	Concrete lean mix
NS	Non Shrinkage mortar grout
OPC	Ordinary Portland cement
SSD	Saturated Surface Dry
NDTs	Non Destructive Tests
DTs	Destructive Tests
ASTM	American Society of Testing Materials
UTM	Universal Testing Machine
FE	Finite Element Analysis
IMRF	Intermediate Moment Resisting Frame
CDP	Concrete Damage Plasticity
E	Young's Modulus
μ	Poisson ratio
m	Eccentricity
β	Dilation angle
γ	Viscosity
σ	Stress

Chapter 1

INTRODUCTION

1.1 General

Structures that are designed to serve for a certain load of occupancy later used for other purposes are quite common around the world especially in Bangladesh. Sometimes qualities are not maintained at job site due to poor supervision which may lead unprecedented hazards of structural safety in several instances. Buildings are designed without following code properly making the structures more at risk. Thereby, structures which are undergoing those types of irregularities are vulnerable from low to high level of structural failure. For instances, failure of Rana plaza may be one of the best example of that kind of failure which already take a lot of peoples life (Manzur et al., 2017). The particular structural failure raises consciousness among engineers to assess Ready Made Garment (RMG) building and other vulnerable structures all over the country. In addition, an in-depth detail engineering assessment (DEA) become crucial for buildings constructed without following seismic (Masi et al., 2012) and other type of lateral loads. A comprehensive evaluation of the overall structure is necessary to DEA as it is an economic and analytical effort for an accurate structural capacity investigation. Material property is one of the key assessment parameters which help to investigate in-built compressive strength of reinforced concrete (RC) structures. According to some codes (e.g. NTC, 2008 [4]; CEN EC8-3, 2005[5]; ACI 228, 1998), evaluation of in-situ concrete strength by both Non-Destructive Tests (NDT) and Destructive Tests (DT) are necessary to understand the material properties. Drilling out a core from the column is one of the most widely known methods for the determination of strength parameters of RC structures. The core drilling test considered as the easy and accurate technique and gained considerable applications in rehabilitation and examinations of structural failures (Lei et al., 2010). However, core drilling may have some adverse impacts on capacity of structural elements which creates concerns among practicing engineers. ASTM indicates some standard procedure ASTM C42, 2004of drilling out a core and testing in laboratory, but it does not refer any indication on column capacity due to core extraction. Also study on effect of column due to core drilling can be seldom found. Among few studies, it has been found from Calavera et al., 1979 and Masi et al., 2012that the effect of restoration is not much effective of low strength concrete.

Also a further study has been conducted by Siddique and Khomeni, 2014where it was found that the core extraction has pronounced effect on column capacity. The building needs safety assessment by core drill method as a process of valuation of strength parameters. Hence a large number of buildings need a wide range of core samples for DEA Masi et al., 2012.The cores drilled from large number of columns may lead an unanticipated risk of the structure. It undergoes a detail quantitative study on effect of core extraction on structural elements especially columns. Hence, an immense study has been undertaken for the evaluation of core drilled effect on columns. Several parameters such as concrete strength, column size, core size, core location, tie bar spacing were considered to the effect after removal of core. First phase of this study is to do several experimental analysis and then validation of FEA model. Then the second phase is to do a parametric study considering above mentioned parameters to provide a guideline. This quantitative guideline in the form of graph will assist a practicing engineer for safe core drilling from an existing concrete column.

1.1.1 Column Core Test

Column core cutting test is done when doubt exists about the in-place concrete quality due either to low strength test results during construction or signs of distress in the structure and also used to provide strength information on older structures. Core test of column involves drilling out a cylindrical concrete specimen out of a member and testing it using standard laboratory procedures to assess the strength parameters of the column materials. According to ACI 318-11 section 5.6.5, the average strength obtained from the core test is expected to be 85 percent and no single core is less than 75 percent of the specified concrete strength fc'.

1.2 Scope of the Study

The outcomes of this research are subject to some limitations in some context. The models are generated only for regular shaped columns which are purely under axial load and subjected to only regularly distributed compressive pressure. Though columns are primarily subject to compressive loading, but outermost columns of a structure are generally subject to some bending moment and columns in reality may be subject to eccentric loading which is ignored in these models. The lateral loads like wind load that may be exerted upon the column which imposes shear stress is also ignored in this study. Moreover, the columns have been casted horizontally which ensured uniform casting throughout the height of columns. On the other hand, vertical casting have some disadvantages over horizontal casting such as rough surfaces, inclusion of water, honeycomb in concrete, uneven mixture of aggregates due to gravity segregation which could possibly affects the experimental results.

1.3 Objective with Specific Aims

The objectives of this research are:

i. To determine the degree of capacity reduction of core-drilled columns.

ii. To evaluate the effect of concrete strength, core diameter and core location on the behavior of core-drilled columns.

iii. To observe the failure pattern under axial load.

iv. To conduct finite element simulation of the tested columns from current and previous studies.

v. To carry out a detail parametric study for investigating the effect of various relevant parameters on capacity of an existing column due to core extraction. The parameter that will be considered in the study are as follows:
   - Column size
   - Core diameter
   - Core location
   - Concrete strength
   - Confinement effect

vi. Finally, to develop a guideline in the form of graphs or charts or tables that will assist engineers to decide on core size and location that could be cut from an existing column.

1.4 Organizational of the Thesis

Chapter 1 refers to the general introduction of the study with scope and specific aims. Chapter 2 discusses some related study that has been done previously and also describes how concrete parameters have been introduced in FEA. In Chapter 3, the experimental setup has been described and the ultimate capacity and crack pattern of tested columns has been analyzed and discussed in details. In Chapter 4, validation of FEA simulation has been discussed by comparing ultimate capacity ratio (FEA./EXP.), load-displacement curve and crack pattern of experimental and FEA results. Chapter 5 describes different parameters such as tie bar spacing, column sizes, core size and concrete strength considered in this research for parametric study. This Chapter also provides description of generalized stress-strain graphs developed for different concrete strength. In Chapter 6, the obtained results found from FEA simulation have been analyzed and discussed. Moreover, in this Chapter, discussion on developed generalized graphical charts is provided. In Chapter 7, pertinent conclusions, inferred from this study, are presented and suggestions for possible future study are given. Finally, appendix is provided at the end of this thesis containing some relevant information for better understanding.

Chapter 2

LITERATURE REVIEW

2.1 General

Concrete is a heterogeneous material and consequently, exhibits a complex nonlinear mechanical behavior. Failure in tension and low confined compression is considered by softening which is defined as decreasing stress with increasing deformations. This softening response is accompanied by a reduction of the unloading stiffness of concrete, and irreversible (permanent) deformations, which are localized in narrow zones often called cracks or shear bands. On the other hand, the behavior of concrete subjected to high confined compression is characterized by a ductile hardening response; that is, increasing stress with increasing deformations. These phenomena should be considered in a constitutive model for analyzing the multiracial behavior of concrete structures. There are many constitutive models for the nonlinear response of concrete proposed in the literature. Commonly used frameworks are plasticity, damage mechanics and combinations of plasticity and damage mechanics. Stress-based plasticity models are useful for the modeling of concrete subjected to triaxial stress states, since the yield surface corresponds at a certain stage of hardening to the strength envelope of concrete (Pramono and Willam, 1989; Pivonka, 2001). Furthermore, the strain split into elastic and plastic parts, represents realistically the observed deformations in confined compression, so that unloading and path-dependency can be described well. However, plasticity models are not able to describe the reduction of the unloading stiffness that is observed in experiments. Conversely, damage mechanics models are based on the concept of a gradual reduction of the elastic stiffness (Ortiz, 1985; Resende, 1987; Carol et al., 2001). For strain-based isotropic damage mechanics models, the stress evaluation procedure is explicit, which allows for a direct determination of the stress state, without an iterative calculation procedure. However, isotropic damage mechanics models are often unable to describe irreversible deformations observed in experiments and are mainly limited to tensile and low confined compression stress states. On the other hand, combinations of isotropic damage and plasticity are widely used for modeling both tensile and compressive failure and many different models have been proposed in the literature (Jason et al., 2006, Farid and Janabi, 1990).

Moreover, the compressive and tensile behavior has been used in this study for different concrete strength which was proposed by Farid and Janabi, 1990.

2.2 Concrete Damaged Plasticity Model in ABAQUS

Constitutive parameters of a material describe its fundamental properties. When elements like concrete is to be considered, a wide range of materials with quantitatively and qualitatively different properties for different loading conditions (compression/tension) has to be taken into account which can be mixed in different proportions. Typically concrete will exhibit much greater compressive strength than tensile strength due to the strength of the bond between the aggregate and the cement paste (Mindess, Young, & Darwin, 2003). The relatively weak bond strength will result in cracking of the concrete which generates instability in general finite element plasticity models. So a large number of parameters need to be identified to successfully model a concrete member. Modeling of failure and fracture has become one of the fundamental issues in structural mechanics particularly in concrete structures and the principal task in failure description is the recognition of cracking patterns. For solving any boundary value problem with location of fracture, a complex constitutive modeling should be considered. Concrete Damaged Plasticity (CDP) is one of the possible constitutive models. The concrete damaged plasticity model in Abaqus/Standard and Abaqus/Explicit is based on the assumption of scalar (isotropic) damage and is designed for applications in which the concrete is subjected to arbitrary loading conditions, including cyclic loading. The model takes into consideration the degradation of the elastic stiffness induced by plastic straining both in tension and compression. It also accounts for stiffness recovery effects under cyclic loading.

2.2.1 Mechanical Behavior of Concrete

The model is a continuum, plasticity-based, damage model for concrete. It assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material. The evolution of the yield (or failure) surface is controlled by two hardening variables, $\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$, linked to failure mechanisms under tension and compression loading, respectively. We refer to

$\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$ as tensile and compressive equivalent plastic strains, respectively. The following sections discuss the main assumptions about the mechanical behavior of concrete.

2.2.2 Uniaxial tension and compression stress behavior

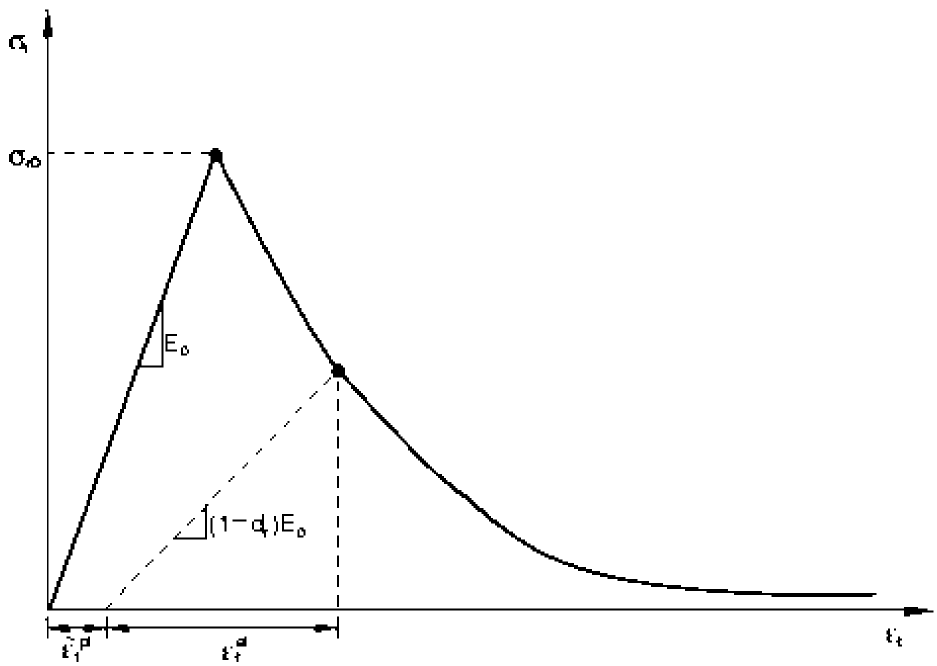


Figure 2-1 Response of concrete to uniaxial loading in tension.(Hibbit et al., 2009)

The model assumes that the uniaxial tensile and compressive response of concrete is characterized by damaged plasticity, as shown in Figure 2-1 and Figure 2-2

Under uniaxial tension, the stress-strain response follows a linear elastic relationship until the value of the failure stress, σ_{t0} is reached. The failure stress corresponds to the onset of micro-cracking in the concrete material. Beyond the failure stress the formation of micro-cracks is represented macroscopically with a softening stress-strain response, which induces strain localization in the concrete structure. Under uniaxial compression, the response is linear until the value of initial yield, σc0. In the plastic regime the response is typically characterized by stress hardening followed by strain softening beyond the ultimate stress, σ_{cu}. This representation, although somewhat simplified, captures the main features of the response of concrete.

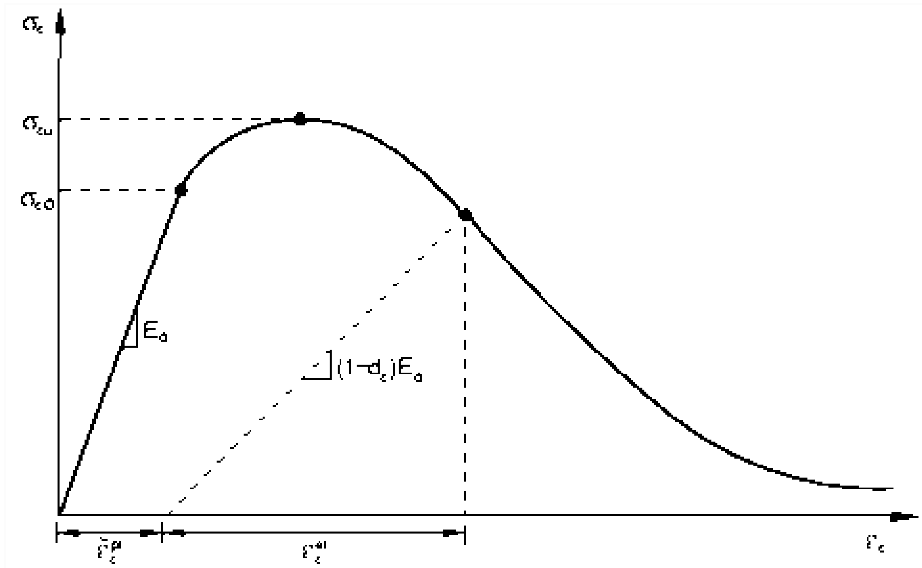


Figure 2-2 Response of concrete to uniaxial loading in compression. (Hibbit et al., 2009)

2.2.3 Stiffness Recovery

In a simple plasticity model, if force is removed after yielding has occurred, the residual plastic strain is found by a rebound function of the modulus of elasticity. Damage parameters of the CDP model modify this rebound function to include damage effects. Stiffness recovery is an important aspect of the mechanical response of concrete under cyclic loading. Abaqus allows direct user specification of the stiffness recovery factors ωt and ωc. The experimental observation in most quasi-brittle materials, including concrete, is that the compressive stiffness is recovered upon crack closure as the load changes from tension to compression. On the other hand, the tensile stiffness is not recovered as the load changes from compression to tension once crushing micro-cracks have developed. This behavior, which corresponds to ωt =0 and ωc =1 is the default used by Abaqus. Uniaxial load cycle assuming default behavior is illustrated in Figure 2-3.

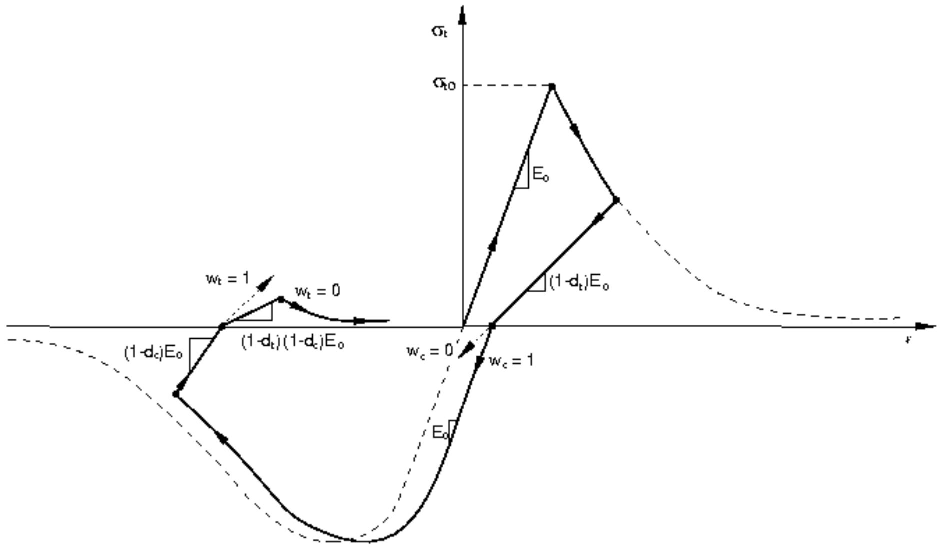


Figure 2-3 Uniaxial load cycle (tension-compression-tension) for the stiffness recovery factors ωt =0 and ωc =1. (Hibbit et al., 2009)

2.3 Element Type

ABAQUS contains a library of solid elements for two-dimensional and three-dimensional applications. The two-dimensional elements allow modeling of plane and axis symmetric problems and include extensions to generalized plane strain. The material description of three-dimensional solid elements may include several layers of different materials, in different orientations, for the analysis of laminated composite solids. A set of nonlinear elements for asymmetric loading of axis symmetric models is also available. Linear infinite elements in two and three dimensions can also be used to model unbounded domains. The solid element library includes iso-parametric elements; quadrilaterals in two dimensions and “CONCRETE” (hexahedra) in three dimensions. These iso-parametric elements are generally preferred for most cases because they are usually the more cost-effective elements that are provided in ABAQUS. They are offered with first- and second-order interpolation and are described in detail in “Solid iso-parametric quadrilaterals and hexahedra”. For practical reasons it is sometimes not possible to use iso-parametric elements throughout a model. For concrete model, 3D solid element C3D8R has been used in different concrete modeling by Tejaswini, and Rama Raju, 2015, Labibzadeh, et al., 2016 to analyze the effect of RCC beams in ABAQUS. These elements can be described as linear hexahedral element (C3D8R) that has been used in simulation. Figure 2-4 and Figure 2-5 represent necessary nodal arrangements of C3D8R element. In ABAQUS, the model can be extruded in any direction; this is why a 3D solid element in "modeling space" using deformable type for column was created. The solid element has eight nodes with three degrees of freedom at each node – translations in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing.

This element should not be used in the following situations:

- for high values of Poisson's coefficient
- for thin plates
- where element size is high

The C3D8R element is a general purpose linear brick element, with reduced integration (1 integration point). This element has only one integration point (instead of 2*2*2=8 integration points) which is located in the middle of element. The shape functions are the same as for the C3D8 element and can be found in partial differential equations from Lapidus, L. and Pinder, 1982.The shape functions of this element are given in the Equation 2-1 to 2-8.

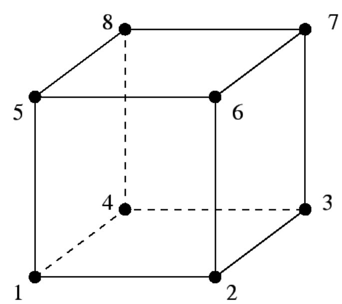


Figure 2-4 8-Node linear brick element (C3D8R).

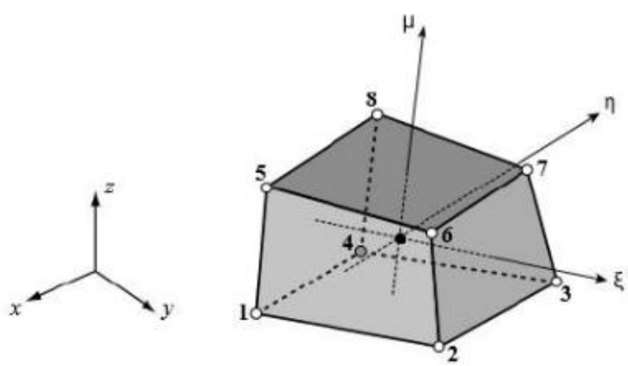


Figure 2-5 GLobal and local axis of C3D8R element

Equations:

$$N1 = \frac{1}{8}(1-\xi)(1-\eta)(1-\mu) \qquad 2\text{-}1$$

$$N2 = \frac{1}{8}(1+\xi)(1-\eta)(1-\mu) \qquad 2\text{-}2$$

$$N3 = \frac{1}{8}(1+\xi)(1+\eta)(1-\mu) \qquad 2\text{-}3$$

$$N4 = \frac{1}{8}(1-\xi)(1+\eta)(1-\mu) \qquad 2\text{-}4$$

$$N5 = \frac{1}{8}(1-\xi)(1-\eta)(1+\mu) \qquad 2\text{-}5$$

$$N6 = \frac{1}{8}(1+\xi)(1-\eta)(1+\mu) \qquad 2\text{-}6$$

$$N7 = \frac{1}{8}(1+\xi)(1+\eta)(1+\mu) \qquad 2\text{-}7$$

$$N8 = \frac{1}{8}(1-\xi)(1+\eta)(1+\mu) \qquad 2\text{-}8$$

Where Ni is the shape function at node i, and $\boldsymbol{\xi}$, $\boldsymbol{\mu}$, $\boldsymbol{\eta}$ are the local coordinates. For modeling of reinforcement, linear two node truss element (T3D2) has been used by Tejaswini, and Rama Raju, 2015, Labibzadeh, et al., 2016. In FEA simulation, T3D2 element has been suggested to use where no bending effect is expected to occur (Reddy, 1993). It has two nodes and three degrees of freedom at each node in the global coordinate. Local coordinate has an axial degree of freedom at each node. Hence, Truss elements are rods that can carry only tensile or compressive loads. The shape functions of this element in local coordinate are presented in the Equation 2-9 and Equation 2-10. The node numbering follows the convention of Figure 2-6.

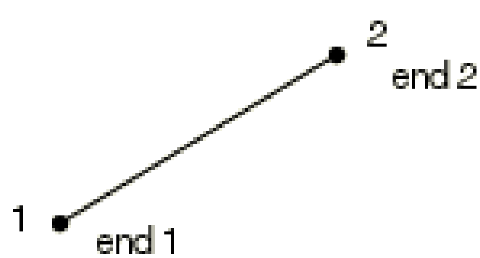


Figure 2-6 2-Node linear 3D truss element (T3D2).

Equations:

$$N1 = \frac{1}{2}(1 - \xi) \qquad 2\text{-}9$$

$$N1 = \frac{1}{2}(1 + \xi) \qquad 2\text{-}10$$

2.5 Previous Study

As safety is the primary concern for every structure, detailed assessment of the structures has become indispensable so that resources are not wasted due to unnecessary rehabilitation (Buckland and Barlett, 1992). As a part of such ongoing assessment process, evaluation of concrete strength through core cutting has been recommended for a large number of buildings. Though, core cutting is one of the most reliable methods to determine strength of existing concrete work (Malhotra, 1976), it has the potential to reduce the capacity of structural element. The relevant ASTM Standard (ASTM C42, 2004) also delegates the safety issues to the prudent judgment of concerned engineers. However, the effect of core drilling on the RC column capacity has not been much addressed in the literature. A thorough literature review reveals two studies (Calavera et al., 1979 and Masi et al., 2012) where some indications can be found on the effect of core cutting on RC column capacity. It has been suggested by Masi et al. (Masi et al., 2012) that restoration can be ineffective in case of low strength concrete. An analytical study by Siddique and Khomeni (Siddique and Khomeni, 2014) also found that effect of core drilling is more pronounced in low strength concrete. Unfortunately, most of the structures that require safety assessment usually have concrete of low strength. Moreover, in many practical instances in the country, cores were required to be cut from a large number of columns of a single building for conducting detailed engineering assessment (DEA). It is evident that effect of core cutting is different for different types of structure and it depends on concrete quality, aggregate type, member size, reinforcement detailing etc. With this end in view, a comprehensive study has been undertaken to investigate the effect of core extraction on capacity of a column and eventually, to develop a guideline that could be followed during core cutting. In this study, brick chips was used as coarse aggregate as many buildings (particularly the old ones) in this country use brick chips as it is cheaper, locally available and light-weight.

Chapter 3

EXPERIMENTAL SETUP AND DATA ANALYSIS

3.1 General

This Chapter presents a brief description of the experimental setup and procedural steps to perform test on a real concrete column with a view to comparing the ultimate capacity after cutting core at different level and also restoration with different filler material tool.

3.2 Material

3.2.1 Stone Chips

In the previous study (Manzur and Ahmed, 2018) brick chips were used as coarse aggregate, but in this study stone chips were used. The Nominal maximum size of stone chips was 18.75mm (¾ inch) downgrade. Stone chips were left for 24 hours to attain SSD condition before concrete mixing. The Fineness Modulus (FM) was obtained as 7.60 from the lab test. The gradation curve that has been obtained from lab test data is shown in Figure 3-1.

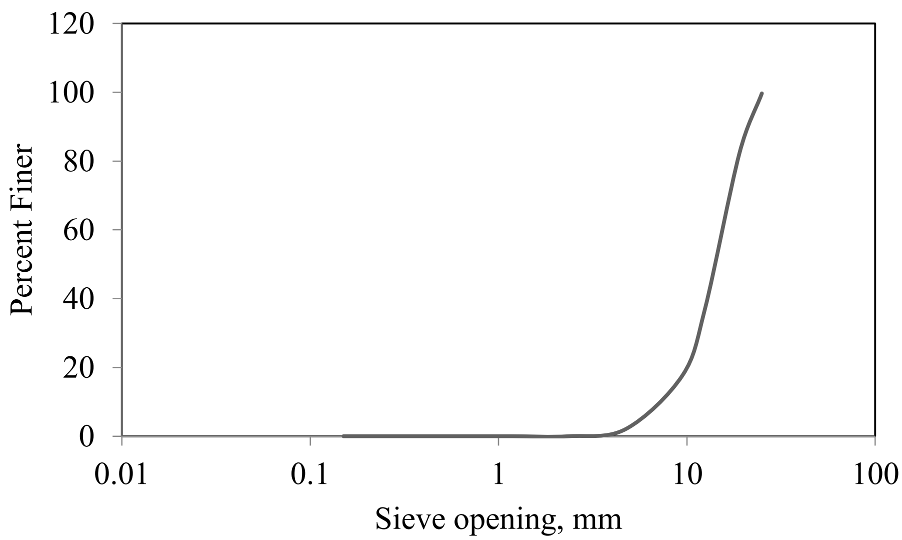


Figure 3-1 Gradation curve of stone.

3.2.2 Sylhet Sand

Sylhet sand were used as fine aggregate. Sand was left for 24 hours to attain SSD condition before concrete mixing. The Fineness Modulus (FM) was found as 2.44 from the lab test. The gradation curve that has been found from lab test data is shown in Figure 3-2.

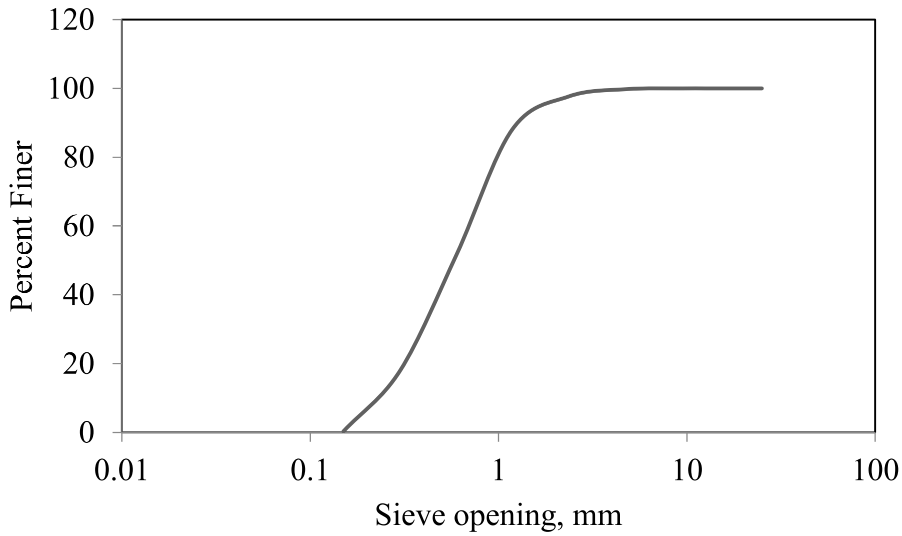


Figure 3-2 Gradation curve of sand.

3.2.3 Cement

The most available ordinary Portland cement (OPC) known as fresh cement is used in this study.

3.3 Instruments

The following instruments were used to conduct various experiments and relevant tests:

- Mechanical Compactor/Vibrator
- Mixture Machine
- Universal Testing Machine (Tinius Olsen).
- Core-cutter
- Ferro scanner

3.4 Concrete Mix Design

The process of selecting suitable ingredients of concrete and determining their relative amounts with the objective of producing a concrete of the required, strength, durability, and workability as economically as possible, is termed the concrete mix design. The proportioning of ingredient of concrete is governed by the required performance of concrete in two states, namely the plastic and the hardened states. If the plastic concrete is not workable, it cannot be properly placed and compacted. The property of workability, therefore, becomes of vital importance. The compressive strength of hardened concrete which is generally considered to be an index of its other properties, depends upon many factors, e.g. quality and quantity of cement, water and aggregates; batching and mixing; placing, compaction and curing. The concrete mix design was conducted to gain the target strength of 20.7 MPa. The ratios were found after several trial and error was 1:1.5:3 (C: FA: CA) by volume with varying water cement ratio of 0.4.

3.5 Sequence of Lab Activities

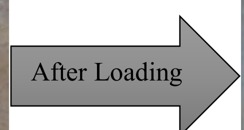
3.5.1 Formation of Formwork

A total of 16 (Sixteen) columns were casted using predetermined concrete mixes with cross sectional dimensions of 200mm X 200mm and height of 1250mm. Wooden formwork was prepared for casting the column. To prepare formwork mango wood was used. The prepared formwork is shown in the Figure 3-3 (a) & (b). In the previous research it was observed that columns broke due to excessive stress concentration at the corner of the head of the column (Siddique and Khomeni, 2014). To eliminate this problem and to get the crack at the desired location (along weak plane) of the column except column head it was made tapered shape. In this regards, column head was made with the size of 300mm x 300mm x 150mm. Figure 3-4 and Figure 3-5 shows the previous study of failure of column at corner of column head due to excessive stress concentration. On the other hand, Figure 3-6 and 3-7 shows the failure of column along the weak plane of the column after pure axial loading.

(a)

(b)

Figure 3-3 Wooden formwork of size 200mm x 200mm x 1250mm



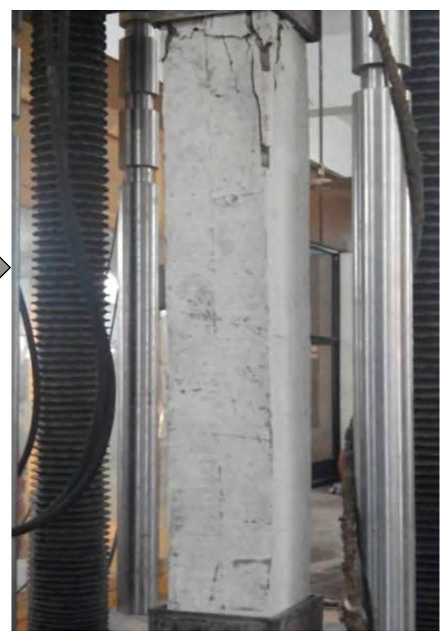
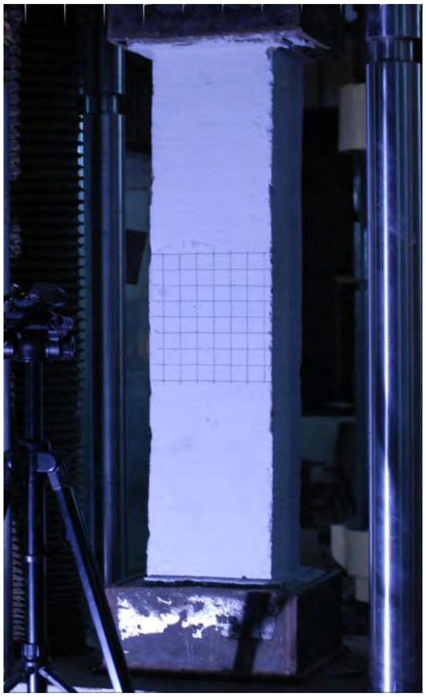
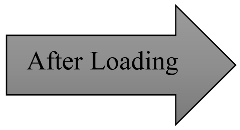


Figure 3-4 Column without head before lab test.

Figure 3-5 Corner of column head broken due to stress concentration after axial loading.





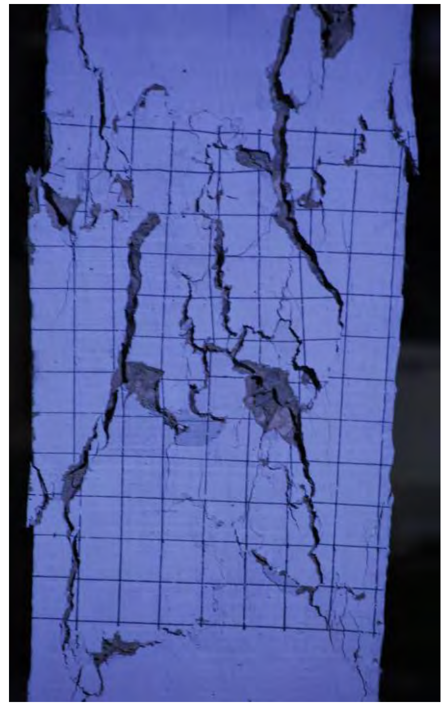


Figure 3-6 Column without head before lab test.

Figure 3-7 Corner of column head broken due to stress concentration after axial loading.

3.5.2 Mixing of concrete

The aggregates are mixed with predetermined mixing ratio 1:1.5:3 (C:FA:CA). Before mixing, stone chips and sand are left for 24 hours to attain SSD condition. The concrete was mixed in mixture machine with predetermined water cement ratio of 0.4 by volume. Water was added in the mixture machine in a controlled way to ensure uniform mixing of aggregate during mixing. Three cylinders were prepared from each mixture to determine concrete yield strength. The cylinders were also cured in the same condition as the column cured to ensure the similar strength of concrete. In this regards, column and cylinder were wrapped by sackcloth and kept wet for 28 days.

3.5.3 Core Cutting

The assessment of RC existing buildings is very important in earthquake engineering, particularly for age old structures and also having poor seismic design. In the process of assessing of RCC existing buildings, investigation procedures have a crucial role to get an adequate knowledge of the structure to be evaluated. Among other factors, materials' properties and, particularly, concrete strength need to be estimated. According to several codes (e.g. NTC, 2008; CEN EC8-3, 2005; ACI 228, 1998;) estimation of in-situ strength has to be based on both Non Destructive Tests (NDTs) (Malhotra, 1976) and Destructive Tests (DTs), core testing is considered to be the most reliable procedure to estimate in-situ concrete strength of an existing RCC elements like beam column, slab, shear-wall and so on.

As per code ASTM C 42, core specimen has to be drilled perpendicular to the surface and not near formed joints of obvious edges of a unit of deposit and cylindrical cores should be drilled out with length-diameter ratio (L/D) greater than or equal to 1 (ASTM C42, 2004). Preferred minimum core diameter is three times the nominal maximum size of the coarse aggregate, but for concrete with nominal maximum aggregate size greater than or equal to 37.5mm (1 $\frac{1}{2}$ in.), it should be at least two times the nominal maximum size of the coarse aggregate (ASTM C42, 2004). There is no universal relationship between the compressive strength of a core and the corresponding compressive strength of a standard-cured molded cylinder, but historically, it has been assumed that core strengths are generally 85% of the corresponding standard-cured cylinder strength (ASTM C42, 2004). ASTM C42 covers obtaining, preparing and testing cores drilled from concrete. But this standard does not purport to address the safety concerns and leaves the responsibility of establishing appropriate safety and health practices and determination of the applicability of regulatory limitations to the user.

The following instruments were used during core test;

- Core-cutter
- Digital Ultrasonic measuring tools

Core cutter is a machine equipped with mechanically powered machine which was placed vertically during core cutting. Before core cutting, Digital Ultrasonic measuring tools was used to determine the arrangement and orientation of reinforcement is shown in Figure 3-8. The size of the cutter was 50mm by 100mm. During core cutting water supply was also necessary to lubricate the cutter. The column was placed horizontally on a rigid surface to reduce movement during drilling by core cutter. Rotary Hammer Drills with selectable pneumatic hammering mechanism which was electrically-powered tools for drilling in concrete surface from which the core was to be extracted is shown in Figure 3-9.

Figure 3-9 Determination of rebar arrangement by scanner.

Figure 3-8 Drilling of core from column.

3.5.4 Core Filling

After core drilling it is required to fill the hole by a suitable filler material to restore the capacity of the member. The commonly used filler materials are found to be hand mix regular concrete mixture. However it is recommended to use Non-Shrinkage high flow able cementitious material grout. Therefore, in this study both types of filler material were used;

- Concrete lean mix (CLN)
- Non Shrinkage mortar grout (NS)

The concrete lean mix is a mixture of cement and water. The mixing ratio for concrete lean mix is 0.45 by weight (w/c). On the other hand, the mixing ratio for Non shrinkage mortar grout is 0.4 by weight (water/NS). Before placing filler material into the core, the core was cleaned to keep it dust free. Then the core is filled by both of the filler material. To determine the strength of filler material, three cubes were made of 50mm x 50mm in size. The mixing ratio for concrete lean mix was kept 0.45 (w/c) and 0.4 (w/ns) by weight for non-Shrinkage grout. The amount of water for non-shrinkage grout was determined by adding water gradually to the grout until flow was occurred. After a uniform mixing three cubes were made of 50mm x 50mm in size. After curing the cube of 14 days, ultimate strength was measured by UTM. From the ultimate strength Figure

3-10, it is obvious that non-shrinkage grout has higher strength compared to concrete lean mix. As non-shrinkage grout is a higher strength material than cement and taking higher amount of load compared to concrete lean mix.

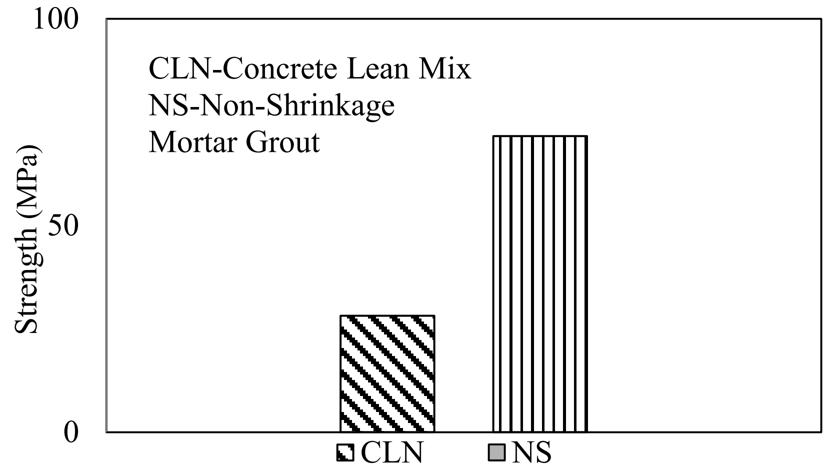


Figure 3-10 Concrete strength bar chart for CLN and NS.

3.5.5 Determination of Concrete Strength

The average concrete strength was found as 30.10 MPa as shown in Table 3-1.

Table 3-1 Cylinder test result

Cylinder Test			
KN	Cylinder Area (mm^2)	MPa	fc$^{'}$ (MPa)
220	7850	28.3	
241	7900	30.6	30.1
245	7800	31.4	

3.5.6 Column Axial Load Test

After completion of all laboratory work the column is ready for pure axial testing. All the columns were then tested in the lab under pure axial loading by the Universal Testing Machine (Tinius Olsen) which is available in the Strength of Materials Laboratory of the Department. As the maximum allowable capacity of the machine is 2000 KN, so the column dimension and concrete strength was designed in such a way to keep the column ultimate strength under control. Before loading by the UTM, each column was kept purely vertical to ensure pure axial loading as well as to eliminate any bending effect. To ensure the verticality of column a vertical measuring tool was used and also the center of loading plate was marked for centering of column.

All the columns were squarely marked at the middle zone where failure is expected to occur is shown in Figure 3-11. All types of columns like normal column (NC), core mid height (CMD), core one third height (COD), restored by Concrete Lean Mix (CLN) and restored by Non-Shrinkage Grout (NS) were tested by pure axial loading. For the column of CLN & NS, filler zone is circularly marked by the marker to differentiate it from the other column is shown in Figure 3-15. The overall test setup for different types of columns are shown from Figure 3-12 through Figure 3-15.

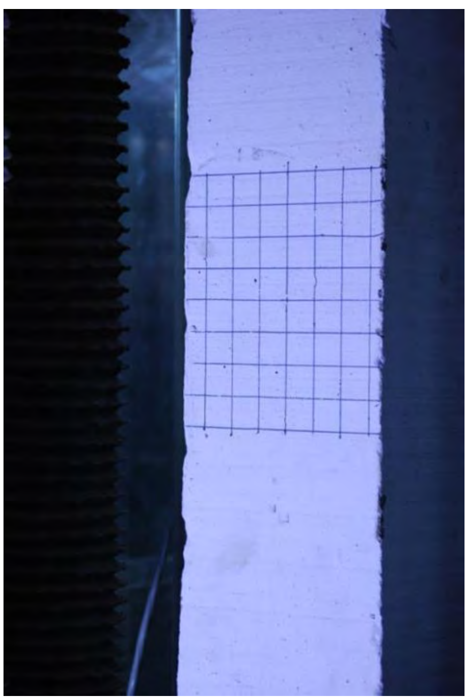


Figure 3-11 Columns were marked squarely at the middle region.

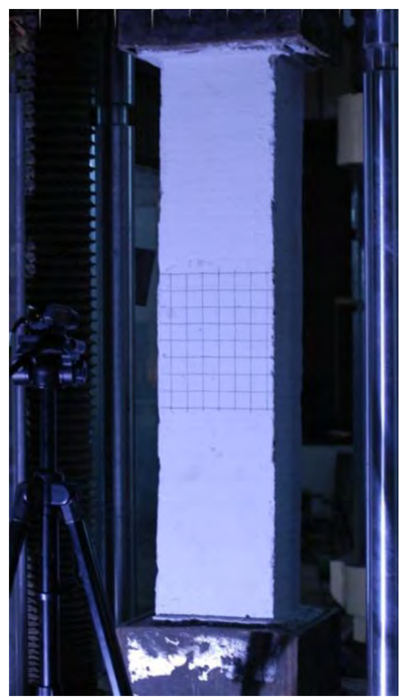


Figure 3-12 Test setup for NC specimen.

Figure 3-14 Test setup for COD specimen.

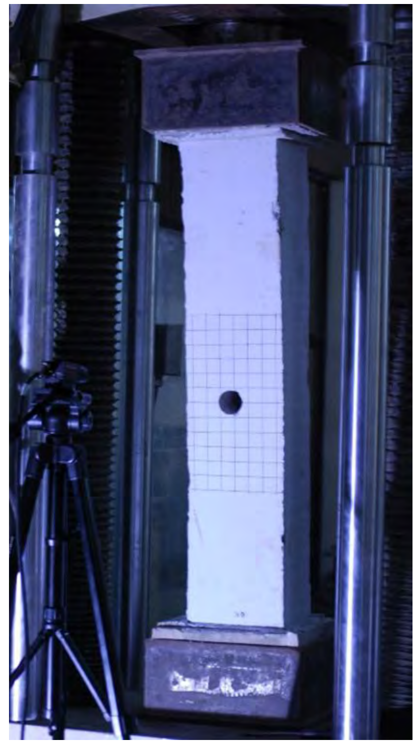


Figure 3-13 Test setup for CMD specimen.

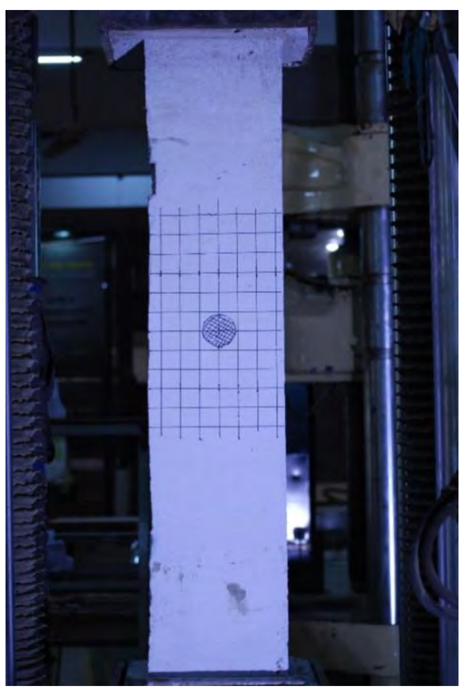


Figure 3-15 Test setup for NS & CLN specimen.

3.6 Data Analysis

The experimental campaign is reported and analyzed, with particular emphasis to the main goal of the study that is pointing out the effects of core drilling and subsequent restoration of axial capacity of columns. Table 3-2 summarizes the test results and Figure 3-16 shows column capacity for different types of column. From Figure 3-16, it is evident that ultimate capacity of normal columns (NC) is always higher than that of columns with core and also lower than the column having core restored by non-shrinkage grout (NS). But in case of column having core restored by concrete lean mix (CLN) shows lower capacity than normal column (NC) which is not very significant. On the other hand, the ultimate capacity lines for columns having core at mid height (CMD) and for columns having core at one third height (COD) are always reasonably lower than the capacity of NC columns. Figure 3-17 shows the percent reduction in capacity of CMD and COD columns with respect to NC columns. It is apparent from 3-17 that capacity of core drilled column varies significantly depending on location of core. It has been observed that COD columns experienced greater reduction in capacity as compared to capacity of CMD columns. Moreover, reduction in capacity of core drilled columns has been found to be variable with concrete strength. However, core location has more pronounced effect on capacity of core drilled columns as compared to concrete strength. The percentage of reductions in capacity of CMD columns have been found as 8.30 % for 30.10 MPa concrete with respect to capacity of NC columns. On the other hand, COD columns showed 19.20 % reduction in axial capacity as compared to NC columns for 30.10 MPa concrete, respectively. The hypothesis behind such higher reduction in axial capacity of COD columns might be due to relatively larger stress concentration near support which is transferred to the core found near the support. As a result, the core of COD column is located near support resulting in low column capacity in comparison with CMD columns. In case of column having core restored by non-shrinkage grout (NS) in Figure 3-16 shows higher capacity than normal column (NC). As the non-shrinkage grout is a higher strength material which is used as a filler material of core resulting in higher capacity of NS column. On the other hand, column having core restored by concrete lean mix (CLN) shows lower capacity in comparison with Normal column (NC). The percentage of capacity reduction for CLN column is not much pronounced. The percentage of capacity increases for NS column is 2.60 %. Again the percentage of capacity reduction for CLN column is 2.0 %.

Table 3-2 Experimental data of column axial load test.

<table>
<tr><td>Compressive strength, fc' (MPa)</td><td colspan="4">30.1</td></tr>
<tr><td rowspan="2">Normal Column</td><td>Axial Capacity (KN)</td><td>1200.0</td><td>1190.0</td><td>1160.0</td></tr>
<tr><td>Avg(KN)</td><td colspan="3">1183.3</td></tr>
<tr><td rowspan="3">Core Mid Height (CMD)</td><td>Axial Capacity (KN)</td><td>1036.0</td><td>1098.0</td><td>1122.0</td></tr>
<tr><td>Avg(KN)</td><td colspan="3">1085.3</td></tr>
<tr><td>% of Capacity Reduction</td><td colspan="3">8.3</td></tr>
<tr><td rowspan="3">Core One third Height (COD)</td><td>Axial Capacity (KN)</td><td>952.0</td><td>952.0</td><td>963.0</td></tr>
<tr><td>Avg(KN)</td><td colspan="3">955.7</td></tr>
<tr><td>% of Capacity Reduction</td><td colspan="3">19.2</td></tr>
<tr><td rowspan="3">Restoration by Mortar Grout (CLN) core at mid height</td><td>Axial Capacity (KN)</td><td>1164.0</td><td>1159.0</td><td>1155.0</td></tr>
<tr><td>Avg(KN)</td><td colspan="3">1159.3</td></tr>
<tr><td>% of Capacity</td><td colspan="3">-2.0</td></tr>
<tr><td rowspan="3">Restoration by Non Shrinkage Mortar Grout (NS) core at mid height</td><td>Axial Capacity (KN)</td><td>1215.0</td><td>1220.0</td><td>1208.0</td></tr>
<tr><td>Avg(KN)</td><td colspan="3">1214.3</td></tr>
<tr><td>% of Capacity</td><td colspan="3">+2.6</td></tr>
</table>

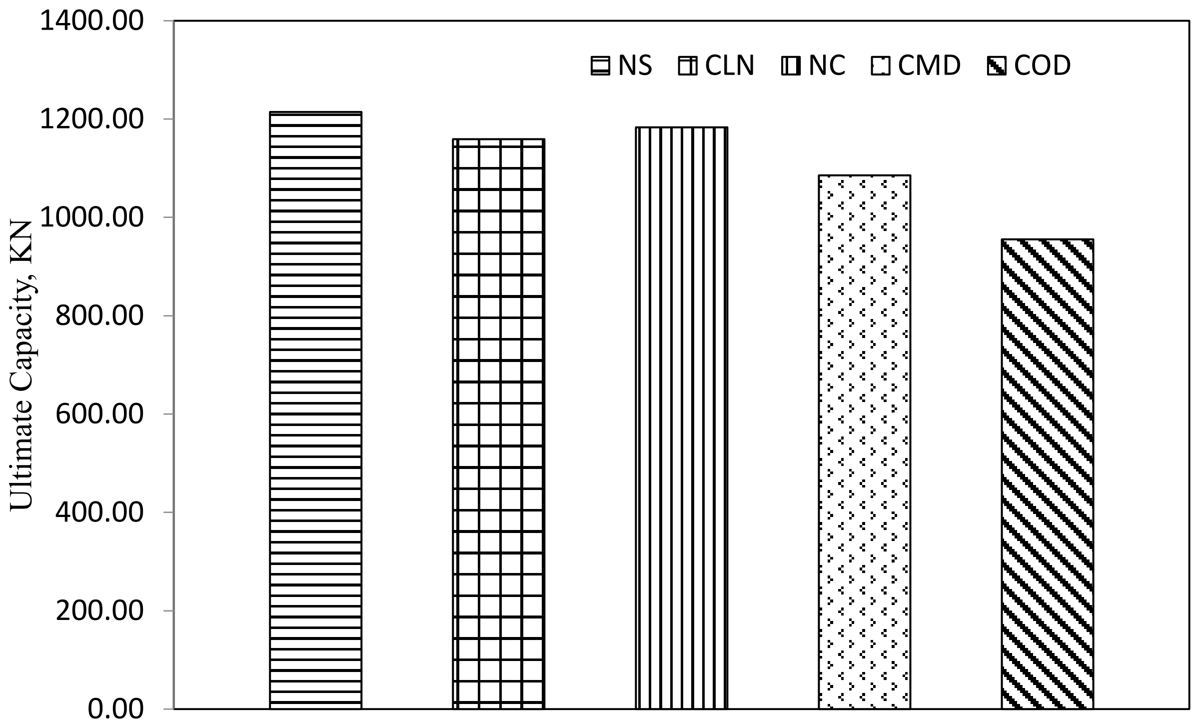


Figure 3-16 Column capacity chart from lab test data.

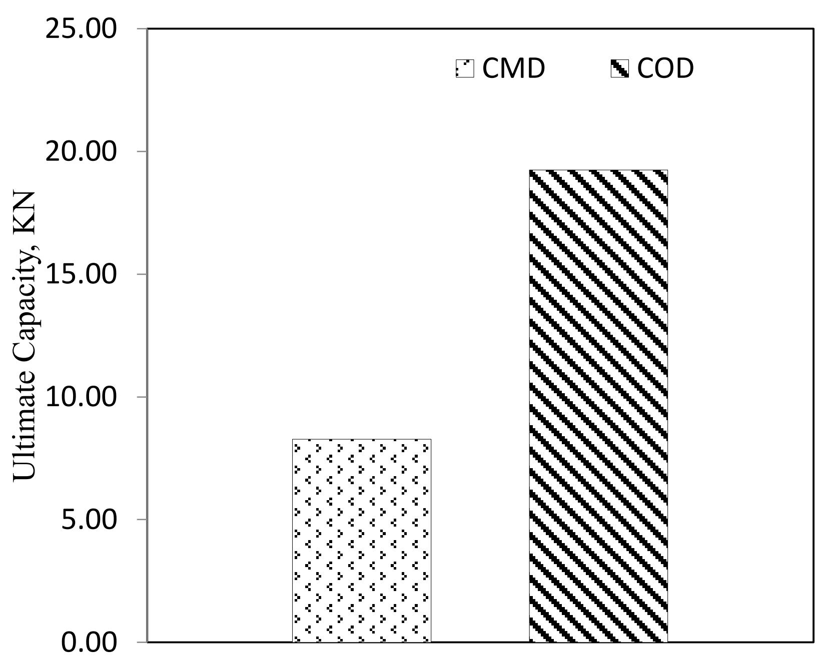


Figure 3-17 Percentage of column capacity reduction for CMD & COD obtained from lab test.

3.7 Crack Propagation

As in this study, columns crack pattern were observed for only pure axial loading. To ensure verticality of column, a vertical measuring tool was used and also the center of loading plate was marked for centering of columns so that no bending effect was incorporated during test. The pure axial load is imposed by Tinius Olsen machine known as Universal Testing Machine (UTM). All the columns were tested under pure axial loading with a constant vertical displacement rate of 3mm/min. The ultimate capacity and crack pattern of each column was observed for the pure axial loading and constant vertical displacement rate. In case of normal column (NC), the crack starts initially at mid region and then propagates diagonally to the outer face of column edge. From the visual observation, the diagonal failure of the crack making an angle of approximate 45 degree (±) which is shown in Figure 3-18. For COD and CMD columns, the initial cracks developed at the vicinity of core location and eventually propagated diagonally towards boundary region of the column. For column having core, crack always initiates from core as shown in Figures 3-19 and 3-20. On the other hand, for CLN column, the crack starts from the core and extends diagonally to the boundary of the column which shows similar type of crack pattern of NC column as shown in Figure 3-21. But in case of NS column, as shown in Figure 3-22, crack starts from the outer region of core extends diagonally or vertically to the column edge. Here crack does not generate from core region due to restoration by high strength non shrinkage grout rather than generates from outer region of core.

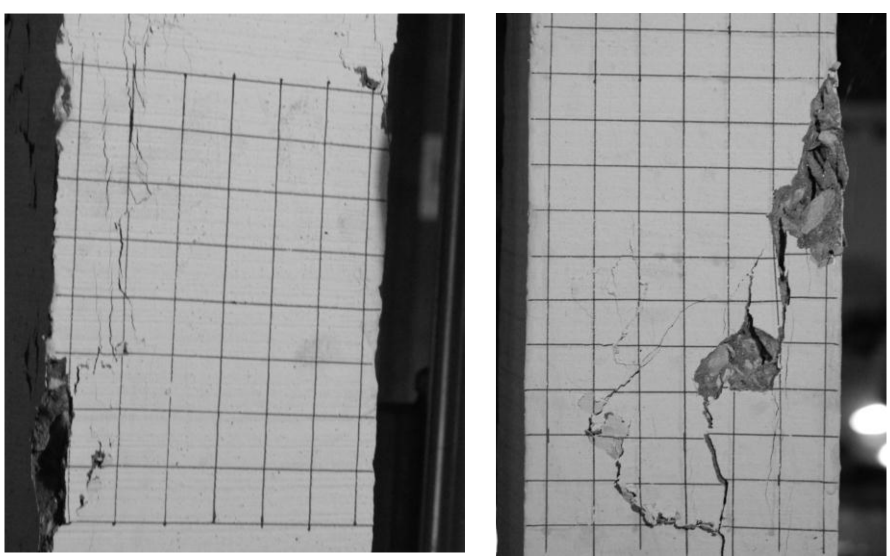


Figure 3-18 Typical crack pattern of NC column.

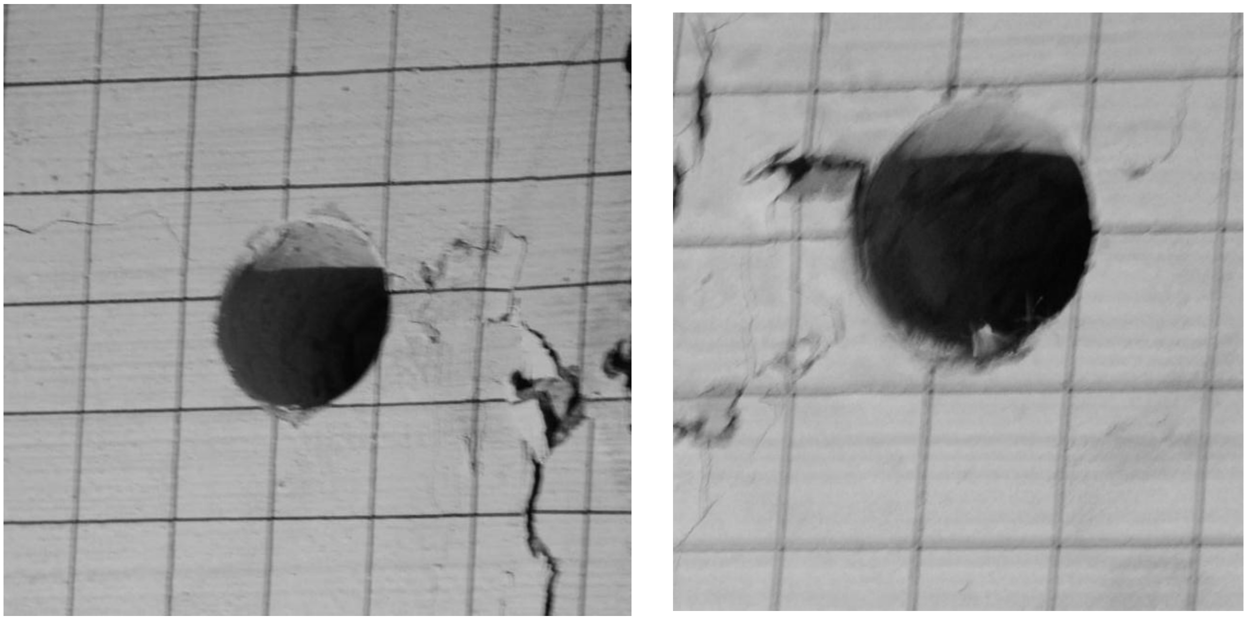


Figure 3-19 Typical crack pattern of COD column.

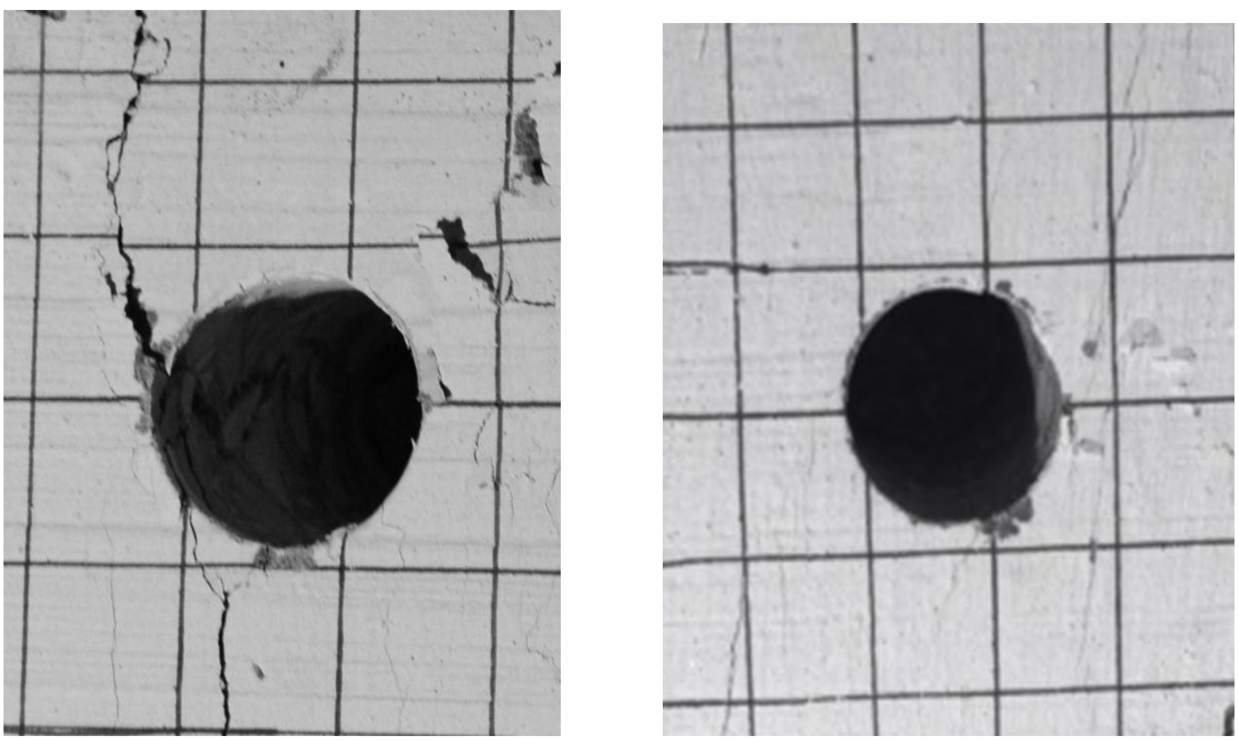


Figure 3-20 Typical crack pattern of CMD column.

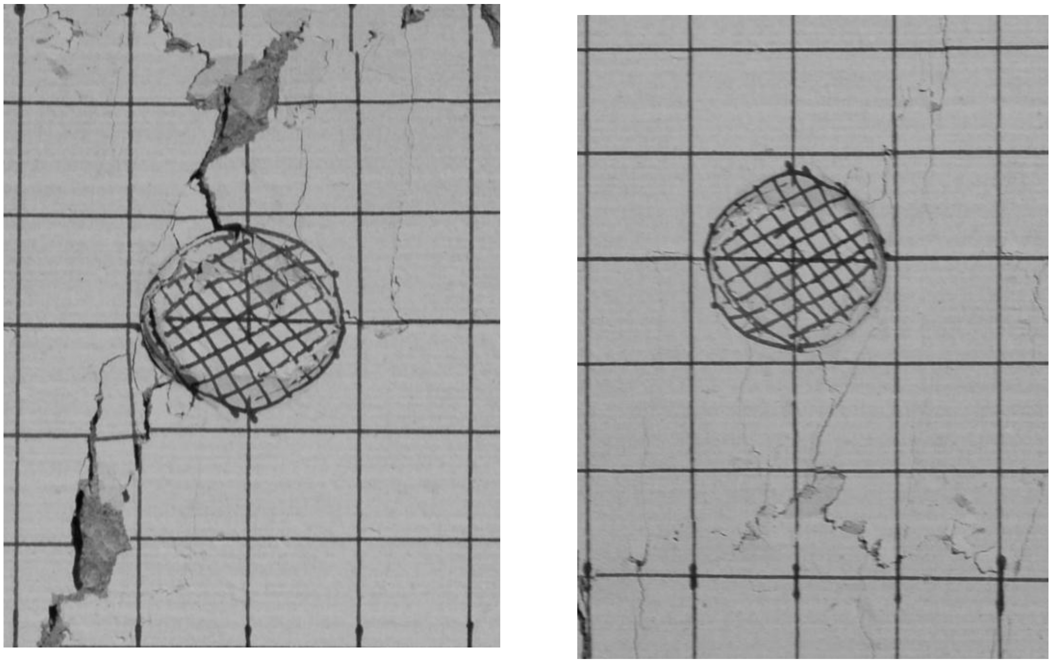


Figure 3-21 Typical crack pattern of CLN column.

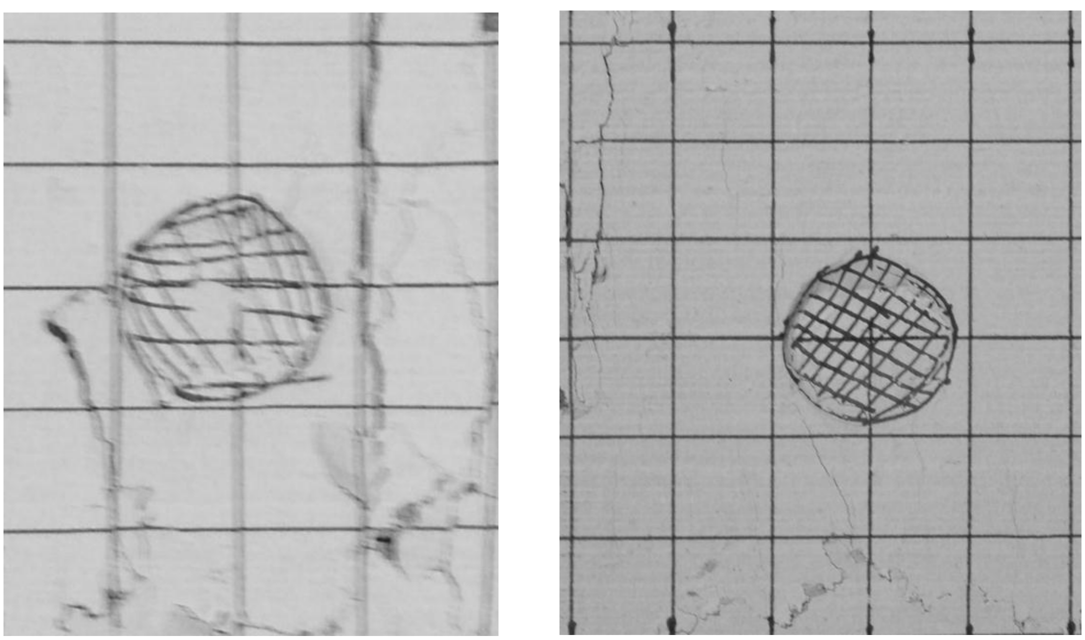


Figure 3-22 Typical crack pattern of NS column.

Chapter 4

FEA SIMULATION AND VALIDATION

4.1 General

As part of the study, finite element analysis of columns with cores has been conducted in order to evaluate effect of various parameters on axial capacity of columns. However, it is extremely important to validate the FEA method with experimental results. Therefore, before conducting FEA of full scale columns, experimental columns were simulated in ABAQUS environment. In this chapter, the FEA of the lab-scale test columns is described and compared with the experimental results in order to validate the analysis procedure.

4.2 Validation of FEA Simulation

Finite element models of experimental columns were developed and analyzed in ABAQUS environment in order to validate FEA results with experimental data. This validation is extremely important to carry out FEA of full scale columns. Concrete damage plasticity (CDP) model was used to observe the failure pattern as well as load carrying capacity of the columns. The parameters of CDP were taken from ABAQUS default values (Reddy, 1993) as shown in Table 4-1

A stress strain relationship for concrete is also needed as input for analysis in ABAQUS. The stress-strain relationship based on concrete strength proposed by (Farid and Janabi, 1990) has been used in this study for FEA of the columns was developed using concrete strength of 30.10 MPa. The compressive and tensile stress-strain curve for concrete having compressive strength of 30.10 MPa is presented in Tables 4-2 and 4-3 respectively. Figures 4-1 and 4-2 show the graphical representation of Tables 4-2 and 4-3.

Table 4-1 Default CDP Parameters from ABAQUS

E, GPa	Poisson ratio, μ	Dilation angle, β	Eccentricity, m	f=fb0/fb	k	Viscosity, γ
25.9	0	35	0.1	1.16	0.666	0

Table 4-2 Compression behavior

Stress, MPa	Strain	Damage	Strain
1.505	0	0	0
13.630057	0.000512377	0	0.000512
23.6335513	0.001024754	0	0.001025
28.7503749	0.001537131	0	0.001537
30.1	0.002049508	0	0.00205
29.3224033	0.002561885	0	0.002562
27.594302	0.003074262	0.195402	0.003074
25.5645647	0.003586639	0.596382	0.003587
14.8309767	0.007173279	0.710133	0.007173
7.33563858	0.014346558	0.894865	0.014347

Table 4-3 Tensile Behavior

Stress, MPa	Strain	Damage	Strain
0.75	0	0	0
1.07	3.33E-05	0	3.33E-05
0.70	0.0001604	0.406411	0.00016
0.32	0.0002798	0.69638	0.00028
0.08	0.0006846	0.920389	0.000685
0.02	0.0010867	0.980093	0.001087

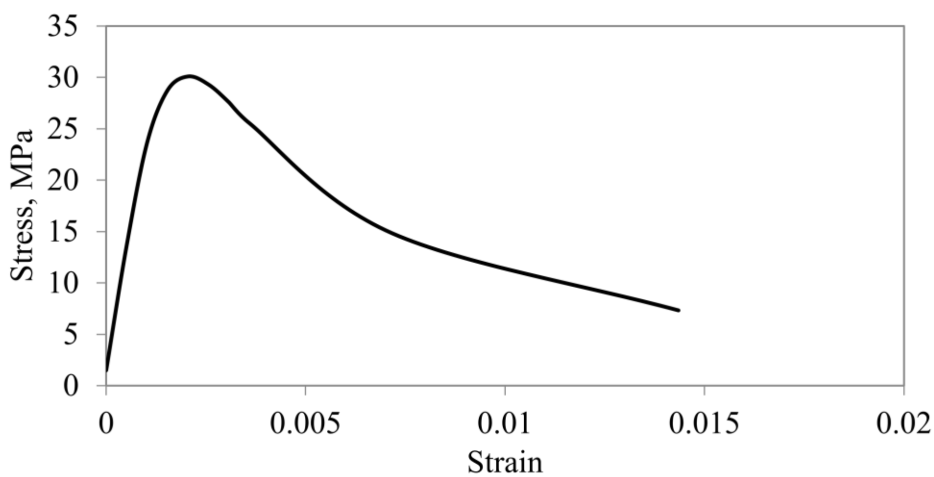


Figure 4-1 Generalized compression behavior for 30.10 MPa concrete. (Farid and Janabi, 1990)

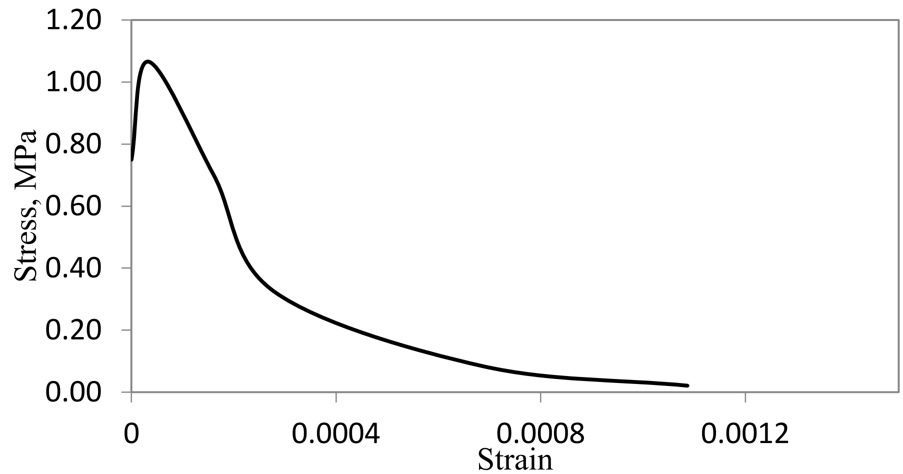


Figure 4-2 Tensile behavior for 30.10 MPa concrete. (Farid and Janabi, 1990)

Mesh configurations were one of the most important parameters in FEA simulation. Some different types of mesh sizes were suggested by Hibbit et al. (2009).To determine optimum mesh size, several FEA analysis were done with different varying mesh sizes. A graph is plotted between column capacities Vs mesh size to determine optimum mesh size as shown in Figure 4-4

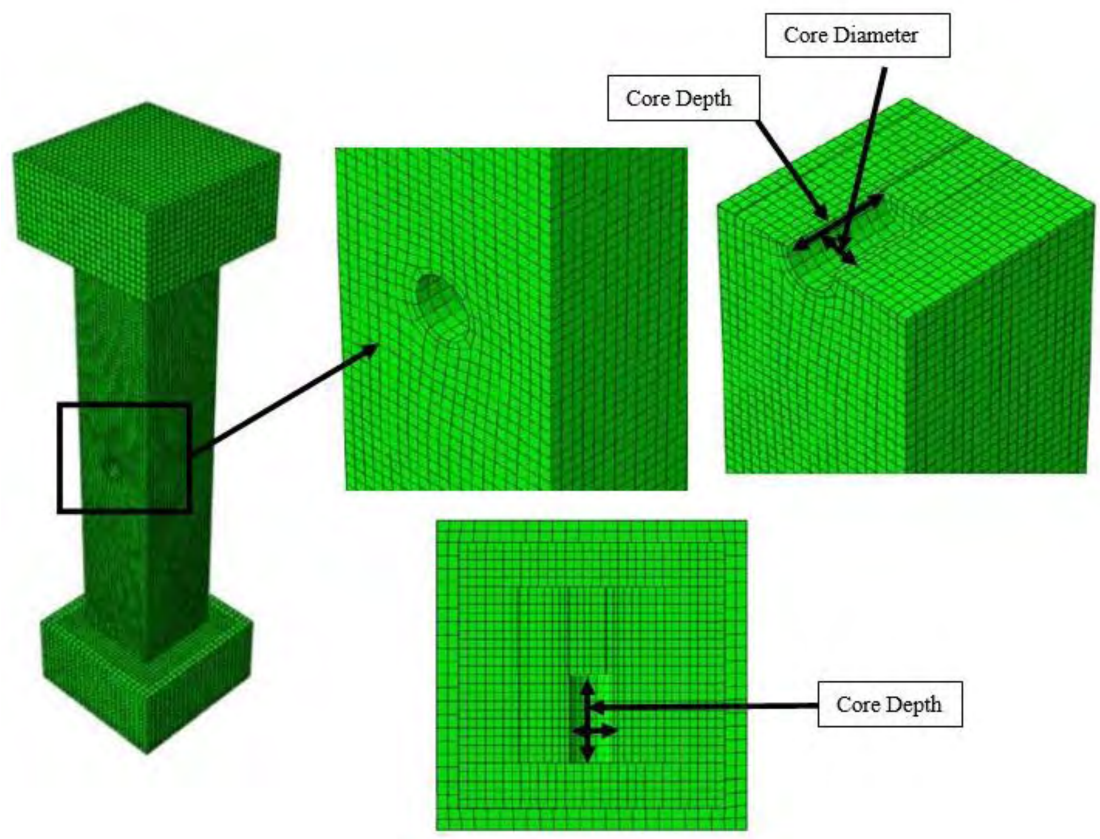


Figure 4-3 3D view of the column after meshing using sweep mesh.

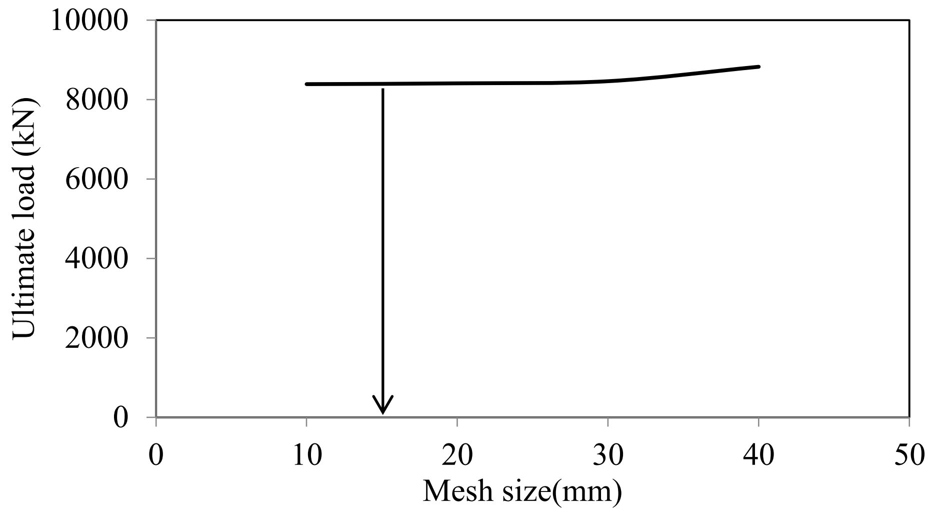


Figure 4-4 Column capacity Vs mesh size.

In the FEA model, the boundary conditions were defined at the bottom support similar to the laboratory test by restraining translation in the vertical direction. The vertical displacement rate was kept same as laboratory test which was equal to 3 mm/min (Siddique, 2014 and Manzur, 2016). The load displacement curve found from machine data was almost similar to the load-displacement curve found from FEA simulation as shown in Figure 4-6. In experimental analysis the column ultimate load was found at 3.6 mm displacement but in case of FEA simulation it was found only 3.1 mm displacement. It is obvious that FEA is more conservative in determination of column ultimate capacity. Again it has been observed that crack starts initially at the mid region of NC column. For CMD and COD columns, the initial cracks developed at the vicinity of core location and eventually propagated diagonally towards boundary region. Crack patterns observed from FEA analysis have been found to be in consensus with the experimental findings. Figure 4-7 shows the overall crack pattern found in laboratory tests and FEA analysis. The axial capacity of different types of columns found from FEA analysis also showed close proximity to the experimental results. The ratio of ultimate strength between FEA and Experiments has been found to be almost 1.0 as shown in Figure 4-8 which also indicates that the experimental capacity and FEA results are quite similar in all instances. The difference between experimental and FEA capacities of NC columns for 30.1 MPa strength was found to be 1.82 %. For CMD and COD columns having strength of 30.1 MPa, the differences between experimental and FEA capacities were found to be 1.16% and 0.80%, respectively. In most cases, experimental values were found to be slightly higher than the FEA results, showing that the FEA has been slightly conservative in obtaining the column capacity. However, in all instances, the difference between experimental results and FEA analysis is insignificant. Therefore, it can be said that simulation of columns in ABAQUS environment has been validated by experimental results. Details of experimental and FEA results are provided in Table 4-4.

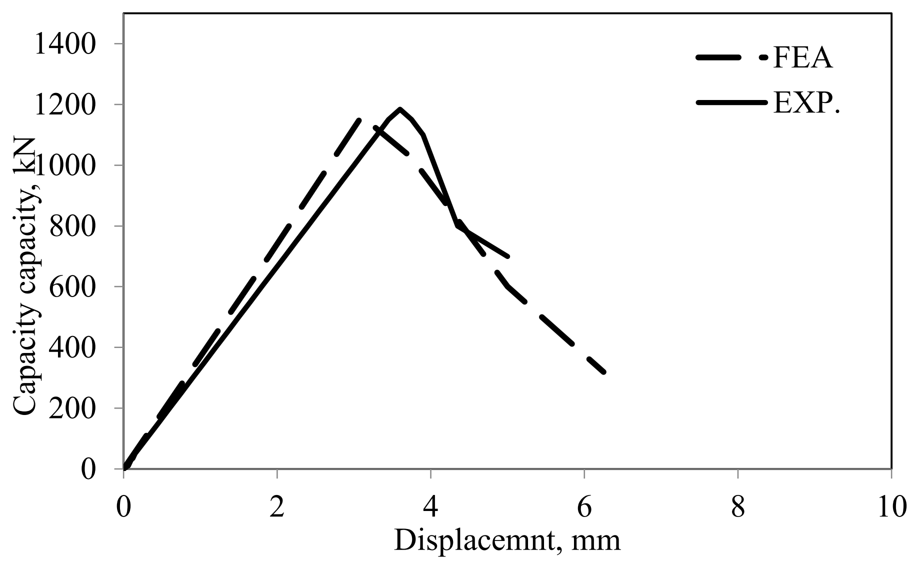


Figure 4-5 Load Vs displacement of EXP. and FEA simulation for NC column.

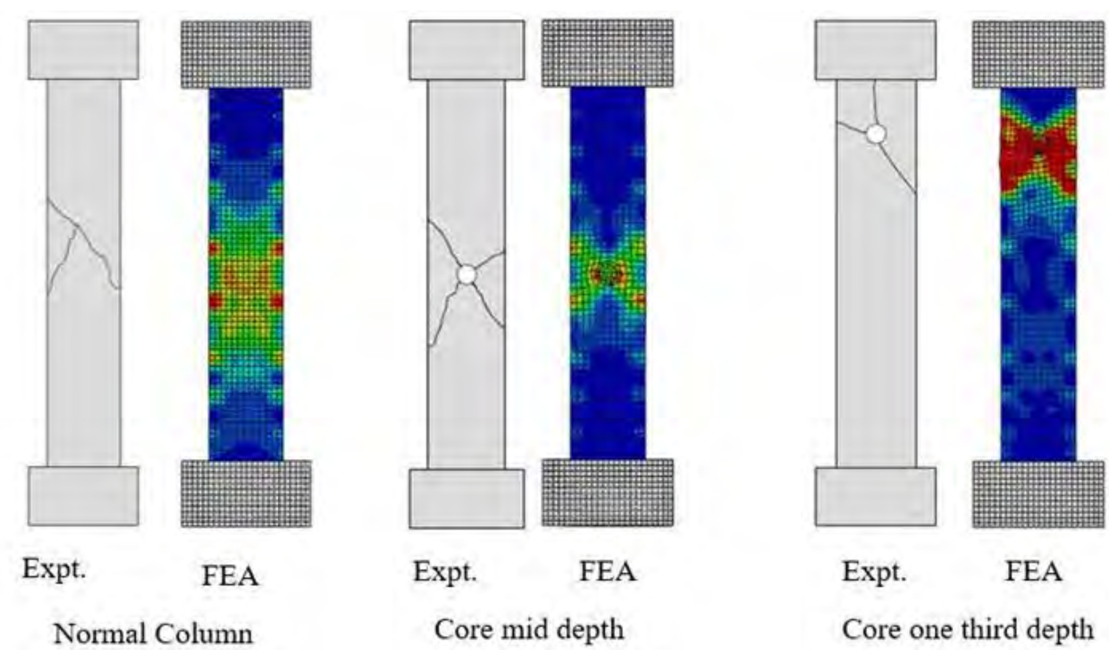


Figure 4-6 Comparison of crack pattern between exp. and FEA.

Table 4-4 Column capacity comparison between experimental and FEA.

Concrete Strength, fc' MPa	Normal Column (NC)		Core Mid height (CMD)				Core One Third Height (COD)			
	Axial Capacity KN (Expt.)	Axial Capacity KN (FEA)	Axial Capacity KN (Expt.)	Axial Capacity KN (FEA)	% of Capacity Reduction (Expt.)	% of Capacity Reduction (FEA)	Axial Capacity KN (Expt.)	Axial Capacity KN (FEA)	% of Capacity Reduction (Expt.)	% of Capacity Reduction (FEA)
30.10	1183.3	1161.7	1085.3	1072.7	8.3	7.7	955.7	948.1	19.20	18.40

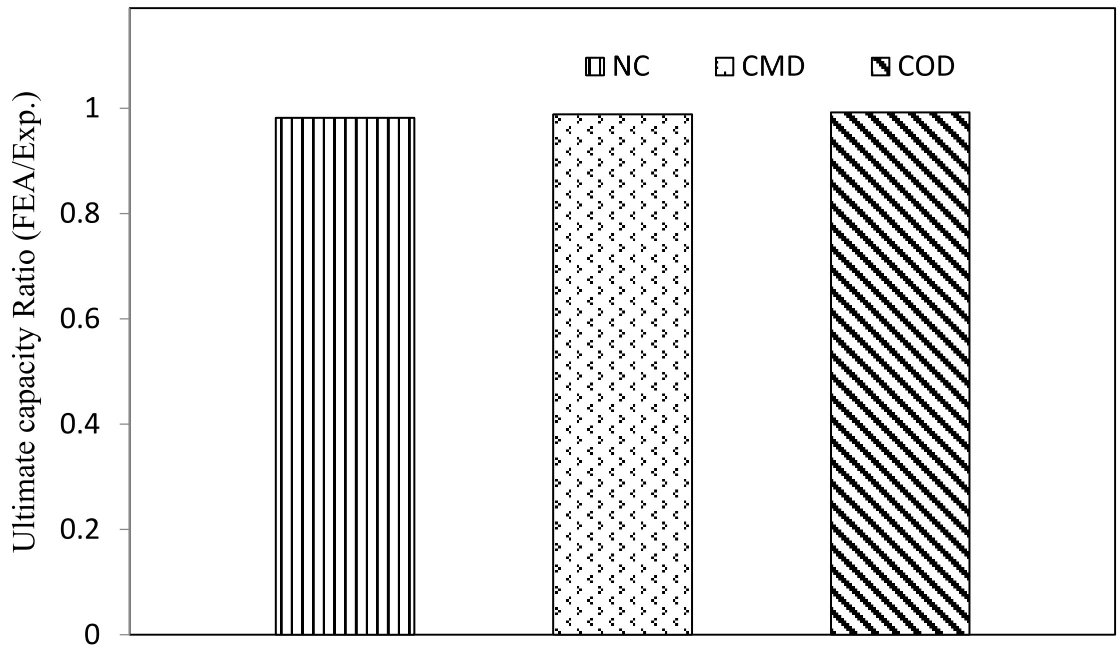


Figure 4-7 Ultimate strength ratio (FEA/EXP.)

Chapter 5

PARAMETRIC STUDY USING FEA

5.1 General

A parametric study has been conducted by simulating behavior of full scale columns with and without cores in ABAQUS environment using Concrete Damage Plasticity (CDP) models. The parameters varied were column dimensions, tie spacing, concrete strength and core size. The varying parameters have been chosen to represent typical column sizes, concrete compressive strength and tie spacing used in the country. The various CDP parameters have been evaluated for different strengths of concrete considered in this study. Eventually, the ultimate capacities of columns under pure axial load have been evaluated and the effect of core cutting on column capacity has been made have been presented in this chapter of thesis.

5.2 Column Sizes, Concrete strength and Core Size

As mentioned above, column size, concrete compressive strength and core sizes have been varied for FEA simulation in ABAQUS environment for a better understanding of the effect of all these parameters on capacity of columns when core has been drilled out. The column dimensions and core size have been varied from 300mm x 300mm to 600mm x 600mm and 50mm x 100mm to 100mm x 200, respectively. The concrete compressive strength has been varied from 13.5 MPa to 34.5 MPa. Table 5-1 shows the values of different parameters considered in this study.

5.3 Tie Bar Spacing

Tie bar spacing of columns has also varied in FEA simulation to observe how confinement affects the capacity of columns due to core cutting. In this regard, three types of tie bar spacing have been used. For Intermediate Moment Resisting Frame (IMRF), tie bar spacing of 100mm near support to one third height and 150mm at mid one third heights of column is being recommended as per code. In addition to this code specified spacing, two more tie bar spacing of 75mm and 125mm near support to one third height and 150mm and 200mm at mid one third height of column, respectively have been considered. The tie bar spacing have been categorized as Category-I, Category-II, Category-III (Table 5-1). Larger spacing has been considered to simulate poorly designed structures since it has been observed that several old buildings do not have code specified tie reinforcement. The main reinforcement has been kept constant to 1% of the cross-sectional area of columns throughout the analysis. The test matrix is shown in Table 5-1.

Table 5-1 Test matrix for parametric study.

Strength, MPa	Column size, mm	Column Status	Core Size, mm	Tie bar Spacing
13.5	300 x 300	Normal column (NC)	50 x 100	Category-I(T1): 75mm Near support to one third height and 125mm at mid one third height of column
17.2	300 x 375		75 x 150	
20.7	300 x 450		100 x 200	Category-II(T2): 100mm Near support to one third height and 150mm at mid one third height of column
27.6	300 x 525	Core mid height (CMD)		
34.5	375 x 375			Category-III(T3): 150mm Near support to one third height and 200mm at mid one third height of column
	375 x 450			
	375 x 525			
	450 x 450			
	450 x 525			
	525 x 525			
	600 x 600			

5.4 CDP Parameters

Concrete damage plasticity (CDP) model based on the combination of damage mechanics and plasticity which is developed to analyze the failure of concrete structures. The aim is to describe the important characteristics of the failure process of concrete subjected to axial loading. This is achieved by combining an effective stress based plasticity model with a damage model based on plastic and elastic strain measures. Furthermore, the model is applied to the structural analyses of tensile and compressive failure.

5.4.1 Element Type

The FEA simulation of a RCC structure needs an accurate model of the structural elements and its composite portion of concrete and steel. An exact model that has been simulatedin ABAQUS is shown in Figure 5-1.

Figure 5-1 ABAQUS normal column solid model with meshing

In ABAQUS, the model can be extruded in any direction; this is why a 3D solid element in “modeling space” using deformable type for column was created in this study. In order to develop concrete column, 8-Node linear brick element (C3D8R) was utilized. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. The Posisson’s coefficient of concrete column was kept as 0.19. Necessary partitions of the concrete column (varying sizes) were made to facilitate meshing at core region. Column was meshed with cubic 3D element that satisfied the FEA model. Also a mesh size vs ultimate load graph was developed which showed that mesh size of 15 mm satisfied the element size. The longitudinal and tie bar were modeled as 2-node truss element which is called T3D2. Truss elements are rods that can carry only tensile or compressive loads. They have no resistance to bending; as in this study column has been tested under pure axial loading, hence, T3D2 has been a good choice to model reinforcement. Figure 5-2 represents detail reinforcement arrangement inside the column.

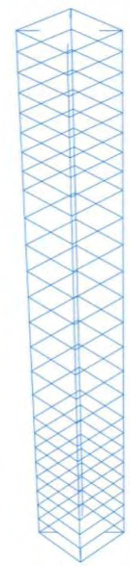


Figure 5-2 ABAQUS normal column steel reinforcement model with meshing.

5.4.2 Strength Parameters

The CDP parameters were determined for the following concrete strength of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa which were used in this parametric study. The CDP parameters for the above mentioned concrete strength are shown in the Table 5-2 through Table 5-16 and Figure 5-1 through Figure 5-10.

Table 5-2 CDP Parameters for concrete strength of 13.5 MPa.

E, GPa	Poisson ratio, μ	Dilation angle, β	Eccentricity, m	f=fb0/fb	k	viscosity, γ
17.6	0.19	35	0.1	1.16	0.666	0

Table 5-3 Generalized compression behavior for concrete strength of 13.5 MPa.

Stress, Mpa	Strain
0.6894759	0
6.2442498	0.000422
10.827086	0.000843
13.171223	0.0012647
13.789518	0.001686
13.433283	0.0021078
12.641599	0.002529
11.711729	0.0029509
6.7944193	0.0059019
3.3606286	0.0118038

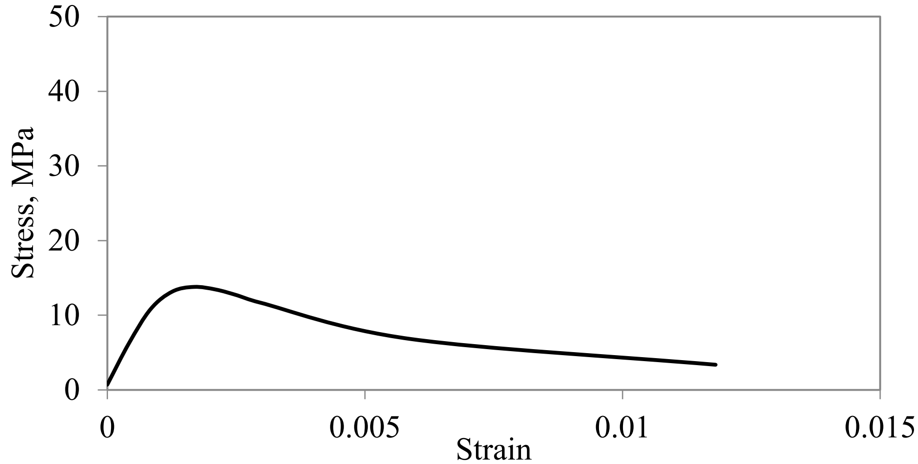


Figure 5-3 Generalized compression behavior for concrete strength of 13.5 MPa. (Farid and Janabi, 1990)

Table 5-4 Tensile behavior for concrete strength of 13.5 MPa.

Stress, MPa	Strain
0.21	0
0.30	3.33E-05
0.20	0.00016
0.09	0.00028
0.02	0.000685
0.01	0.001087

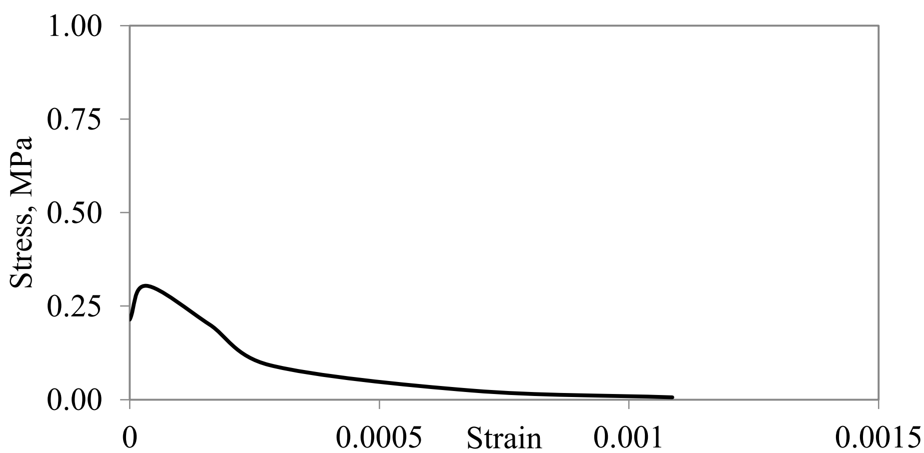


Figure 5-4 Tensile behavior for concrete strength of MPa. (Farid and Janabi, 1990)

Table 5-5 CDP Parameters for concrete strength of 17.2 MPa.

E, GPa	Poisson ratio, μ	Dilation angle, β	Eccentricity, m	f=fb0/fb	k	viscosity, γ
19.65	0.19	35	0.1	1.16	0.666	0

Table 5-6 Generalized compression behavior for concrete strength of 17.2 MPa.

Stress, MPa	Strain
0.86184489	0
7.80531222	0.000446
13.5338574	0.000891
16.464029	0.0013372
17.2368977	0.001783
16.7916036	0.0022287
15.8019987	0.002674
14.6396607	0.0031202
8.49302417	0.0062405
4.20078578	0.012481

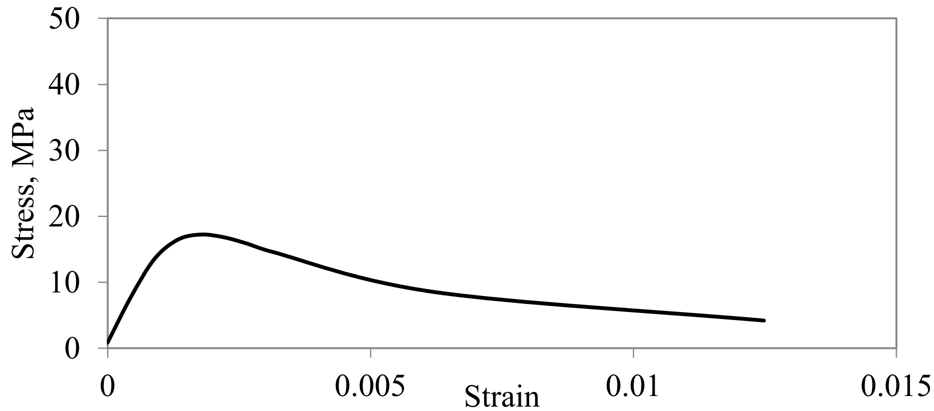


Figure 5-5 Generalized compression behavior for concrete strength of 17.2 MPa. (Farid and Janabi, 1990)

Table 5-7 Tensile behavior for concrete strength of 17.2 MPa.

Stress, MPa	Strain
0.27	0
0.38	3.33E-05
0.25	0.00016
0.12	0.00028
0.03	0.000685
0.01	0.001087

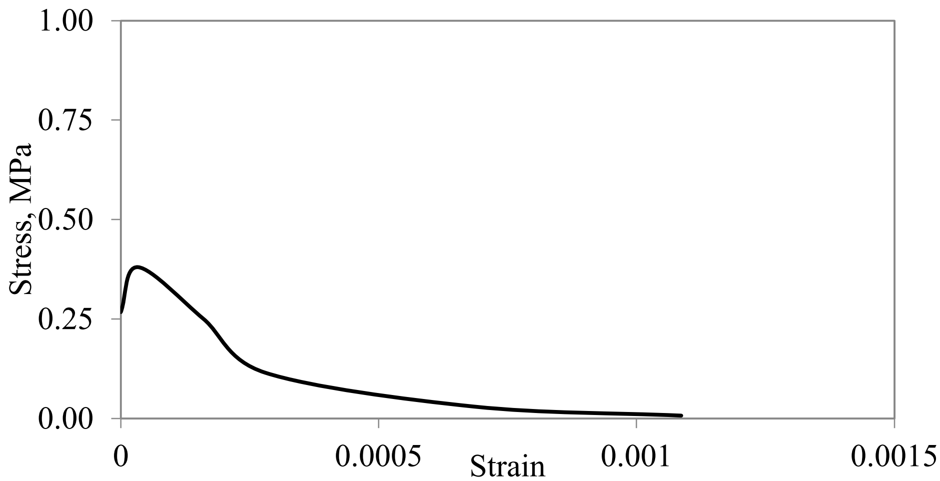


Figure 5-6 Tensile behavior for concrete strength of 17.2 MPa. (Farid and Janabi, 1990)

Table 5-8 CDP Parameters for concrete strength of 20.7 MPa.

E, GPa	Poisson ratio, μ	Dilation angle, β	Eccentricity, m	f=fb0/fb	k	viscosity, γ
21.52	0.19	35	0.1	1.16	0.666	0

Table 5-9 Generalized compression behavior for concrete strength of 20.7 MPa.

Stress, MPa	Strain
1.0342139	0
9.3663747	0.000467
16.240629	0.000933
19.756835	0.0014
20.684277	0.001866
20.149924	0.002333
18.962398	0.002799
17.567593	0.003266
10.191629	0.006532
5.0409429	0.013063

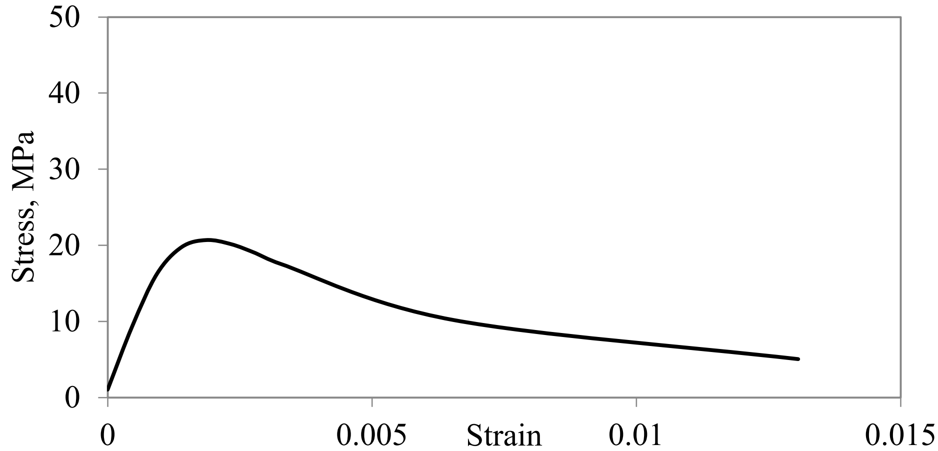


Figure 5-7 Generalized compression behavior for concrete strength of 20.7 MPa. (Farid and Janabi, 1990)

Table 5-10 Tensile behavior for concrete strength of 20.7 MPa.

Stress, MPa	Strain
0.32	0
0.46	3.33E-05
0.30	0.00016
0.14	0.00028
0.04	0.000685
0.01	0.001087

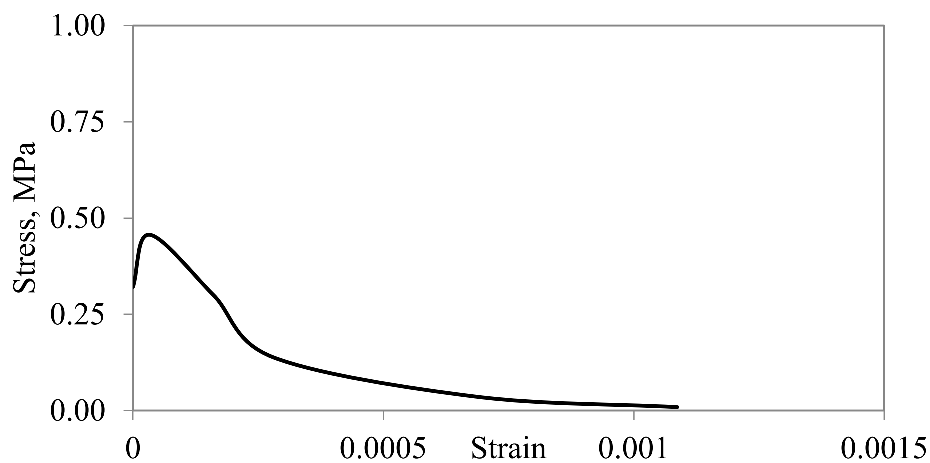


Figure 5-8 Tensile behavior for concrete strength of 20.7 MPa. (Farid and Janabi, 1990)

Table 5-11 CDP Parameters for concrete strength of 27.6 MPa.

E, GPa	Poisson ratio, μ	Dilation angle, β	Eccentricity, m	f=fb0/fb	k	viscosity, γ
24.85	0.19	35	0.1	1.16	0.666	0

Table 5-12 Generalized compression behavior for concrete strength of 27.6 MPa.

Stress, MPa	Strain
1.3789518	0
12.4885	0.000501
21.654172	0.001003
26.342446	0.001504
27.579036	0.002005
26.866566	0.002507
25.283198	0.003008
23.423457	0.003509
13.588839	0.007019
6.7212572	0.014037

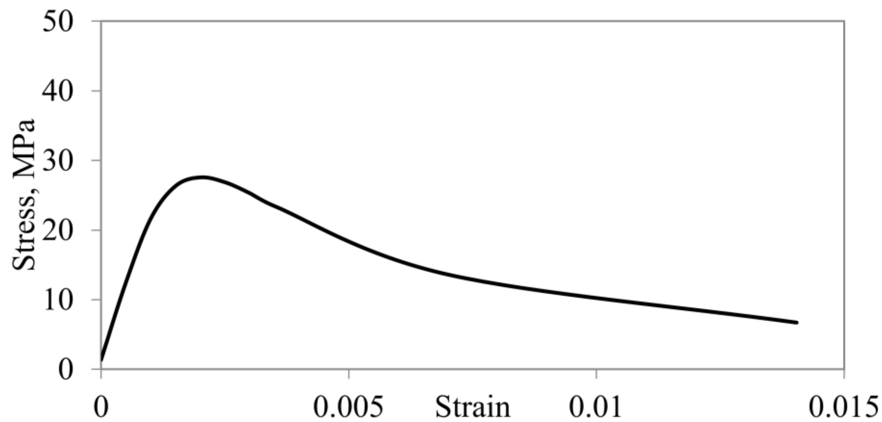


Figure 5-9 Generalized compression behavior for concrete strength of 27.6 MPa. (Farid and Janabi, 1990)

Table 5-13 Tensile behavior for concrete strength of 27.6 MPa.

Stress, MPa	Strain
0.43	0
0.61	3.33E-05
0.40	0.00016
0.18	0.00028
0.05	0.000685
0.01	0.001087

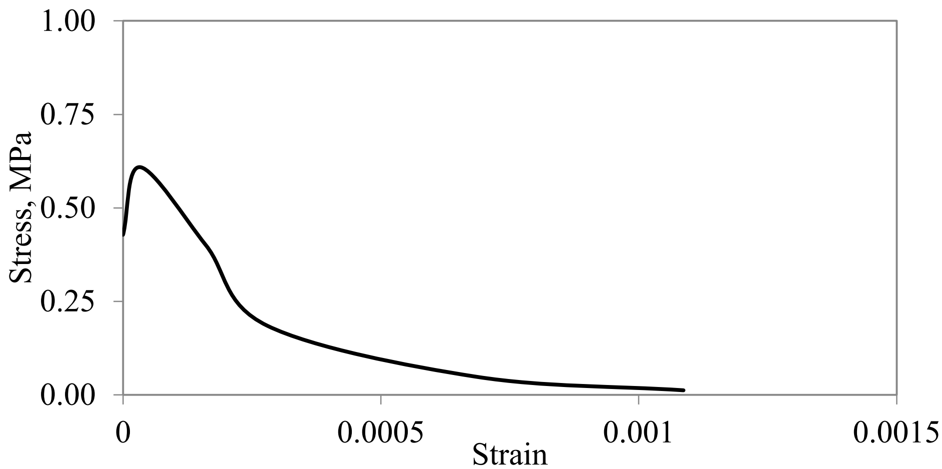


Figure 5-10 Tensile behavior for concrete strength of 27.6 MPa.

Table 5-14 CDP Parameters for concrete strength of 34.5 MPa. (Farid and Janabi, 1990)

E, GPa	Poisson ratio, μ	Dilation angle, β	Eccentricity, m	f=fb0/fb	k	viscosity, γ
27.78	0.19	35	0.1	1.16	0.666	0

Table 5-15 Generalized compression behavior for concrete strength of 34.5 MPa.

Stress, MPa	Strain
1.72	0
15.61	0.000530
27.07	0.001060
32.93	0.00159
34.47	0.002120
33.58	0.00265
31.60	0.003181
29.28	0.003711
16.99	0.007421
8.40	0.014842

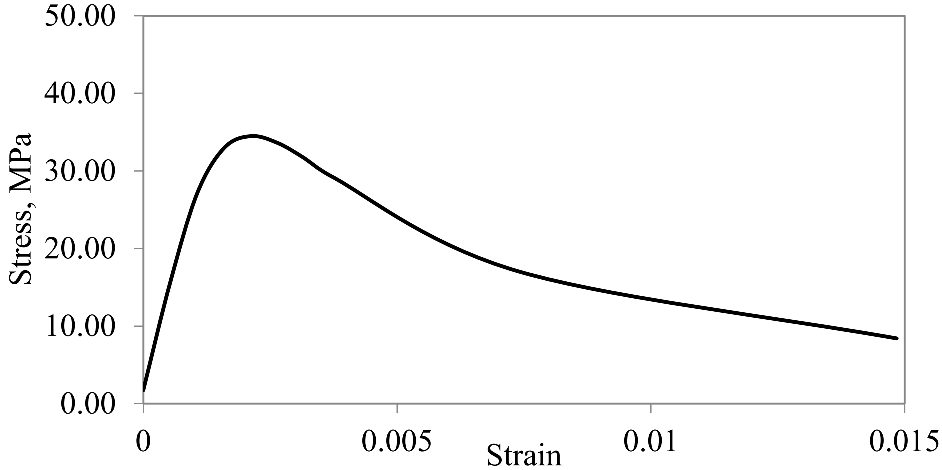


Figure 5-11 Generalized compression behavior for concrete strength of 34.5 MPa. (Farid and Janabi, 1990)

Table 5-16 Tensile behavior for concrete strength of 34.5 MPa.

Stress, MPa	Strain
0.54	0
0.76	3.33E-05
0.50	0.00016
0.23	0.00028
0.06	0.000685
0.02	0.001087

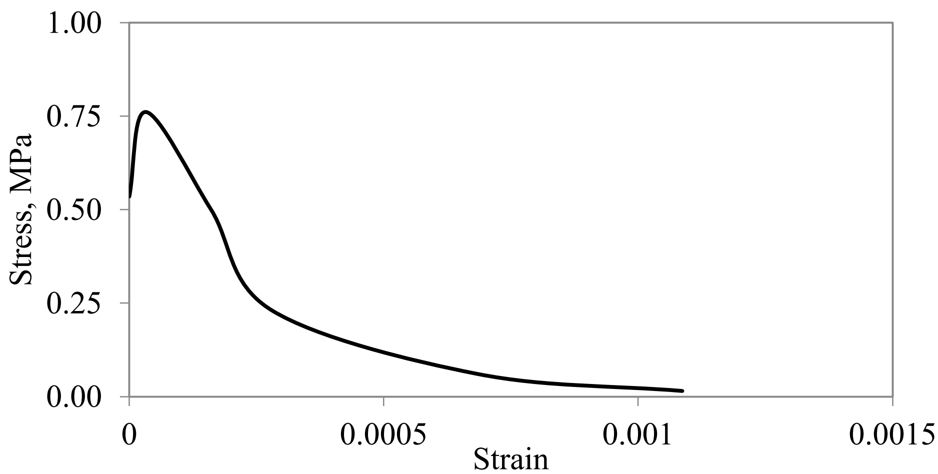


Figure 5-12 Tensile behavior for concrete strength of 34.5 MPa. (Farid and Janabi, 1990)

Chapter 6

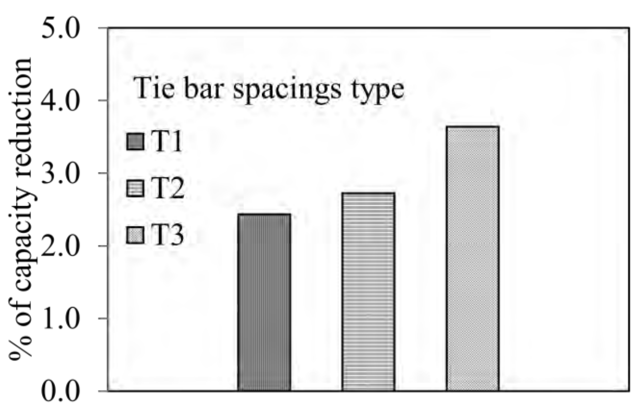
RESULT AND DISCUSSION OF PARAMETRIC STUDY

6.1 General

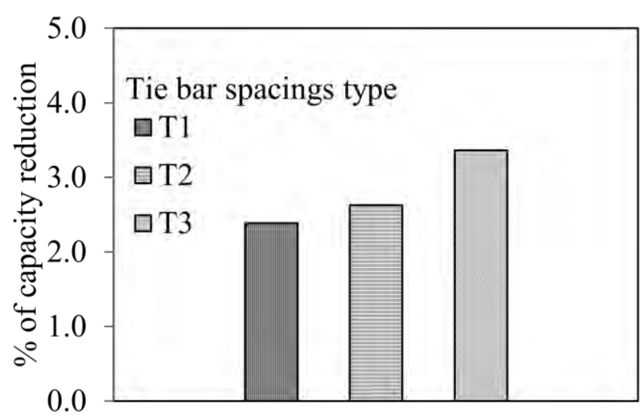
Several parameters have been incorporated in this study for a comprehensive evaluation of the effect of core cutting on behavior of columns. In the previous chapter, the validation of the FEA models by investigating the profound conformation with the experimental results has been discussed for several factors that are directly or indirectly affecting the strength of core drilled column. For a broader perspective, some additional factors have been included in this chapter such as three different types of ties bar spacing (Category-I- 75mm & 125mm, Category-II- 100mm & 150mm, Category-III-150mm & 200mm), five types of concrete strength(13.5, 17.2, 20.7, 27.6 and 34.5 MPa), three different core sizes(50mm, 75mm, 100mm) and various column sizes (300mm x 300mm to 600mm x 600mm). All these parameters have been considered and simulated in FEA to establish a correlation among those parameters and to obtain a guideline in a graphical form for assurance of safe core extraction.

6.2 Tie Bar Spacing

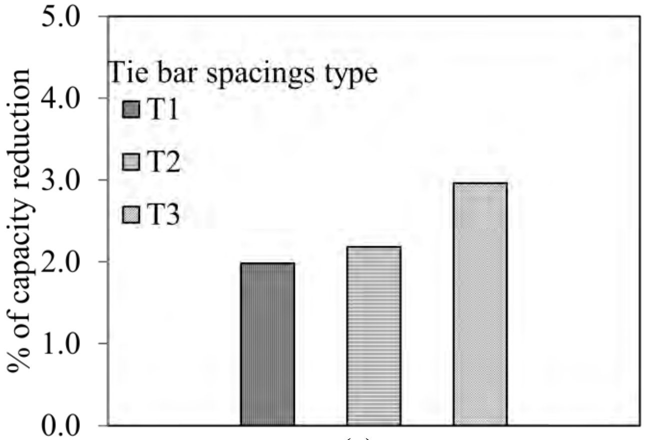
Tie bar spacing of columns has been varied in three categories (Category-I, Category-II and Category-III) in FEA simulation to observe how tie bar spacing affects the capacity of columns due to core cutting (Table 5-1). From FEA simulation, it has been observed that tie bar spacing has significant effect on capacity reduction of columns with 50mm drilled cores as compared to NC columns without cores for concrete strength of 13.5 MPa as shown in Figure 6-1 (a) to Figure 6-1 (i). In case of a 300mm x 300mm column in Figure 6-1(a), there has been a capacity reduction of 2.43%, 2.72% and 3.64% corresponding to tie bar spacing of Category-I, Category-II and Category-III, respectively. With an increase in tie bar spacing from Category-I to Category-II and Category-II to Category-III, there has been a significant rise in capacity reduction of 12.02% and 33.64%, respectively. Similarly, for 300mm x 375mm column in Figure 6-1(b), capacity reductions of 2.39%, 2.63% and 3.36 % have been obtained for the three types of tie bar spacing of Category-I, Category-II, Category-III, respectively. The percentage increase in capacity reductions has been obtained as 10.08% and 28.08% for an increase in tie bar spacing from Category-I to Category-II and Category-II to Category-III, respectively. A similar pattern has been obtained for 300mm x 450mm columns in Figure 6-1(c) with corresponding capacity reductions of 1.98%, 2.18% and 2.96 % for Category-I, Category-II and Category-III, respectively. The increase in capacity reductions obtained in this case has been 10.10% and 35.80% for an increase in tie bar spacing from Category-I to Category-II and Category-II to Category-III, respectively. For other higher sizes of columns from Figure 6-1(d) to 6-1(i), it has been noticed that the capacity reduction due to core drilling has also increased with an increase of tie bar spacing. In case of square column of 375mm x 375mm as shown in Figure 6-1(d), the capacity reduction of core drilled column has been obtained as 1.59%, 1.96% and 2.86%, for tie bar spacing of Category-I, Category-II and Category-III, respectively. There has been an increase in capacity reduction of 23.09% and 46.54% corresponding to an increase in tie bar spacings from Category-I to Category-II and Category-II to Category-III, respectively. It has also been noticed that the capacity reduction has always been higher for column having greater tie bar spacing. The tie bar spacing effect is quite similar for other higher sizes of column as shown in Figure 6-1(e) through 6-1(i).

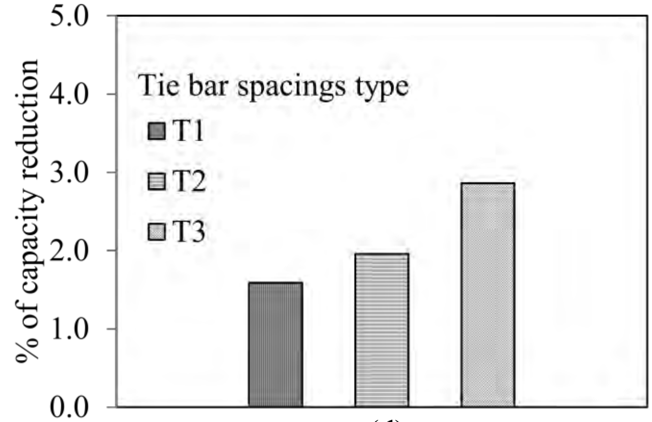


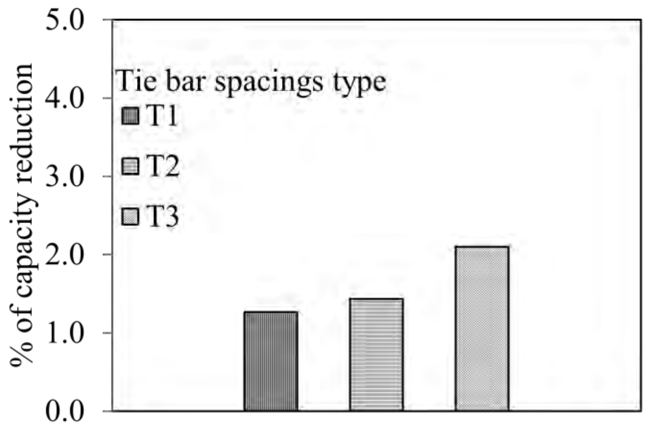
(a)

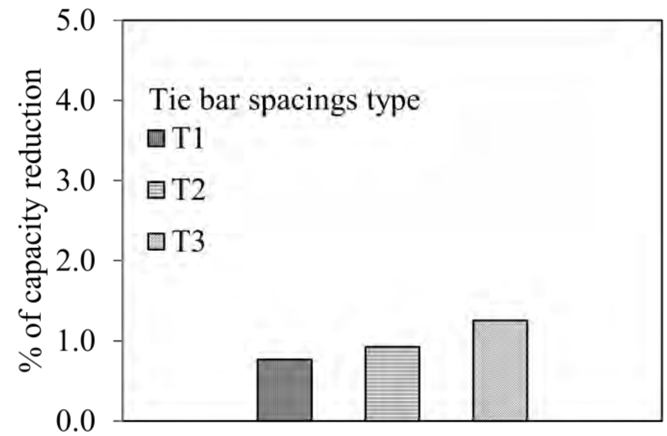


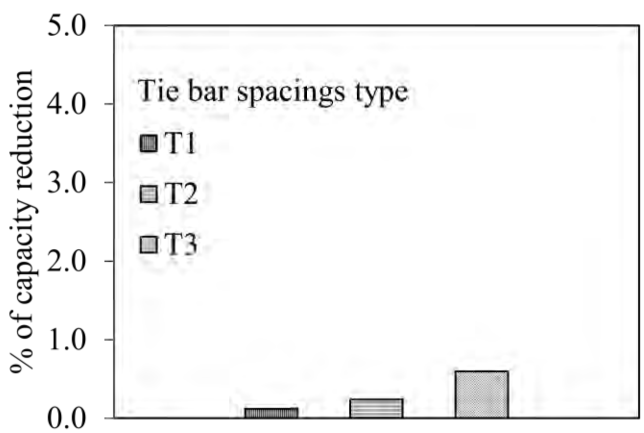
(b)

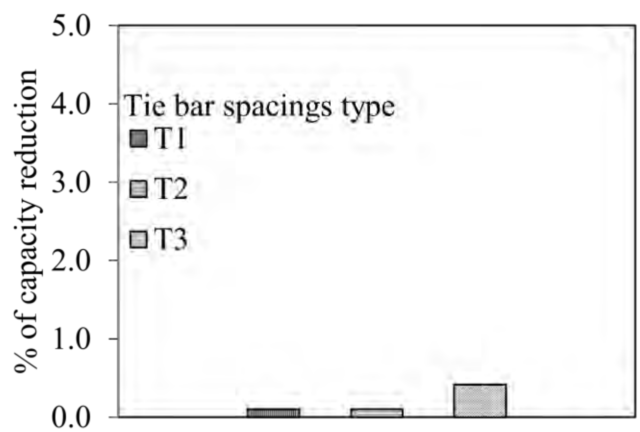

% of capacity reduction
5.0
4.0
3.0
2.0
1.0
0.0
Tie bar spacings type
T1
T2
T3
(c)

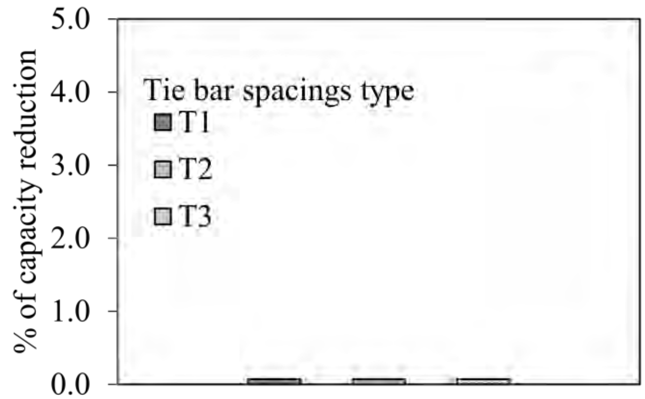

% of capacity reduction
5.0
4.0
3.0
2.0
1.0
0.0
Tie bar spacings type
T1
T2
T3
(d)


% of capacity reduction
5.0
4.0
3.0
2.0
1.0
0.0
Tie bar spacings type
T1
T2
T3
(e)


% of capacity reduction
5.0
4.0
3.0
2.0
1.0
0.0
Tie bar spacings type
T1
T2
T3
(f)


% of capacity reduction
5.0
4.0
3.0
2.0
1.0
0.0
Tie bar spacings type
T1
T2
T3
(g)

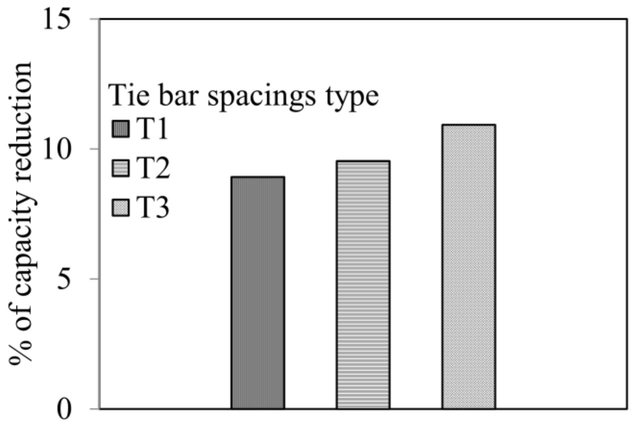

% of capacity reduction
5.0
4.0
3.0
2.0
1.0
0.0
Tie bar spacings type
T1
T2
T3
(h)



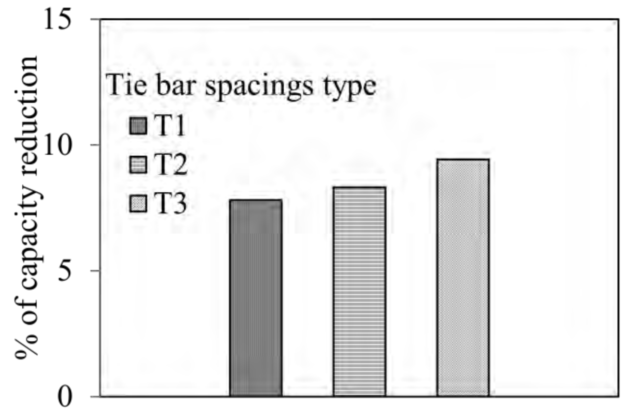
(i)

Figure 6-1 Effect of lateral reinforcement spacing on columns with 50mm x 100mm for 13.5 MPa concrete. (a) 300mm x 300mm, (b)300mm x 375mm, (c)300mm x 450mm, (d)375mm x 375mm, (b)300mm x 525mm (f)450mm x 450mm (g) 450mm x 525mm (h) 525mm x 525mm & (i) 600mm x 600mm.

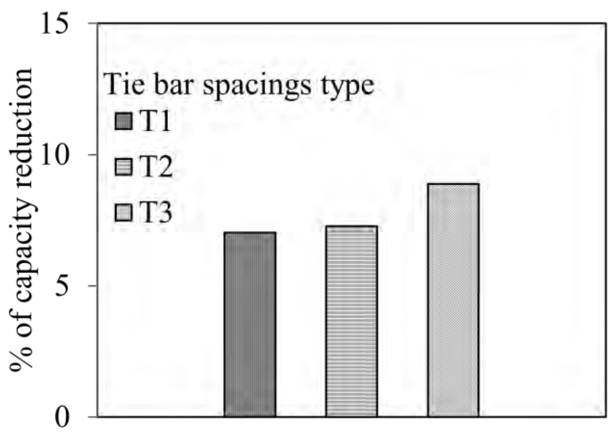
A similar analysis has been done for columns having core with 75mm diameter with concrete strength of 13.5 MPa as shown in Figure 6-2(a) to 6-2(i). For 300mm x 300mmcolumn size as shown in Figure 6-2(a), the capacity reduction has been found as 8.92%, 9.54% and 10.92 % for tie bar spacing of Category-I, Category-II and Category-III, respectively. As the tie bar spacing has increased from Category-I to Category-II and Category-II to Category-III, there has been an increase in capacity reduction by 6.95% and 14.46%, respectively. A similar pattern has been obtained for 150mmx375mm columns in Figure 6-2(b) with corresponding capacity reductions of 7.81%, 8.32% and 9.42 % for three different tie bar spacing of Category-I, Category-II and Category-III, respectively. As the tie bar spacing has increased from Category-I to Category-II and Category-II to Category-III, the capacity reduction has increased by 6.53% and 13.22%, respectively. Similarly, for 150mmx450mm column in Figure 6-2(c), capacity reductions of 7.03%, 7.27% and 8.89% have been obtained for the three types of tie bar spacing. The percentage increase in capacity reduction has been obtained as 3.49% and 22.20% for an increase in tie bar spacing from Category-I to Category-II and Category-II to Category-III, respectively. In case of square column of 375mm x 375mm size as shown in Figure 6-2(d), it has been noticed that the capacity reduction of core drilled column has been found as 3.71%, 3.91% and 5.90% for tie bar spacing of column of Category-I, Category-II and Category-III, respectively. As the tie bar spacing has been increased from Category-I to Category-II and Category-II to Category-III, the capacity reduction has increased by 5.50% and 50.80%, respectively. Similar effect has been noticed for other higher sizes of columns such as 150mm x 525mm, 375mm x 450mm, 450mm x 450mm, 525mm x 525mm as shown from Figure 6-2(e) to 6-2(i).



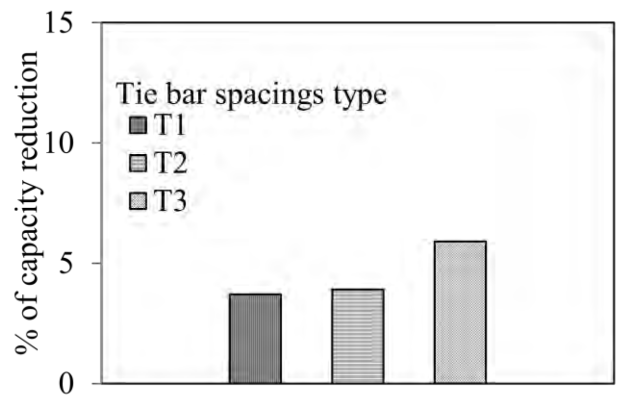
(a)



(b)



(c)



(d)

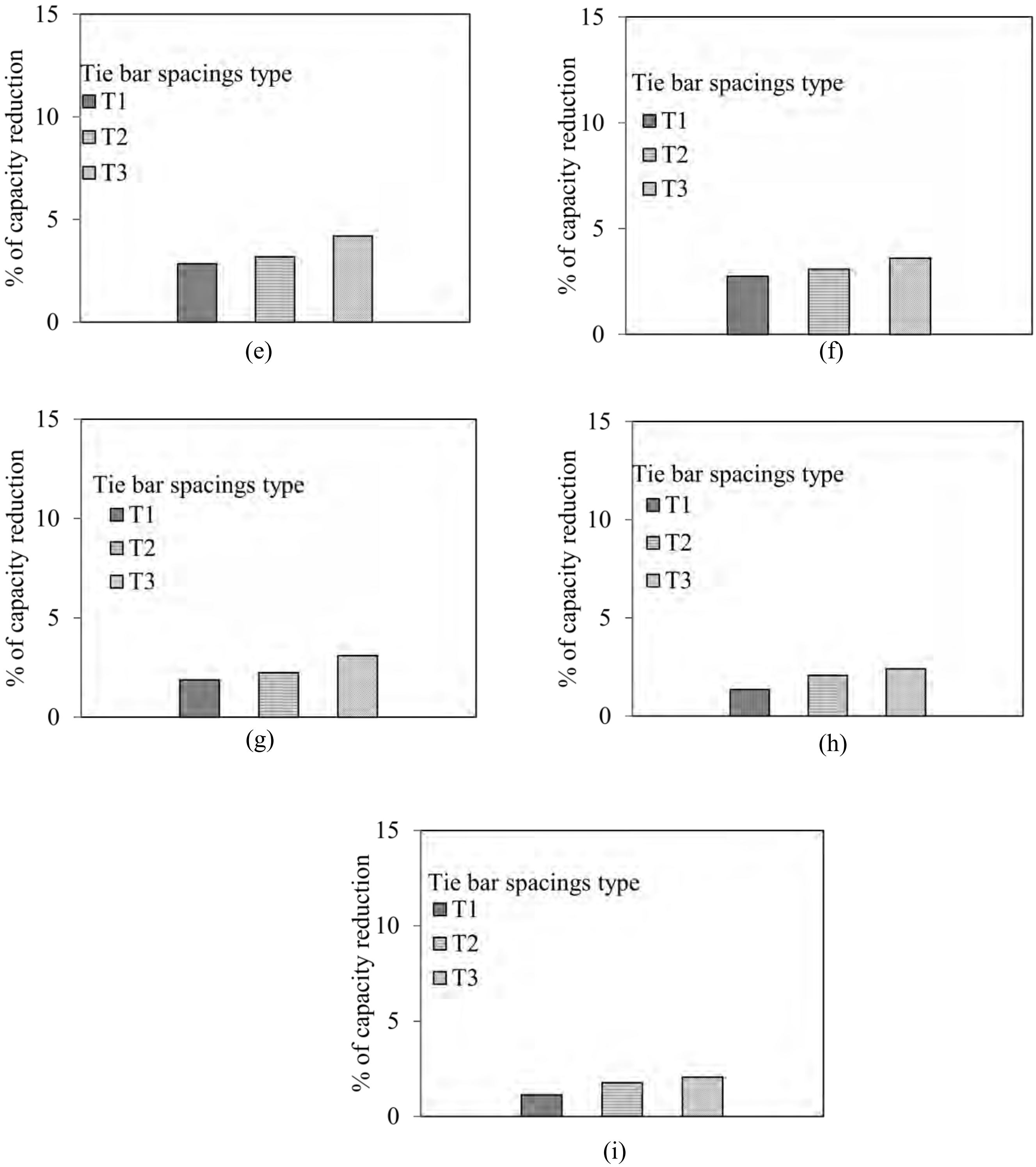
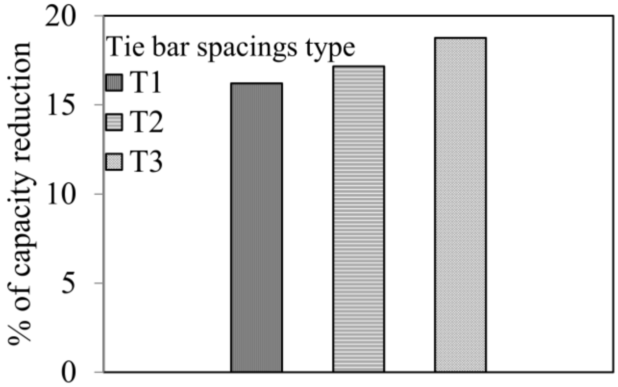


Figure 6-2 Effect of lateral reinforcement spacing on columns with 75mm x 150mm for 13.5 MPa concrete. (a) 300mm x 300mm, (b)300mm x 375mm, (c)300mm x 450mm, (d)375mm x 375mm, (b)300mm x 525mm (f)450mm x 450mm (g)450mm x 525mm (h) 525mm x 525mm & (i) 600mm x 600mm.

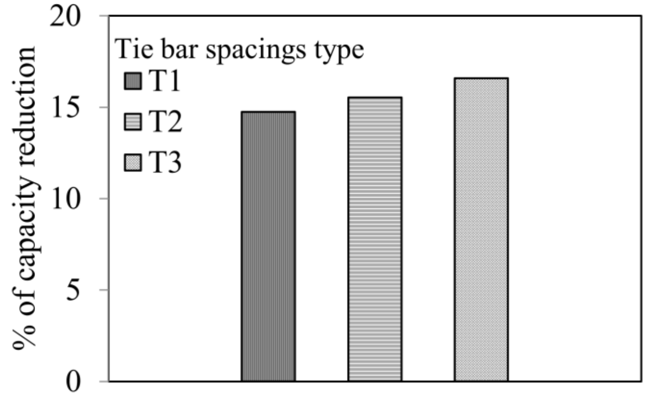
A similar FEA analysis has been performed for columns of concrete strength 13.5 MPa for 100mm core and the results has been illustrated from Figure 6-3(a) through 6-3(i). In case of a 300mm x 300mmcolumn in Figure 6-3(a), there has been a capacity reduction of 16.22%, 17.17% and 18.77% corresponding to tie bar spacing of Category-I, Category-II and Category-III, respectively. As the tie bar spacing has been increased from Category-I to Category-II and Category-II to Category-III, there has been a significant rise in capacity reduction by 5.85% and 9.30%, respectively. Similarly, for a 150mmx375mm column in Figure 6-3(b), capacity reductions of 14.75%, 15.54% and 16.59 % have been obtained for the three types of tie bar spacing of Category-I, Category-II and Category-III, respectively. With an increase in tie bar spacing from Category-I to Category-II and Category-II to Category-III, the capacity reduction has been increased by 5.35% and 6.75%, respectively. A similar pattern has been obtained for 150mmx450mm columns in Figure 6-3(c) with corresponding capacity reductions of 3.69%, 14.18% and 15.37 % for different spacing. The increase in capacity reduction obtained in this case has been 3.57% and 8.39% for an increase in tie bar spacing from Category-I to Category-II and Category-II to Category-III, respectively. A square column of 375mm x 375mm as shown in Figure 6-3(d) represents similar effect as a rectangular column. In case of square column, as illustrated in Figure 6-13(d), it has been observed that the capacity reduction of core drilled column has been found to be 10.78%, 11.39% and 12.70%, respectively for different tie bar spacings. The capacity reduction showed an elevation of 5.65% and 11.50% corresponding to increase tie bar spacing from Category-I to Category-II and Category-II to Category-III, respectively. It has been obvious from the observed outcomes that the capacity reduction has been always higher for column having greater tie bar spacing. The tie bar spacing effect is quite similar for other higher sizes of column as shown in Figure 6-3(e) through 6-3(i).

The FEA simulation is done on different varying concreter strength of 17.2, 20.7, 27.6 and 34.5 MPa and the tie bar spacing effect is quite similar as 13.5 MPa concrete as discussed above as shown in Chapter -7 Figure 7-1 through Figure 7-15 and their results is also shown as a tabular form in APPENDIX Table 8-1 through Table 8-6.



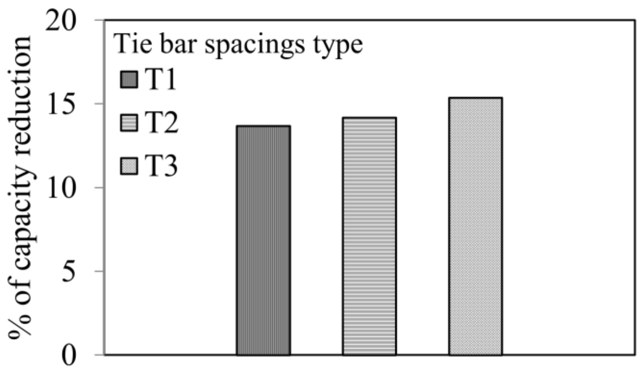
20
15
10
5
0
% of capacity reduction
Tie bar spacings type
T1
T2
T3

(a)



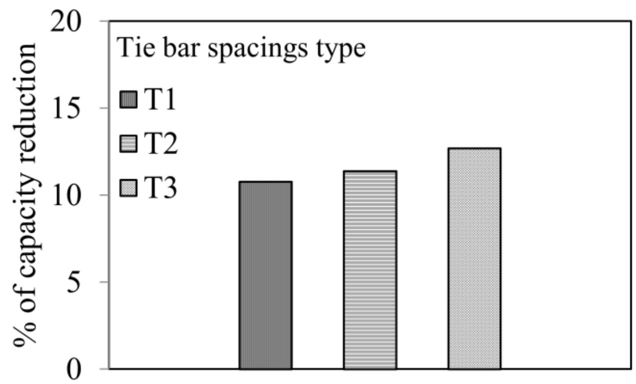
20
15
10
5
0
% of capacity reduction
Tie bar spacings type
T1
T2
T3

(b)



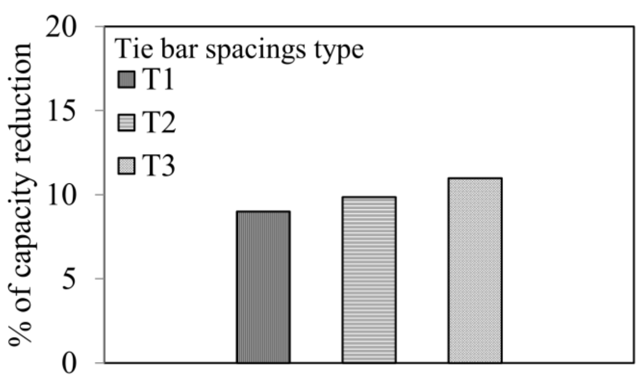
20
15
10
5
0
% of capacity reduction
Tie bar spacings type
T1
T2
T3

(c)



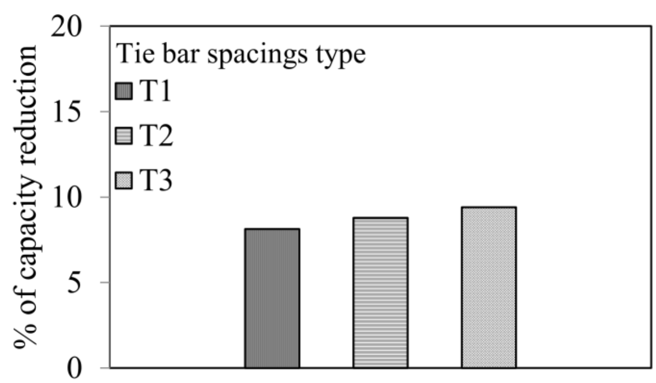
20
15
10
5
0
% of capacity reduction
Tie bar spacings type
T1
T2
T3

(d)



20
15
10
5
0
% of capacity reduction
Tie bar spacings type
T1
T2
T3

(e)



20
15
10
5
0
% of capacity reduction
Tie bar spacings type
T1
T2
T3

(f)

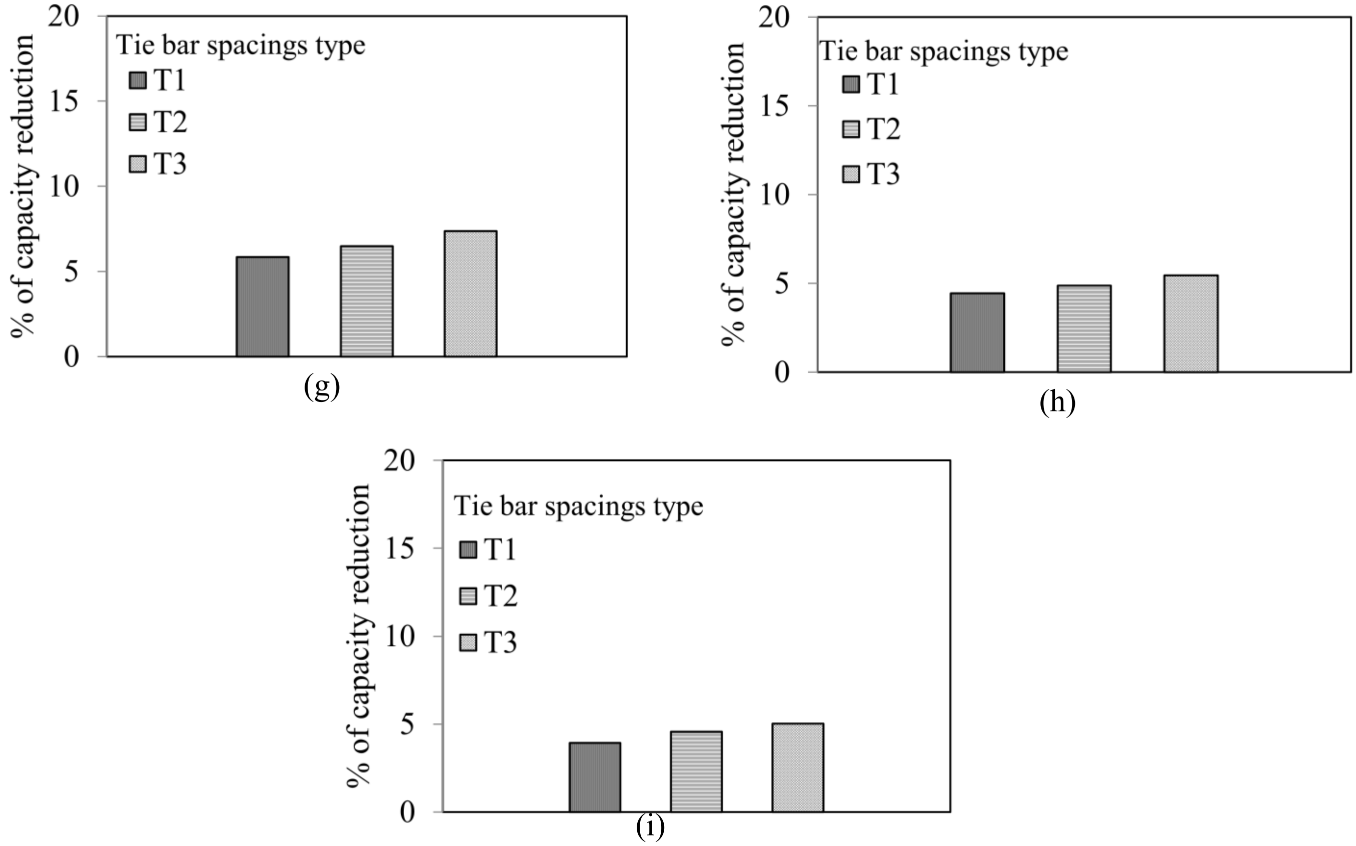


Figure 6-3 Effect of lateral reinforcement spacing on columns with 100mm x 200mm for 13.5 MPa concrete. (a) 300mm x 300mm, (b)300mm x 375mm, (c)300mm x 450mm, (d)375mm x 375mm, (b)300mm x 525mm (f)450mm x 450mm (g)450mm x 525mm (h) 525mm x 525mm & (i) 600mm x 600mm.

It has been observed in all cases that the tie bar spacing has a pronounced effect on capacity reduction of columns with higher reduction of capacity corresponding to higher spacing. A similar behavior has been obtained from the simulation of both square and rectangular columns. So it can be concluded that higher tie bar spacing would result in a higher amount of capacity reduction for columns with cores of any size. This phenomenon is accountable to the fact that smaller tie bar spacing increases confinement at core region which ultimately increases load carrying capacity of core drilled columns. Increased confinement assists in reducing stress concentration effect at the core region of columns. As a result, smaller tie bar spacing increases stiffness which results in higher core column capacity.

A similar FEA simulation has also been performed on different concrete strengths of 17.2, 20.7, 27.6 and 34.5 MPa which has also portrayed a similar effect of tie spacing on column behavior.in all cases. The corresponding graphs for other different compressive strength of concrete have been illustrated in Figure 7-1 through Figure 7-15 of Chapter-7 and are also shown as a tabular form in APPENDIX Table 8-1 through Table 8-6.

6.3 Concrete Strength

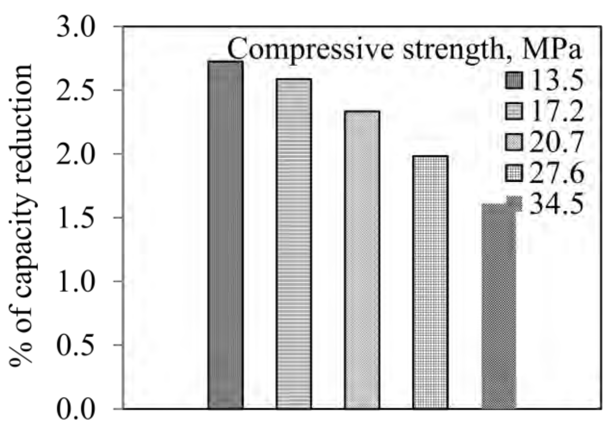
In this section, the compressive strength of concrete has been varied in FEA simulation to observe it affects the percentage of capacity reduction of column due to core cutting. In this regards, five different concrete strengths of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa have been used. The range of concrete strength has been selected in such a way so as to replicate the typical concrete strengths requirement found in the field study. From the outcomes of FEA simulation for a certain core diameter of 50mm, it can been noticed from Figure 6-4(a) through 6-4(i) that strength of concrete has a significant effect on percentage of capacity reduction of core drilled column as compared to NC columns without cores.

In case of 300mm x 300mmcolumns in Figure 6-4(a), there has been capacity reductions of 2.72%, 2.59%, 2.34%, 1.98% and 1.61 % corresponding to concrete strengths of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa, respectively. With a progressive increase in concrete strength from 13.5 to 17.2 MPa, 17.2 to 13.5 MPa, 13.5 to 27.6 MPa & 27.6 to 34.5 MPa, there has been significant decreases in capacity reduction by 5.01%, 9.76%, 15.13% and 18.62%, respectively.

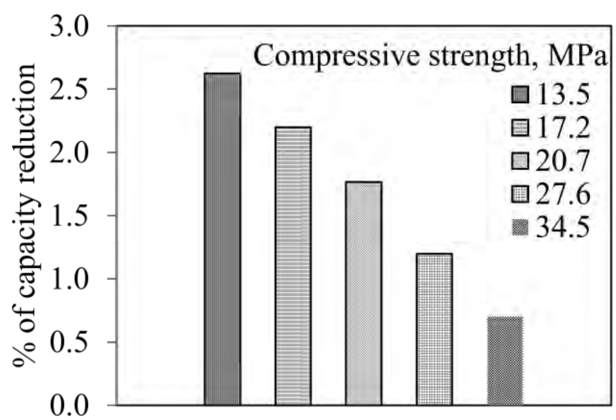
Similarly, for 150mm x 375mm columns in Figure 6-4(b), capacity reductions of 2.63%, 2.20%, 1.77%, 1.20% and 0.70 % have been obtained for concrete strengths of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa, respectively. Corresponding percentage decreases in capacity reduction of 2.63%, 2.20%, 1.77%, 1.20% and 0.70 % have been obtained for sequential increases in concrete strength from 13.5 to 17.2 MPa, 17.2 to 13.5 MPa, 13.5 to 27.6 MPa & 27.6 to 34.5 MPa, respectively. A similar pattern has been obtained for 150mmx450mm columns in Figure 6-4(c) with corresponding capacity reductions of 2.18%, 1.78%, 1.32%, 0.97% and 0.62% for different concrete strengths of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa, respectively. The decreases in capacity reduction obtained in this case of progressive increase have been 18.41%, 25.96%, 26.32% and 36.41% for an increase in concrete strength from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, respectively.

For other higher sizes of columns from Figure 6-4(d) through 6-4(i), it has been noticed that the capacity reduction due to core drilling has also decreased with an increasing concrete strength. In case of square columns of 375mm x 375mm as shown in Figure 6-4 (d), it has been observed that the capacity reductions of core drilled column have been found as 1.96%, 1.67%, 1.37%, 0.94% and 0.63% for the five concrete strengths analyzed in this research. The decreased capacity reductions has been found as 14.45%, 18.18%, 31.29% and 32.89% corresponding to increase in concrete strength from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, respectively. It has been inferred that the reduction in column capacity has been progressively becoming less prominent with an increase in concrete strength.. The concrete strength effect is quite similar for other higher sizes of columns as shown in Figure 6-4(e) through 6-4(i).

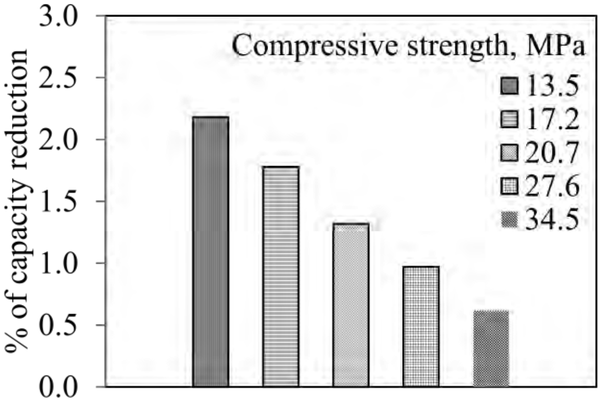
For other greater sizes of column in Figure 6-4(e) through 6-4(i), it is noticed that the percentage of capacity reduction is decreasing with the increasing of concrete strength. In each column it is observed that the percentage of capacity reduction is always smaller for column having greater concrete strength.



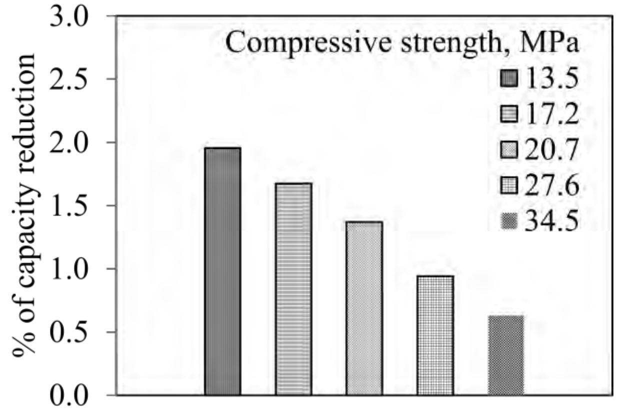
(a)



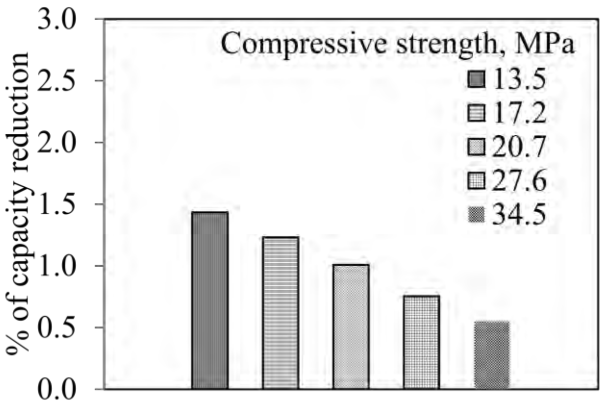
(b)


Compressive strength, MPa
13.5
17.2
20.7
27.6
34.5
% of capacity reduction
0.0
0.5
1.0
1.5
2.0
2.5
3.0

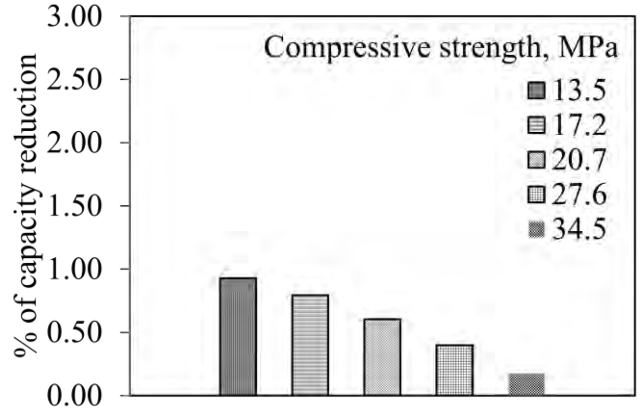
(c)


Compressive strength, MPa
13.5
17.2
20.7
27.6
34.5
% of capacity reduction
0.0
0.5
1.0
1.5
2.0
2.5
3.0

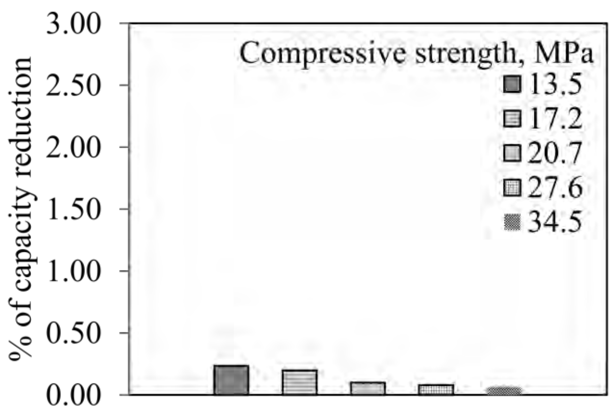
(d)


Compressive strength, MPa
13.5
17.2
20.7
27.6
34.5
% of capacity reduction
0.0
0.5
1.0
1.5
2.0
2.5
3.0

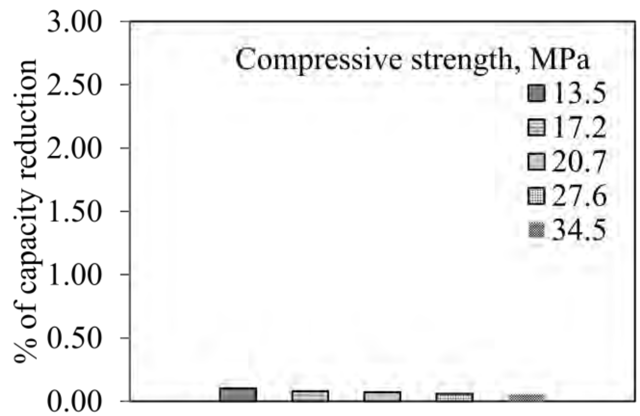
(e)


Compressive strength, MPa
13.5
17.2
20.7
27.6
34.5
% of capacity reduction
0.00
0.50
1.00
1.50
2.00
2.50
3.00

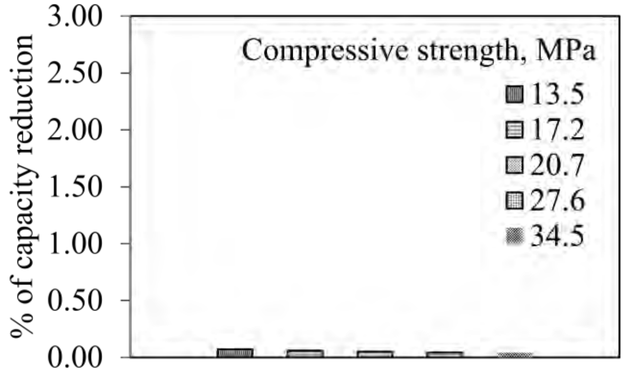
(f)


Compressive strength, MPa
13.5
17.2
20.7
27.6
34.5
% of capacity reduction
0.00
0.50
1.00
1.50
2.00
2.50
3.00

(g)


Compressive strength, MPa
13.5
17.2
20.7
27.6
34.5
% of capacity reduction
0.00
0.50
1.00
1.50
2.00
2.50
3.00

(h)



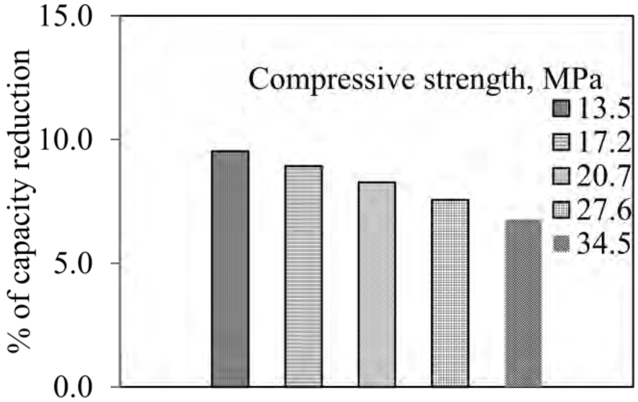
(i)

Figure 6-4 Effect of concrete strength on core cutting of columns in case core dimensions of 50mm x 100mm and tie bar sapcing of Category-II(T2). (a) 300mm x 300mm, (b)300mm x 375mm, (c)300mm x 450mm, (d)375mm x 375mm, (b)300mm x 525mm (f)450mm x 450mm (g)450mm x 525mm (h) 525mm x 525mm & (i) 600mm x 600mm.

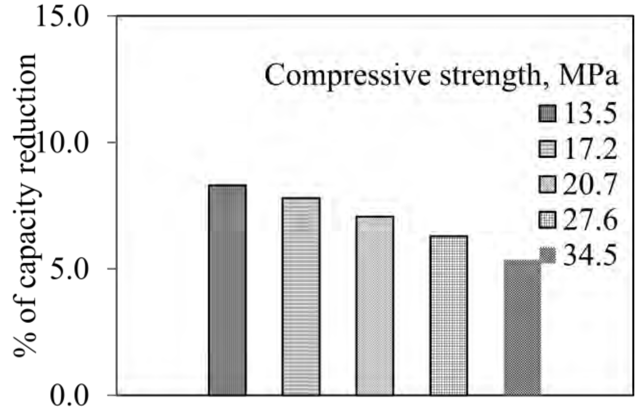
T

strengths as shown in Figure 6-5(a) through 6-5(i). For 300mm x 300mmcolumn size as shown in Figure 6-5(a), the capacity reduction has been found to be 9.54%, 8.94%, 8.28%, 7.57% and 6.77% for concrete strengths of 13.5, 17.2, 20.7, 27.6, and 34.5 MPa, respectively. As the concrete strength has been increased from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, the capacity reduction has been decreased by 6.24%, 7.398%, 8.60% and 10.48%, respectively. A similar pattern has been obtained for 150mmx375mm columns in Figure 6-5(b) with corresponding capacity reductions of 8.32%, 7.80%, 7.07%, 6.30% and 5.35% for five different concrete strengths. As the concrete strength has been increased from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, the capacity reduction has been decreased by 6.19%, 9.39%, 10.89% and 15.01%, respectively. Similarly, for 150mmx450mm columns in Figure 6-5(c), capacity reductions of 7.27%, 6.63%, 5.71%, 4.85% and 4.12% have been obtained for the five different types of concrete strength. Percentage decreases in capacity reduction by 8.77%, 13.93%, 14.98% and 15.22% have been obtained for an increase in concrete strength from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, respectively. In case of square columns of 375mmx375mm size as shown in Figure 6-5(d), it has been noticed that the capacity reduction of core drilled columns has been found to be 3.91%, 3.65%, 3.29%, 2.82% and 2.32% for concrete strengths of 13.5, 17.2, 20.7, 27.6, and 34.5 MPa, respectively. As the concrete strength has been increased from 13.5 to 17.2 MPa, 17.2 to 20.7

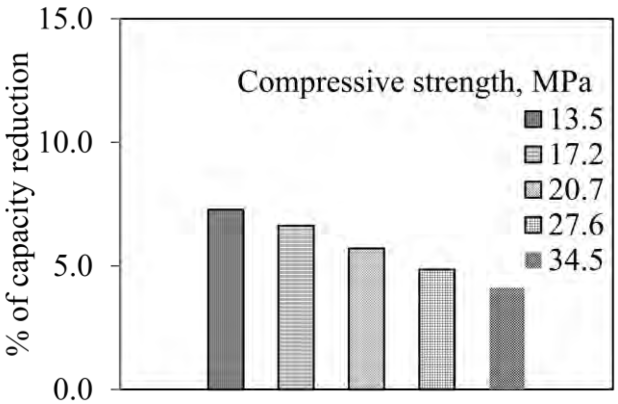
MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, the capacity reduction decreased by 6.68%, 10.01%, 14.11% and 17.98% respectively. Similar effect has been noticed for other higher sizes of columns such as 150mm x 525mm, 375mm x 450mm, 450mm x 450mm, 525mm x 525mm as shown in Figure 6-5(e) through 6-5(i).



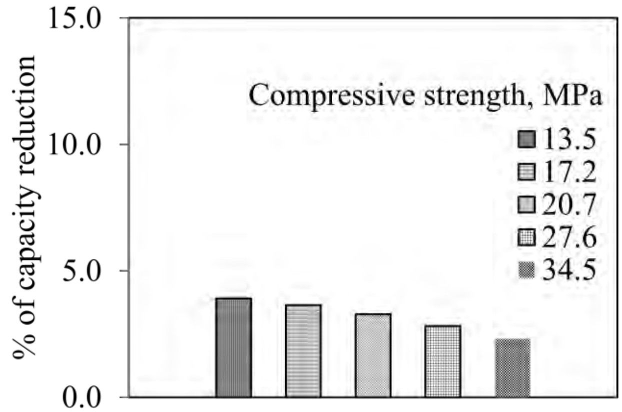
(a)



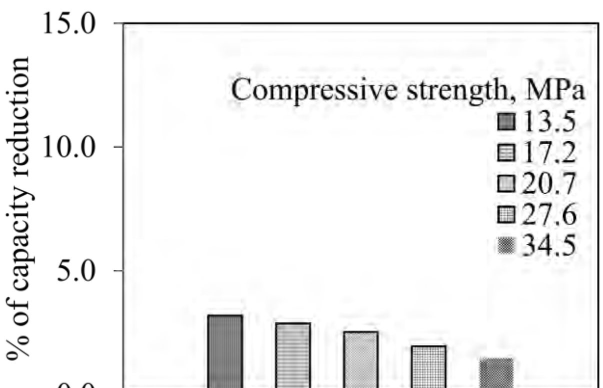
(b)



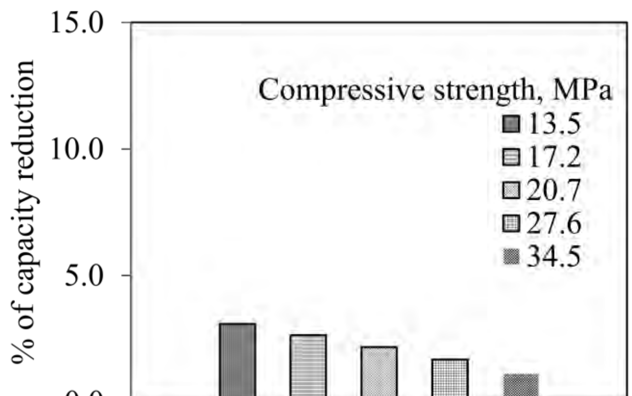
(c)



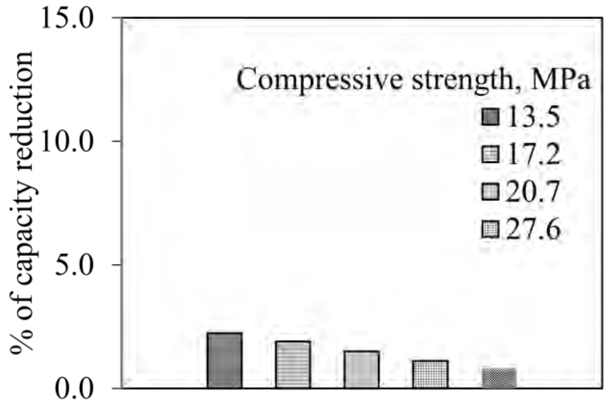
(d)



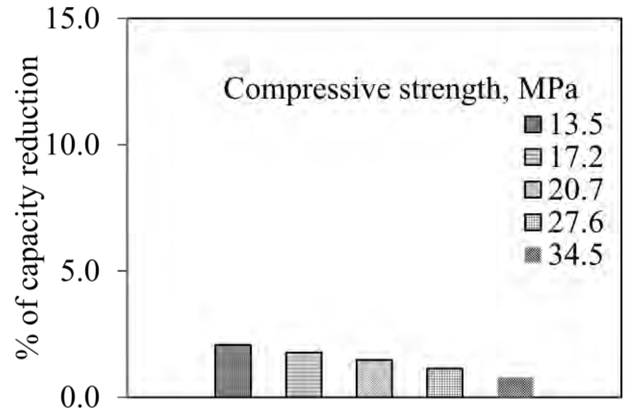
(e)



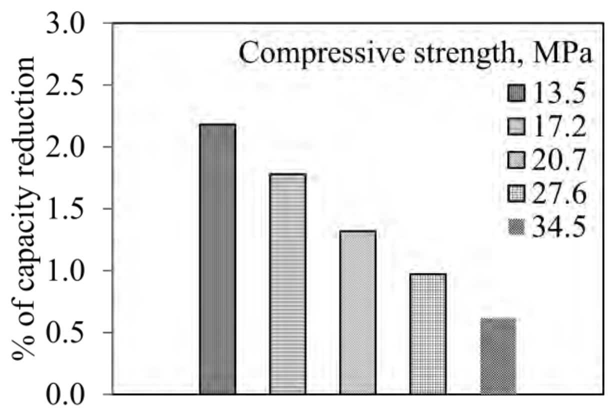
(f)



(g)



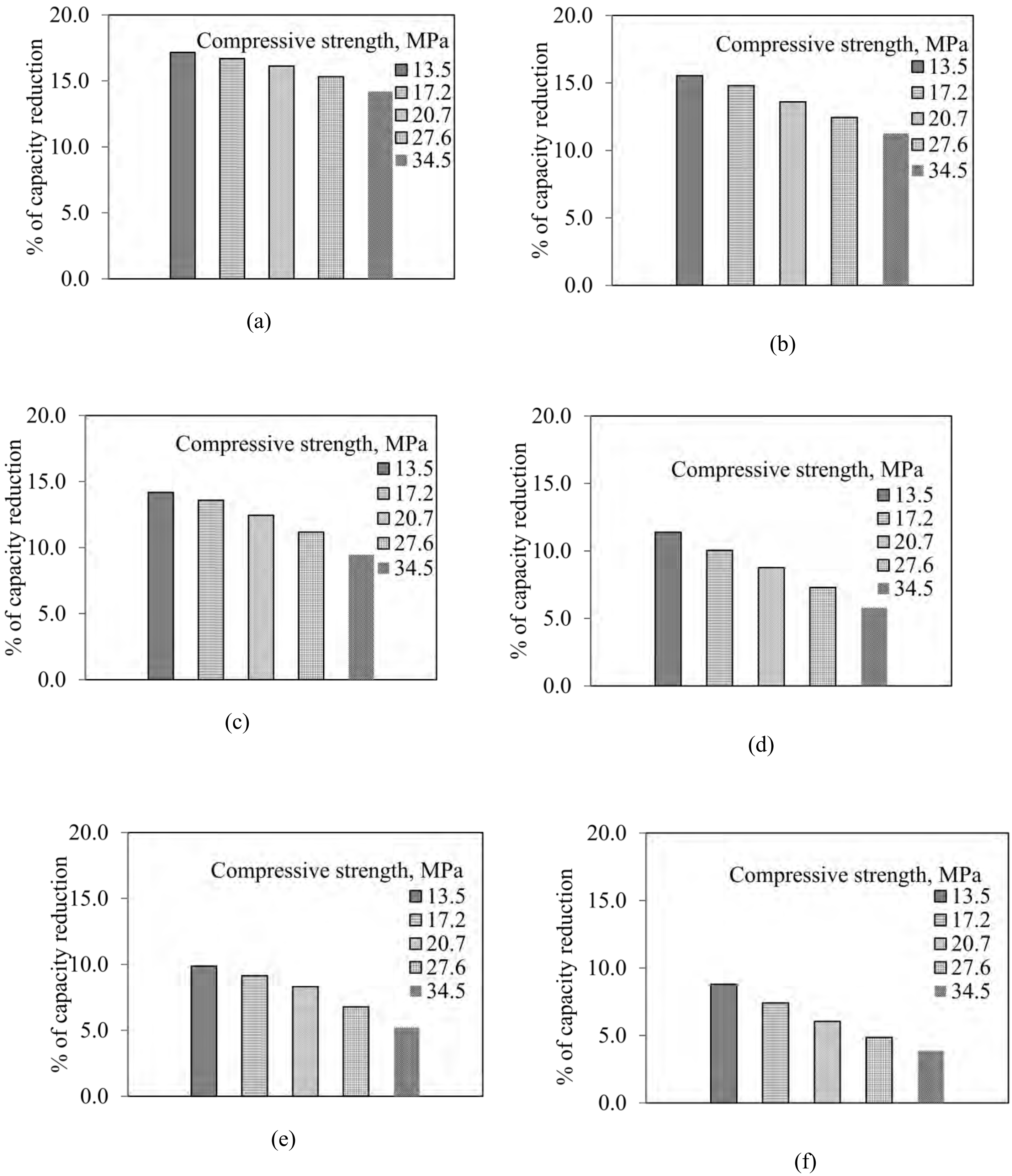
(h)



(i)

Figure 6-5 Effect of concrete strength on core cutting of columns in case core dimensions of 75mm x 150mm and tie bar sapcing of Category-II(T2). (a) 300mm x 300mm, (b)300mm x 375mm, (c)300mm x 450mm, (d)375mm x 375mm, (b)300mm x 525mm (f)450mm x 450mm (g)450mm x 525mm (h) 525mm x 525mm & (i) 600mm x 600mm.

A similar FEA analysis has been performed for 100mm core diameter for the same range of concrete strengths and the effect has been obtained to be quite similar as discussed above for core diameters of 50mm and 75mm cores as being illustrated in Figure 6-6(a) to 6-6(i). In case of 300mm x 300mmcolumns in Figure 6-6(a), there have been capacity reductions of 17.17%, 16.71%, 16.14%, 15.32% and 14.19% corresponding to concrete strengths of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa, respectively. As the concrete strength has been increased from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, there has been a significant decrease in capacity reduction of 2.68%, 3.41%, 5.08% and 7.32%, respectively. Similarly, for 150mmx375mm columns in Figure 6-6(b), capacity reductions of 15.54%, 14.80%, 13.60%, 12.44% and 11.27% have been obtained for concrete strengths of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa, respectively. With an increase in concrete strength from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, the capacity reduction has been decreased as 4.73%, 8.07%, 8.53% and 9.45% respectively. A similar pattern has been obtained for 150mmx450mm columns in Figure 6-6(c) with corresponding capacity reductions of 14.18%, 13.59%, 12.45%, 11.17 and 9.47% for concrete strengths of 13.5, 17.2, 20.7, 27.6 and 34.5 MPa, respectively. The decrease in capacity reduction obtained in this case has been 4.15%, 8.43%, 10.28% and 15.22% for a progressive increase in concrete strength from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, respectively. Square columns of 375mm x 375mm, as shown in Figure 6-6(d), represent similar effect as compared to rectangular columns. In case of square columns as illustrated in Figure 6-6(d), the capacity reduction of core drilled column has been found to be 11.39%, 10.05%, 8.77%, 7.29% and 5.79%, respectively for the range of concrete strengths involved. The decreased capacity reductions have been found to be 11.78%, 12.72%, 16.80% and 20.62% corresponding to increase in concrete strength from 13.5 to 17.2 MPa, 17.2 to 20.7 MPa, 20.7 to 27.6 MPa & 27.6 to 34.5 MPa, respectively. It has been noticed that the capacity reduction is always smaller for columns having greater concrete strength. The concrete strength effect has been quite similar for other higher sizes of columns as shown in Figure 6-6(e) to 6-6(i).



% of capacity reduction
Compressive strength, MPa
13.5
17.2
20.7
27.6
34.5
0.0
5.0
10.0
15.0
20.0
(a)
(b)
(c)
(d)
(e)
(f)

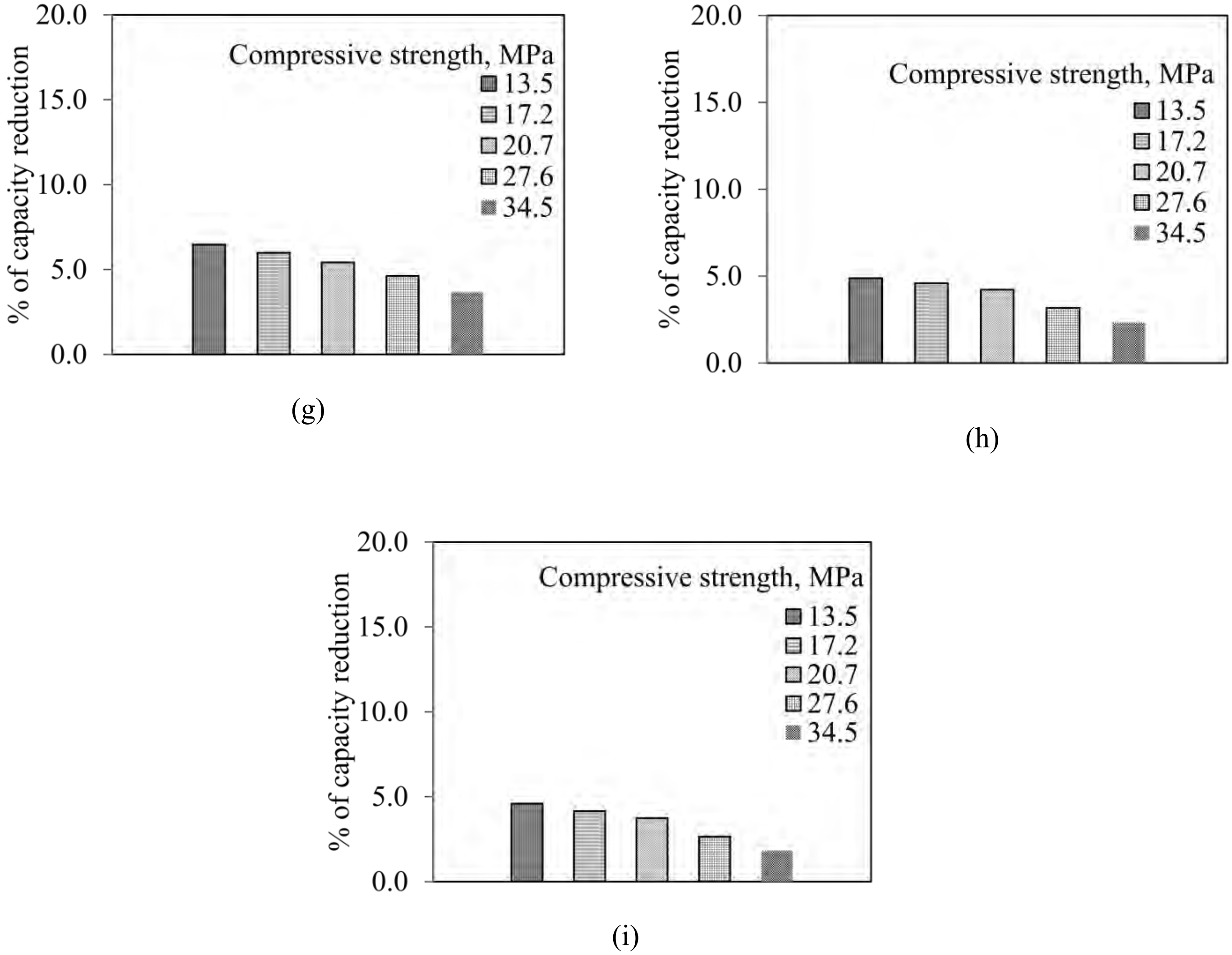
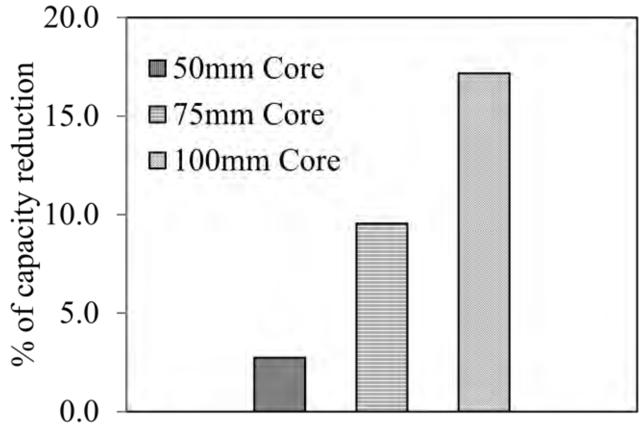


Figure 6-6 Effect of concrete strength on core cutting of columns in case core dimensions of 100mm x 200mm and tie bar sapcing of Category-II(T2). (a) 300mm x 300mm, (b)300mm x 375mm, (c)300mm x 450mm, (d)375mm x 375mm, (b)300mm x 525mm (f)450mm x 450mm (g)450mm x 525mm (h) 525mm x 525mm & (i) 600mm x 600mm.

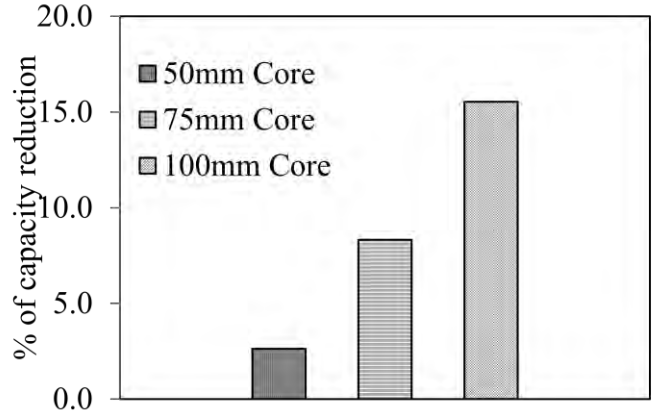
6.4 Effect of Core Size

The objective of this study has been to evaluate the effect of core cutting on column capacity. An extraction of core from concrete column significantly affects the column capacity as discussed in Chapter 4. According to the field study it has been observed three different sizes of core such as 50mm, 75mm and 100mm is being prevalent in the construction practice to evaluate the concrete strength. Hence, three different types of cores of 50mm, 75mm and 100mm diameter have been analyzed in FEA simulation with different varying concrete parameters. From FEA simulations for 50mmdrilled cores, it has been noticed from Figure 6-7(a) to 6-7(i) that core size has a significant effect on capacity reduction for a certain concrete strength of 13.5 MPa concrete as compared to NC columns without cores. In Figure 6-7(a), three different sizes core (50mm, 75mm 100mm) have been drilled out from 150mm x 150mm columns and the capacity reductions have been found as 2.72%, 9.54% and 17.17%, respectively. With an increase in core size from 50mm to 75mm and 75mm to 100mm, there has been a significant increase in capacity reduction of 250% and 80%, respectively. In case of column 150mm x 375mm columns in Figure 6-7(b), the capacity reduction of column has been found to be 2.63%, 8.32% and 15.54% for core sizes of 50mm, 75mm and 100mm, respectively. With an increase in core size from 50mm to 75mm and 75mm to 100mm, the capacity reduction has been increased by 216.66% and 86.84%, respectively.

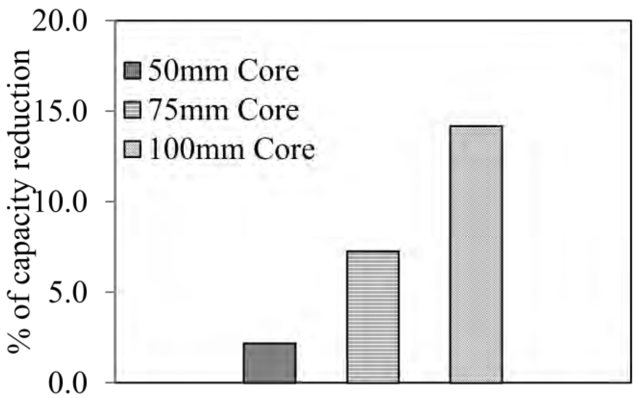
A similar pattern has been obtained for 150mm x 450mm columns in Figure 6-7(c) with corresponding capacity reductions of 2.18%, 7.27% and 14.18% for different core sizes of 50mm. 75mm, 100mm, respectively. The increase in capacity reduction obtained in this case has been 233.33% and 95.0% for an increase in core size from 50mm to 75mm, 75mm to 100mm, respectively. For other higher sizes of column from Figure 6-7(d) through 6-7(i), it has been noticed that the capacity reduction due to core drilling has also increased with an increasing core size. In case of square columns of 375mm x 375mm as shown in Figure 6-7(d), it has been observed that the capacity reduction of core drilled column has been 1.96%, 3.91% and 11.39% for 50mm, 75mm 100mm core, respectively. The capacity reduction has elevated by 100.0% and 190.91% for an increase of core size from 50mm to 75mm, 75mm to 100mm, respectively in this case. The core size effect is quite similar for other higher sizes of column as shown in Figure 6-7(e) to 6-7(i).


20.0
15.0
10.0
5.0
0.0
% of capacity reduction
50mm Core
75mm Core
100mm Core

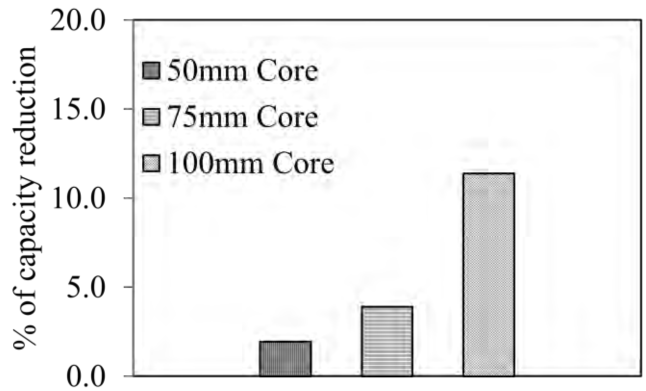
(a)


20.0
15.0
10.0
5.0
0.0
% of capacity reduction
50mm Core
75mm Core
100mm Core

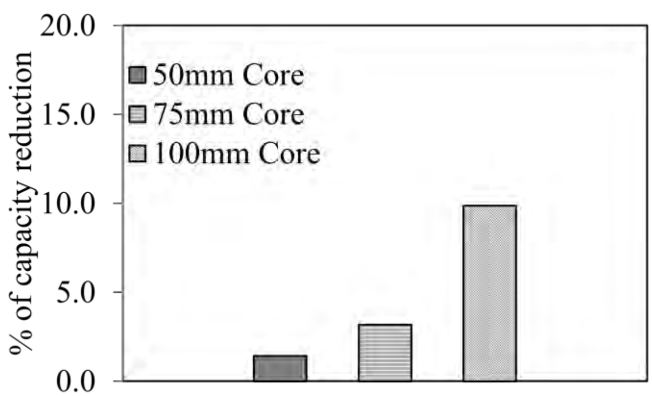
(b)


20.0
15.0
10.0
5.0
0.0
% of capacity reduction
50mm Core
75mm Core
100mm Core

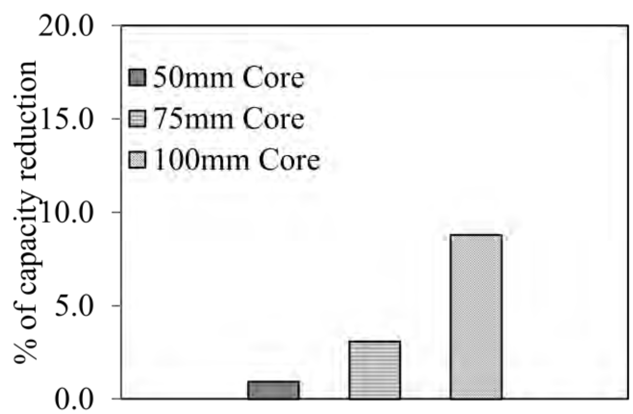
(c)


20.0
15.0
10.0
5.0
0.0
% of capacity reduction
50mm Core
75mm Core
100mm Core

(d)


20.0
15.0
10.0
5.0
0.0
% of capacity reduction
50mm Core
75mm Core
100mm Core

(e)


20.0
15.0
10.0
5.0
0.0
% of capacity reduction
50mm Core
75mm Core
100mm Core

(f)

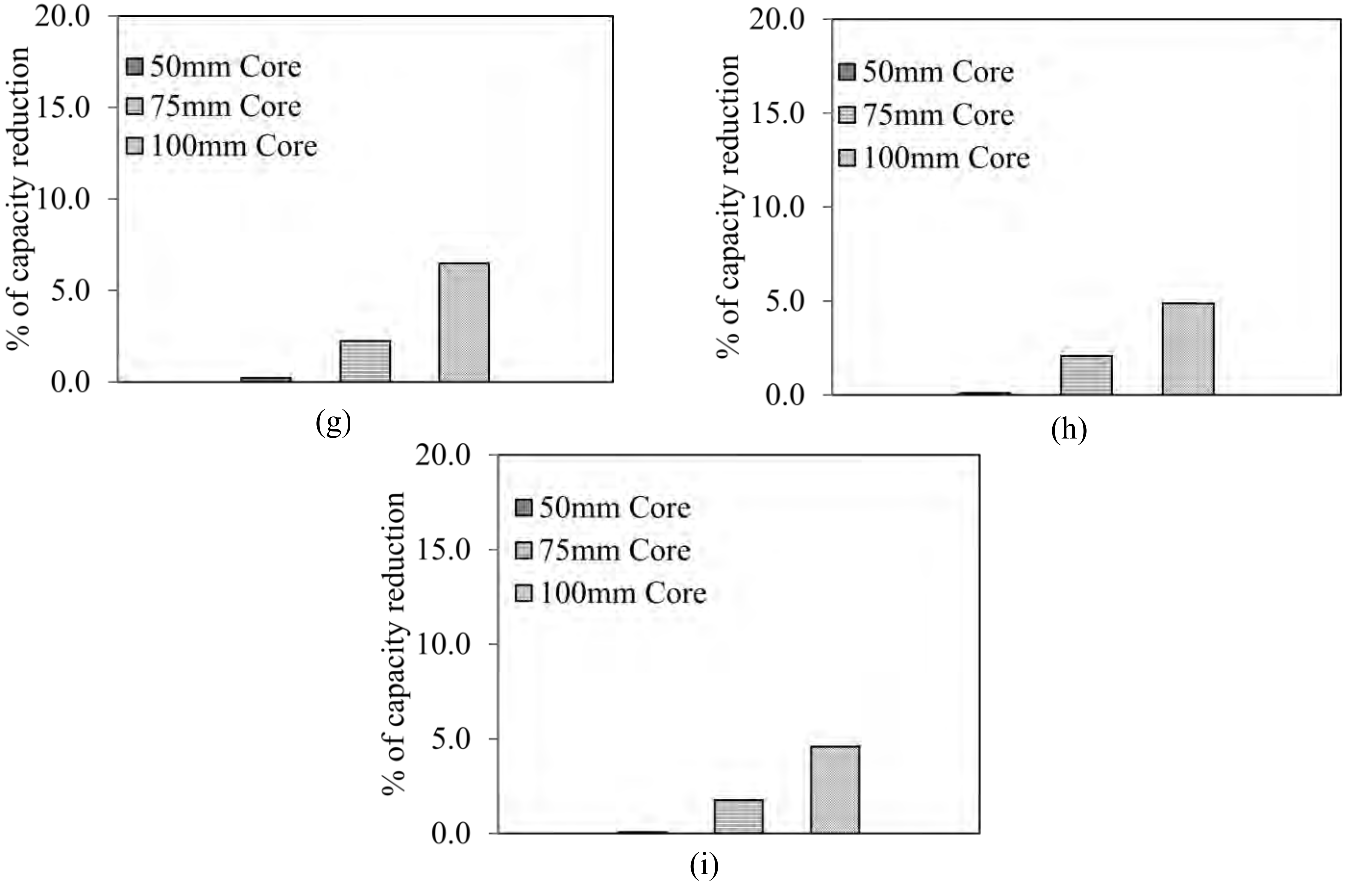


Figure 6-7 Effect of core diamateron percentage of capacity reduction of concrete columns for concrete strength of 13.5MPa and tie bar sapcing of Category-II(T2) (a) 300mm x 300mm, (b)300mm x 375mm, (c)300mm x 450mm, (d)375mm x 375mm, (b)300mm x 525mm (f)450mm x 450mm (g)450mm x 525mm (h) 525mm x 525mm & (i) 600mm x 600mm.

It has been observed in all cases that the core size has a pronounced effect on capacity reduction of columns. A similar behavior has been obtained from the simulation of both square and rectangular columns. An increase in core diameter from 50mm to 75mm corresponds to an increase in projected extraction area by 225% as compared to 177% for an increase in core diameter to 100mm from 75mm. Hence, it can be concluded that the effect of core diameter of 100mm has been most prominent due to the maximum area of concrete removed from a column cross-section. The FEA simulation has also been performed on different concrete strengths of 17.2, 20.7, 27.6 and 34.5 MPa and the core size effect has been obtained to be similar in all cases. The corresponding graphs for other different tie bar spacing have been illustrated in Figure 7-1 through Figure 7-15 of Chapter-7 and are also shown as a tabular form in APPENDIX Table 8-1 through Table 8-6.

6.5 Development of Graphical charts

One of the prime objectives of this study is to provide a general guideline and to establish a quantitative correlation among the contributing parameters that affect the capacity of core drilled columns. The developed correlations will assist practicing engineers to ensure safe core drilling from columns. Figure 6-9 shows a typical graphical representation of correlation among column size and core size for a concrete strength of 13.5 MPa and tie bar spacing of Category-I. It is obvious from the figure that the percentage reduction increases with the increase in core size from 50 mm to 100 mm. Also, the correlation is shown for different cross-sectional area of column, as shown at right of each line of the graph, and the lines for percentage capacity reduction are shifting downward i.e. higher cross sectional areas exhibits lower reduction. It has been noticed that the percentage reduction is more pronounced for 100 mm core compared to 75 mm core. Again 50 mm core shows lower percentage reduction compared to 75 mm core. As a result the percentage reduction line is steeper between 75 mm and 100 mm core as compared to 50 mm and 75 mm core. The similar analysis has been done for higher concrete strength of 17.2 MPa and tie bar spacing of category-I as shown in Figure 7-2. The result, obtained in this case, also exhibits similar pattern. For any arbitrary column size, the percentage of capacity reduction due to core extraction can be found for concrete strength of 20.7MPa and tie bar spacing of Category-I from Figure 6-11. Figures 6-12 and 6-13 represent percentage capacity reduction of columns for tie bar spacing of category-I and concrete strength of 27.6 MPa and 34.5 MPa, respectively. It is obvious from Figures 6-9 through 6-13 that the percentage reduction line is shifting downward with the increasing of column cross-sectional area. In all cases, it is noticed that 100mm core shows higher percentage capacity reduction compared to 75 mm core. A similar effect has also been obtained between 50 mm core and 75 mm core where 75 mm core shows greater reduction. For tie bar spacing of category-II, the percentage of capacity reduction can also be found for different concrete strength from Figure 6-14 to 6-18. The capacity reduction lines in this case are also shifting downward with the increasing of column cross-section area. But the overall percentage reduction is higher in this case as compared to tie bar spacing of Category-I as higher tie bar spacing results in higher amount of capacity reduction due to core extraction. When it is required to determine the capacity reduction of any arbitrary column sizes for concrete strength of 13.5 MPa and tie bar spacing of category-II, then Figure 6-14 can be used. Similarly, Figures 6-15 to 6.18 can be used for columns having higher concrete strength. The generalized graphs have also been developed for tie bar spacing of category-III and five different concrete strengths. All capacity reduction lines in this case are also similar in pattern. Among all three type of tie bar spacing, category-III always shows higher capacity reduction as compared to other tie bar spacing category. On the other hand, category-II always shows higher percentage of capacity reduction as compared to category-I. This phenomenon is accountable to the fact that smaller tie bar spacing increases confinement at core region which ultimately increases load carrying capacity of core drilled columns. Increased confinement assists in reducing stress concentration at the core region of columns. As a result, smaller tie bar spacing increases stiffness which results in higher core column capacity.

In order to validate the reliability of the graphical charts, a sample example is being shown in Table 6-1 and Figure 6.8.

Table 6-1 Sample Calculation

Given Data:

Concrete Strength= 20.7 MPa

Column Size= 250mm x 500mm

Core Size= 100mm x 150mm

Tie bar spacing= Category-II (T2)

Solution:

Percentage of Capacity

FEA= 6.88 %

From Graph= 7.03 %

It is found that the percentage of capacity reduction for a given concrete strength of 20.7 MPa obtained from the graph is quite similar to FEA simulation result. The percentage of variation of results has only been limited to about 2% with respect to FEA results. Hence, the high conformity between the results indicates the reliability of the developed graphical charts shown in Figures 6-9 to 6-23. In case of a core extraction, the graphs will aid an engineer to determine the safe core sizes based on parameters (concrete strength, tie spacing, column size) that can be obtained from drawings or site inspection. This is substantial particularly for low-strength columns which are usually more vulnerable to a drastic reduction in ultimate capacity due to core extraction.

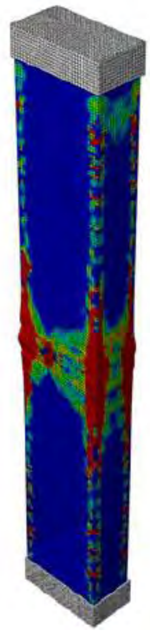


Figure 6-8 Column (250 mm x 500 mm) crack at ultimate capacity

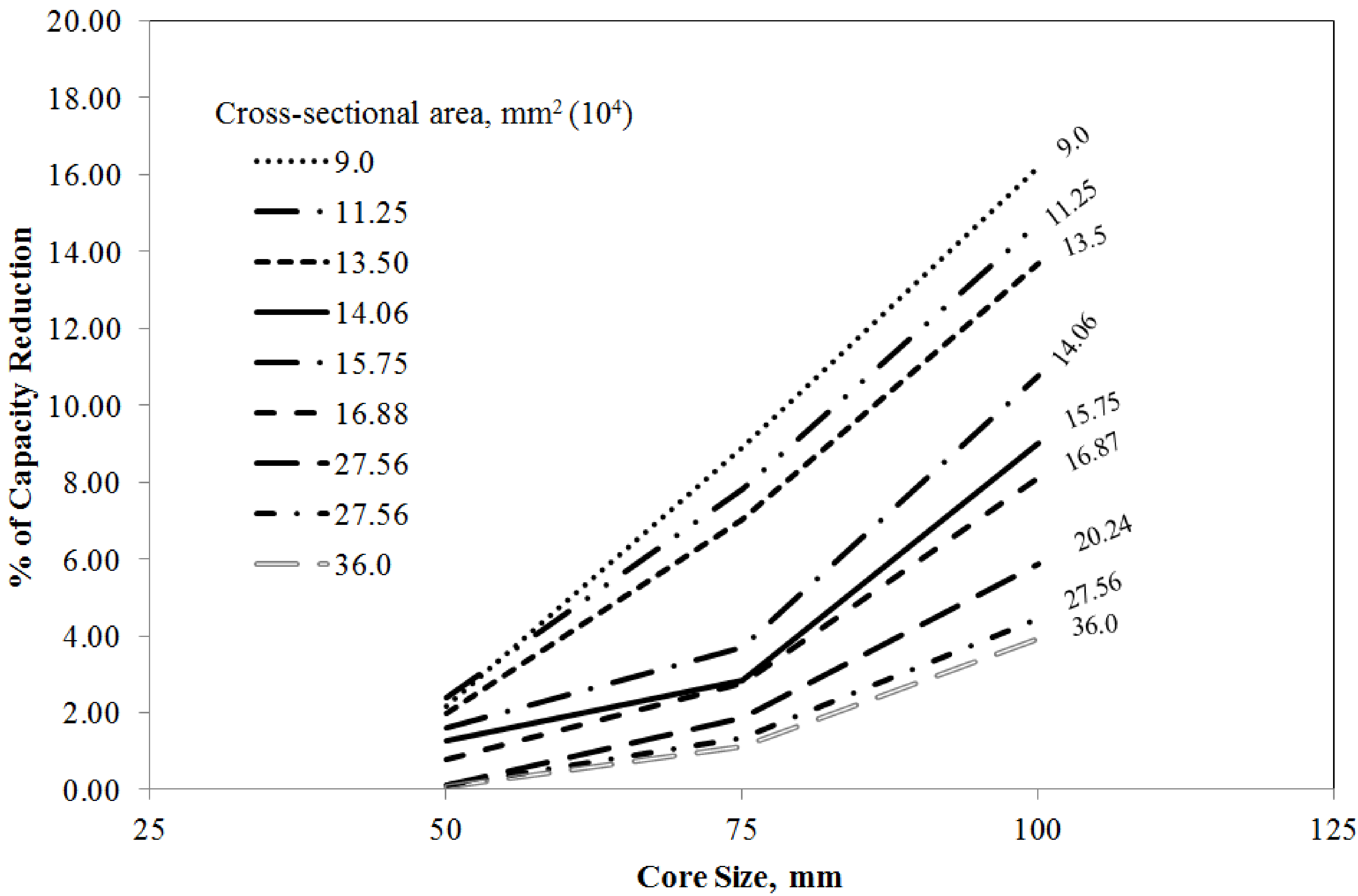


Figure 6-9 Core size Vs percentage of capacity reduction for 13.5 MPa concrete of tie bar spacing Category-I.

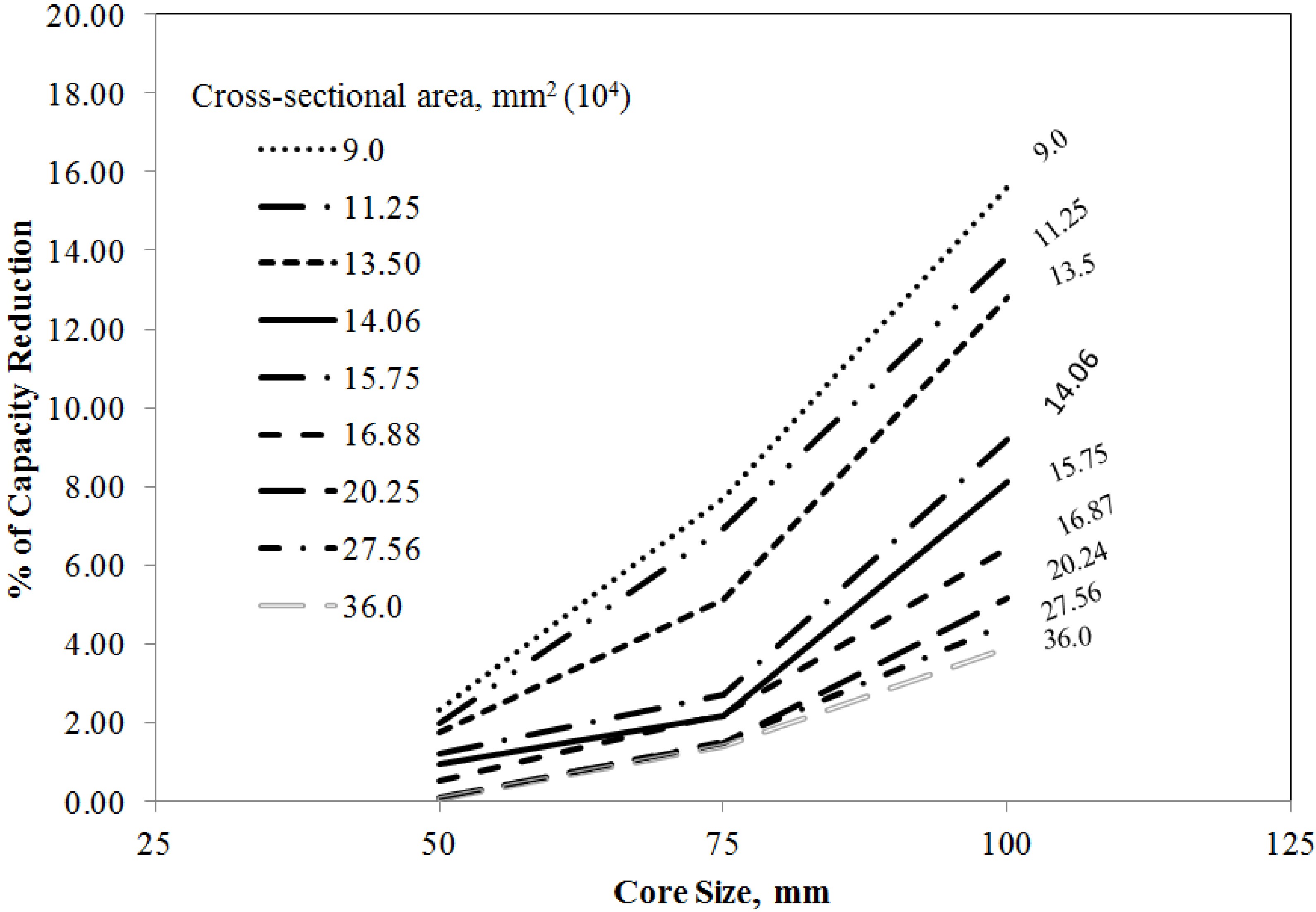


Figure 6-10 Core size Vs percentage of capacity reduction for 17.2 MPa concrete of tie bar spacing Category-I.

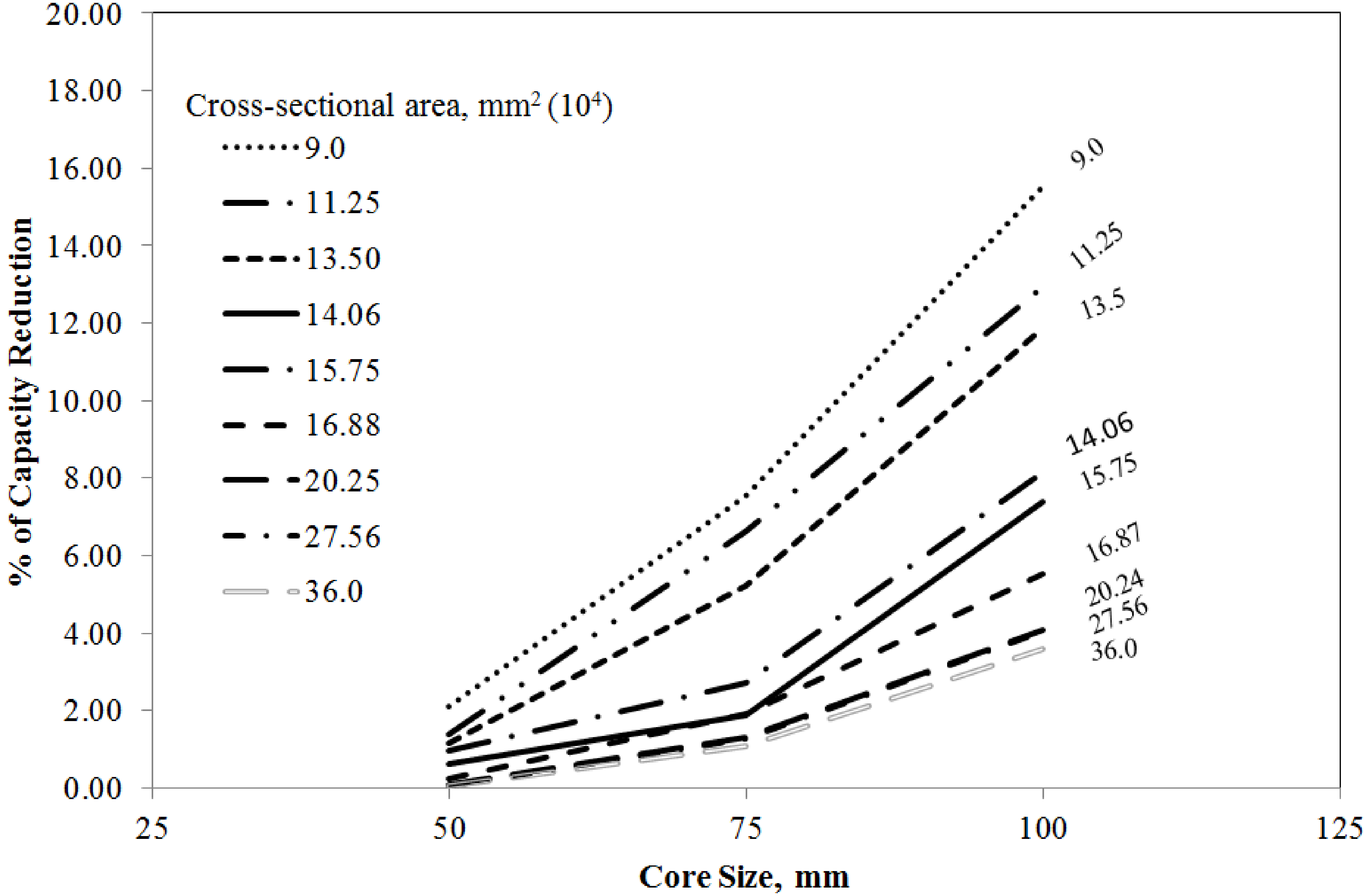


Figure 6-11 Core size Vs percentage of capacity reduction for 20.7 MPa concrete of tie bar spacing Category-I.

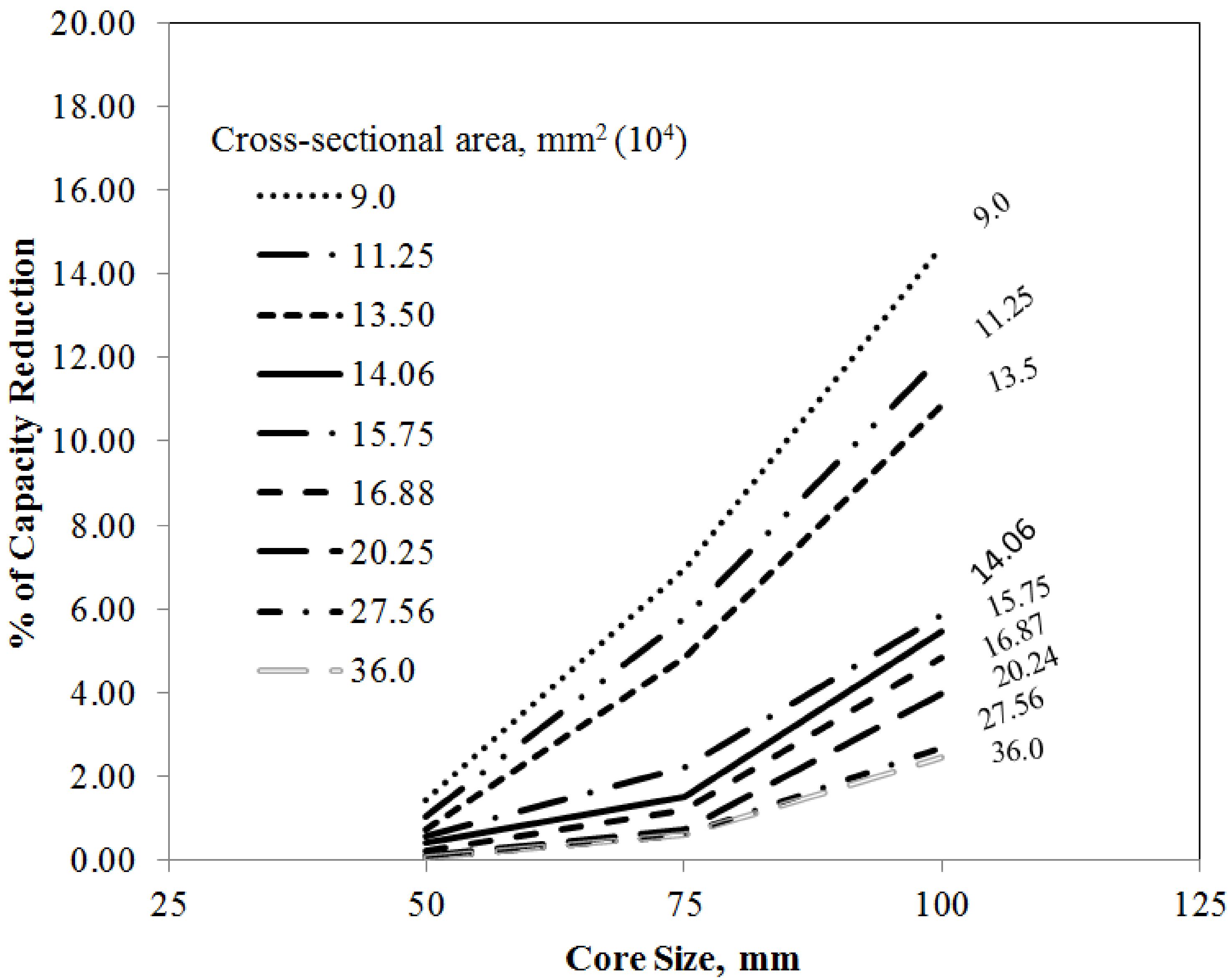


Figure 6-12 Core size Vs percentage of capacity reduction for 27.6 MPa concrete of tie bar spacing Category-I.

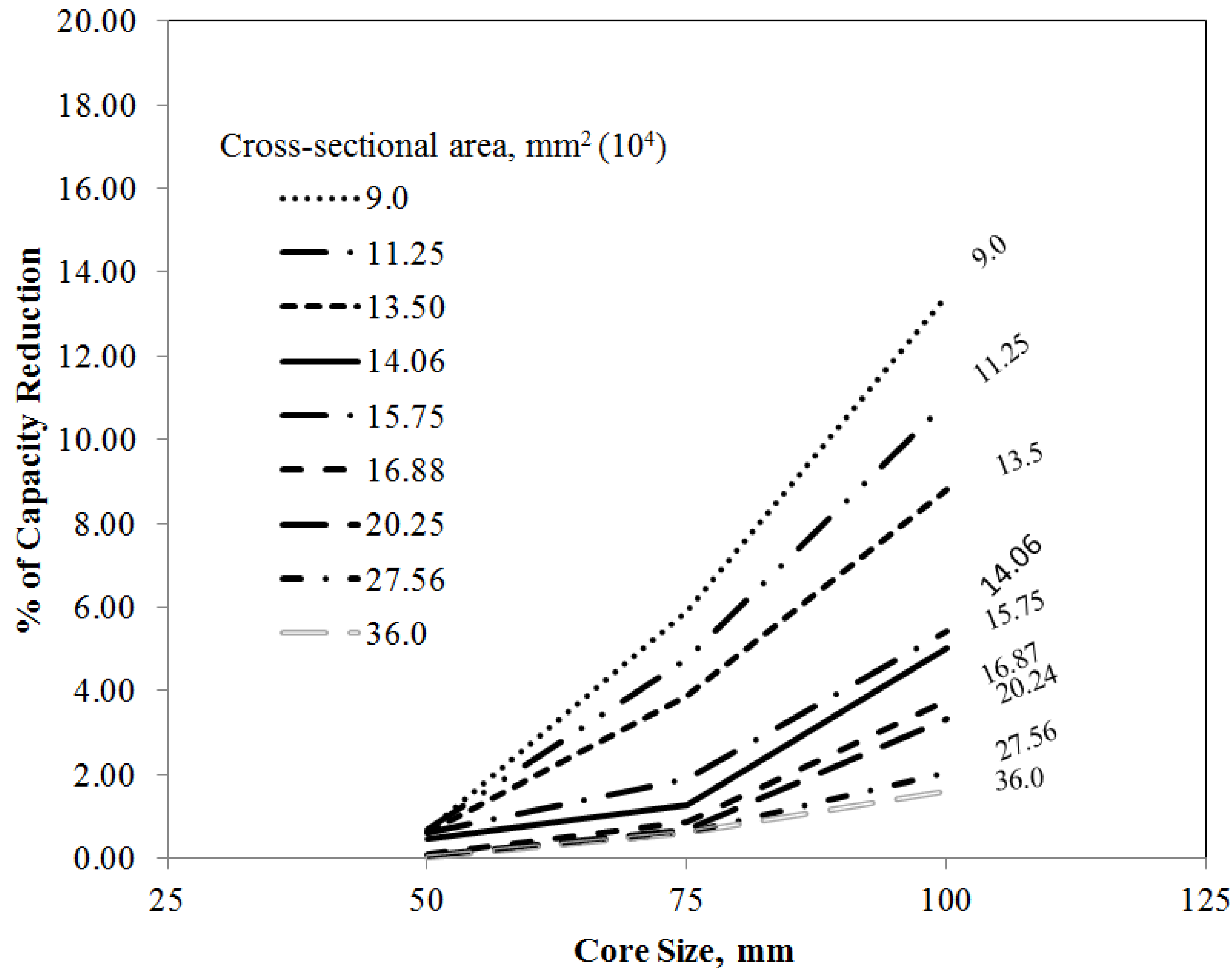


Figure 6-13 Core size Vs percentage of capacity reduction for 34.5 MPa concrete of tie bar spacing Category-I.

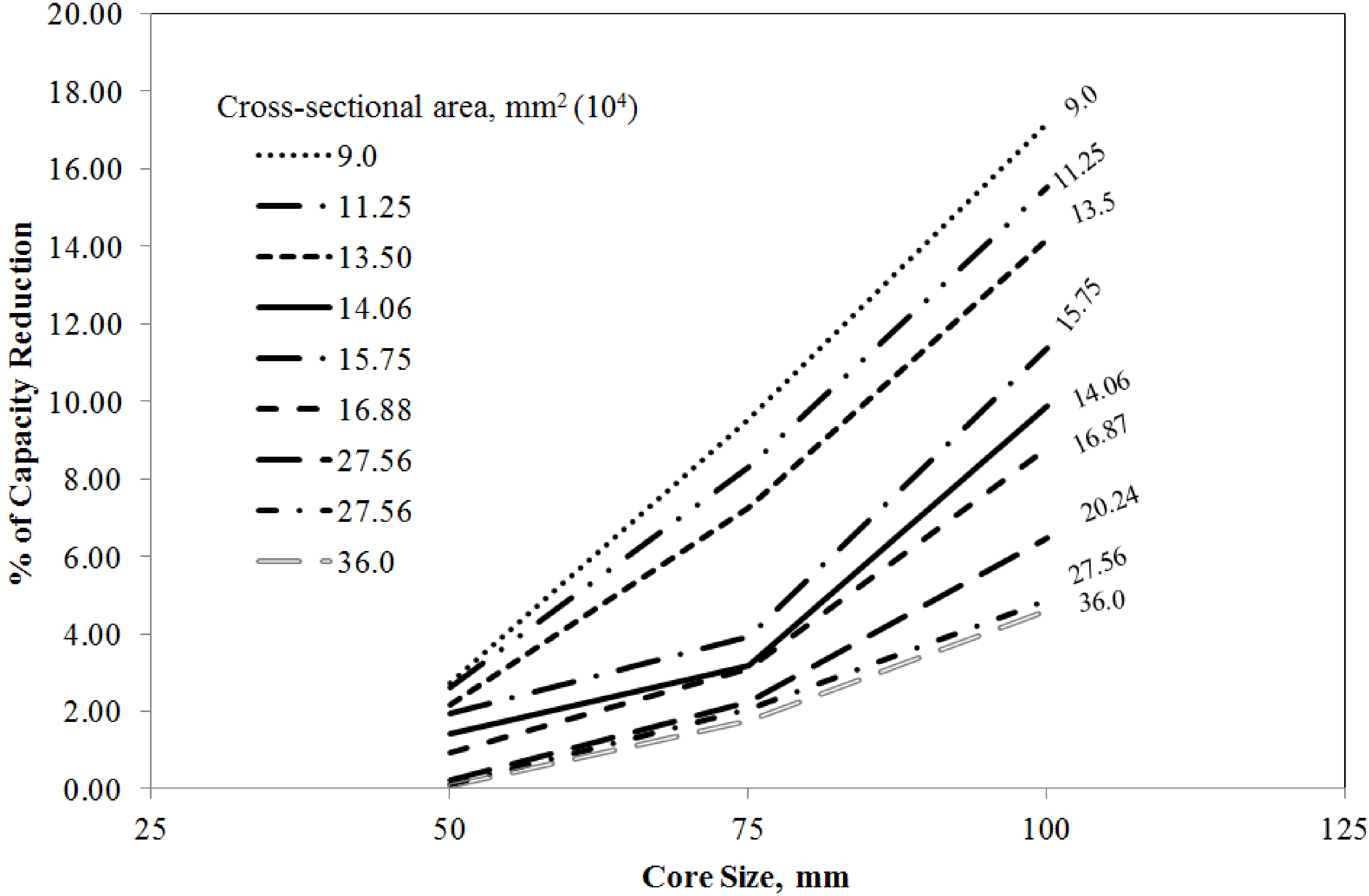


Figure 6-14 Core size Vs percentage of capacity reduction for 13.5 MPa concrete of tie bar spacing Category-II.

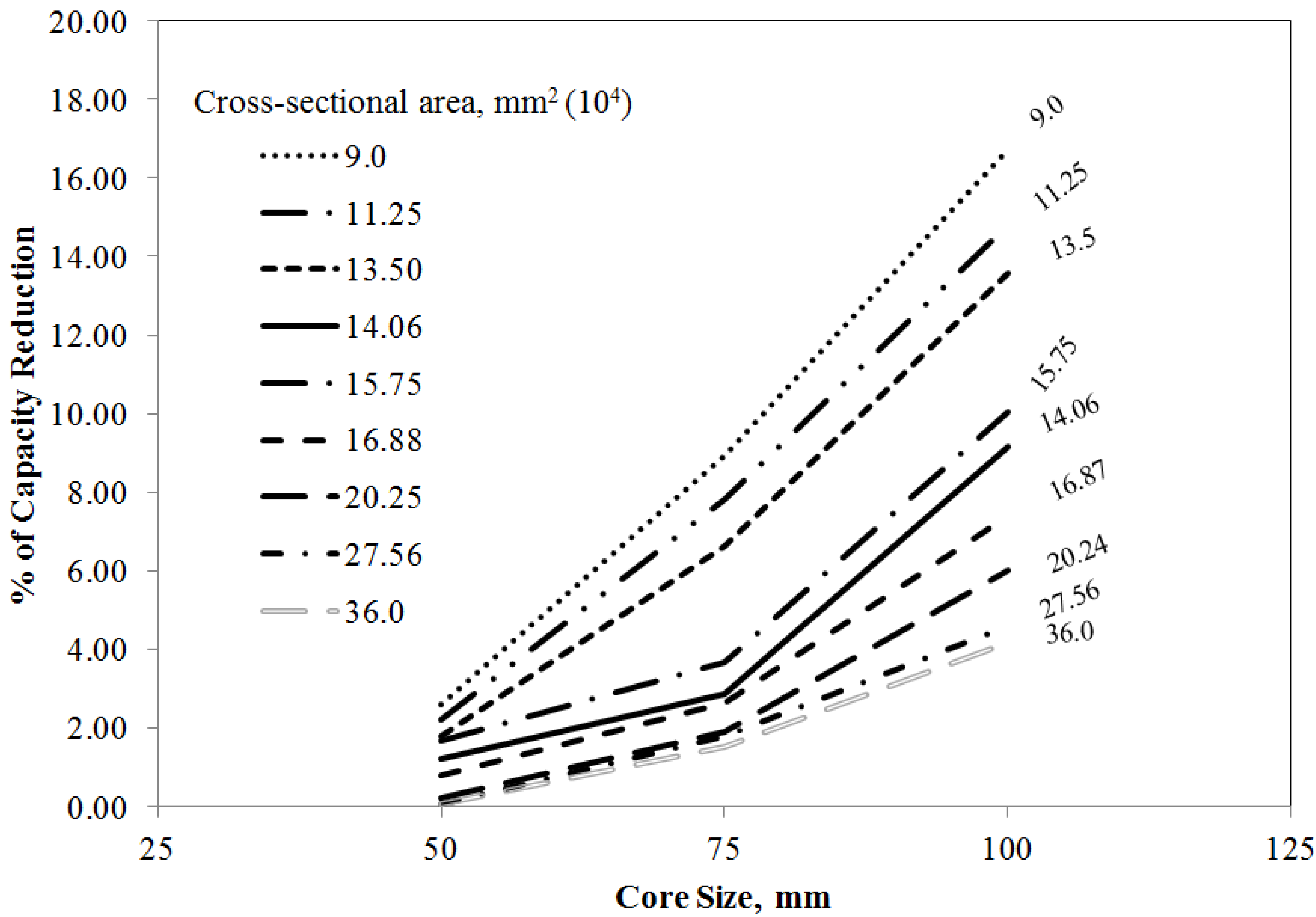


Figure 6-15 Core size Vs percentage of capacity reduction for 17.2 MPa concrete of tie bar spacing Category-II.

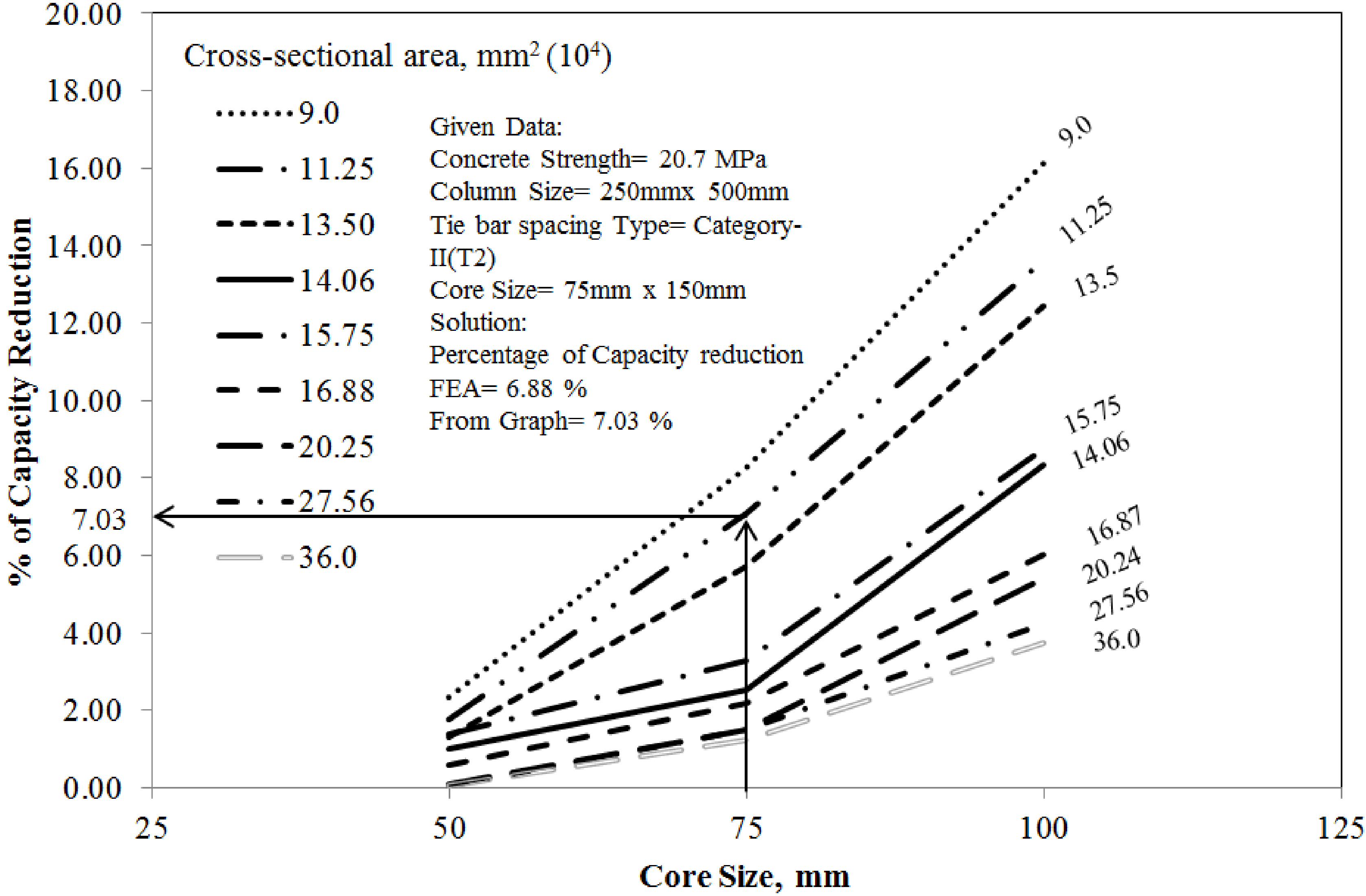


Figure 6-16 Core size Vs percentage of capacity reduction for 20.7 MPa concrete of tie bar spacing Category-II.

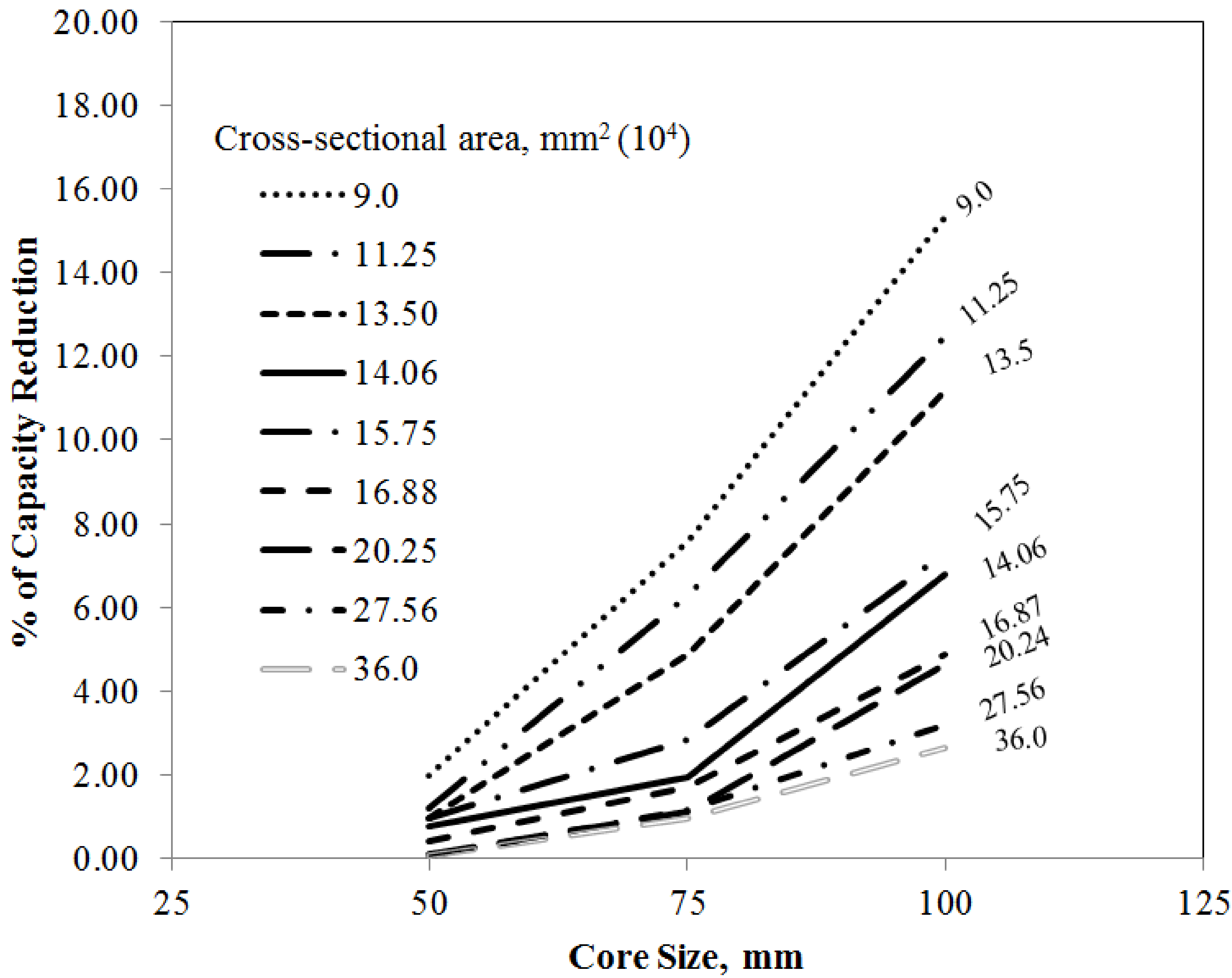


Figure 6-17 Core size Vs percentage of capacity reduction for 27.6 MPa concrete of tie bar spacing Category-II.

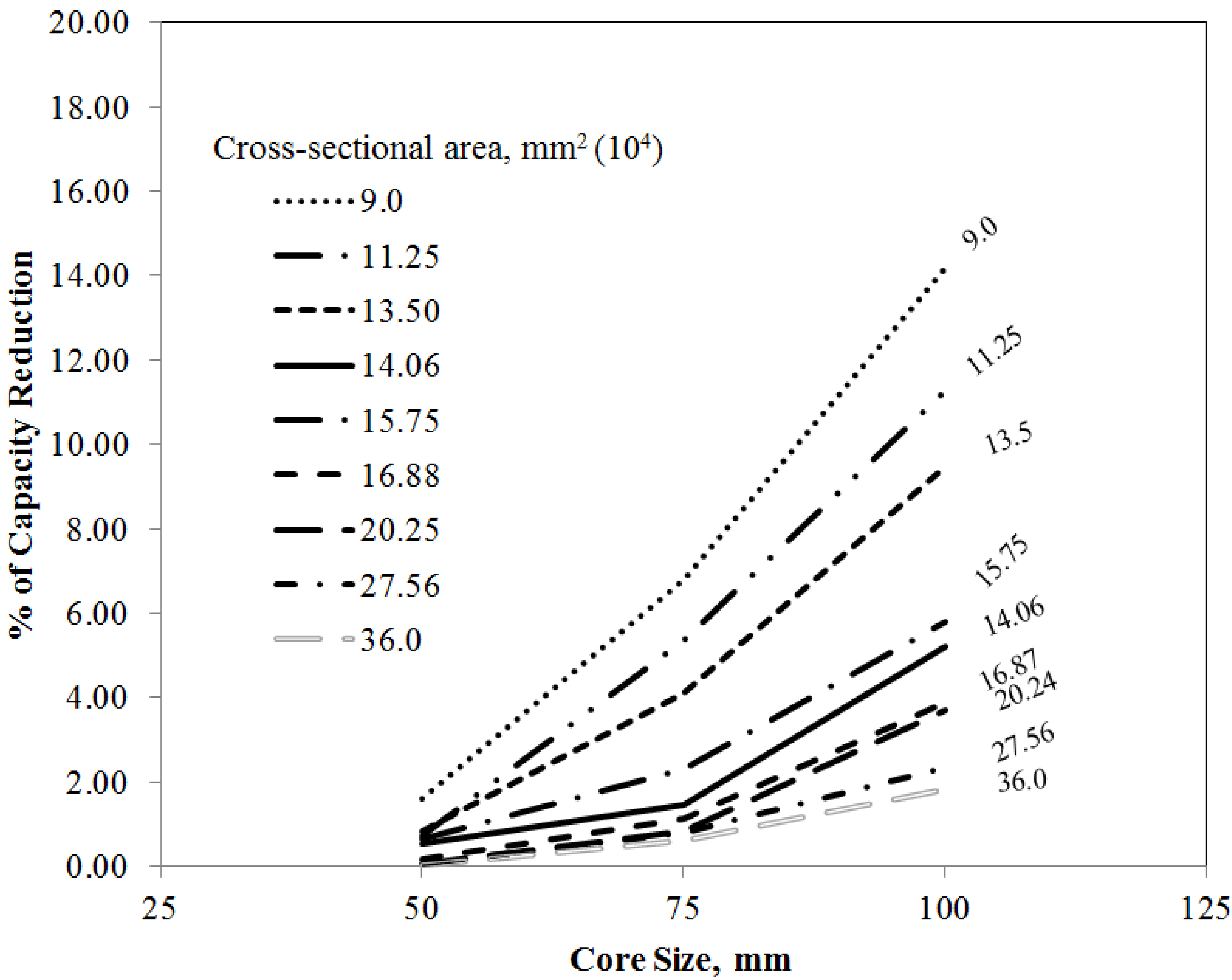


Figure 6-18 Core size Vs percentage of capacity reduction for 34.5 MPa concrete of tie bar spacing Category-II.

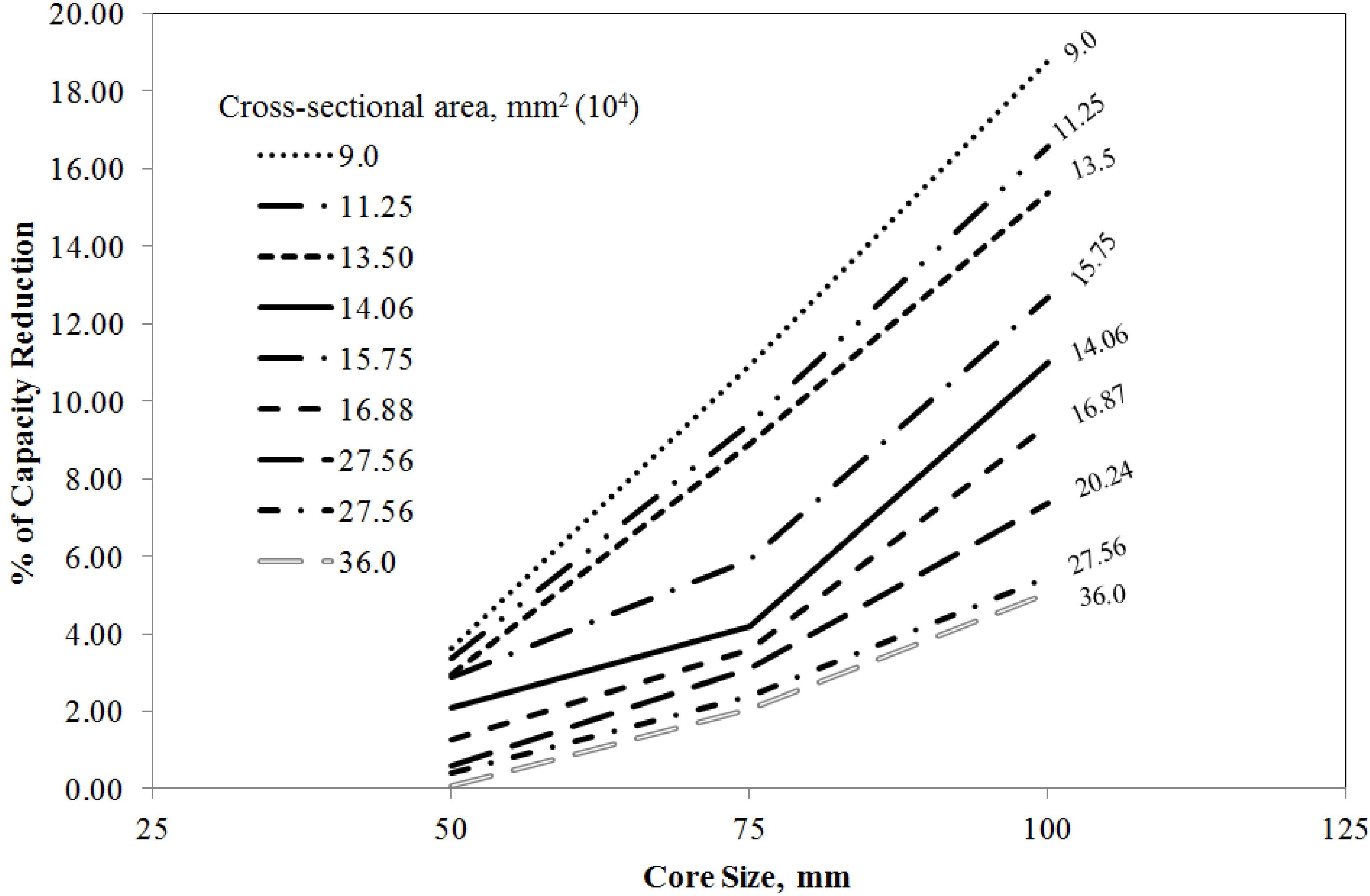


Figure 6-19 Core size Vs percentage of capacity reduction for 13.5 MPa concrete of tie bar spacing Category-III.

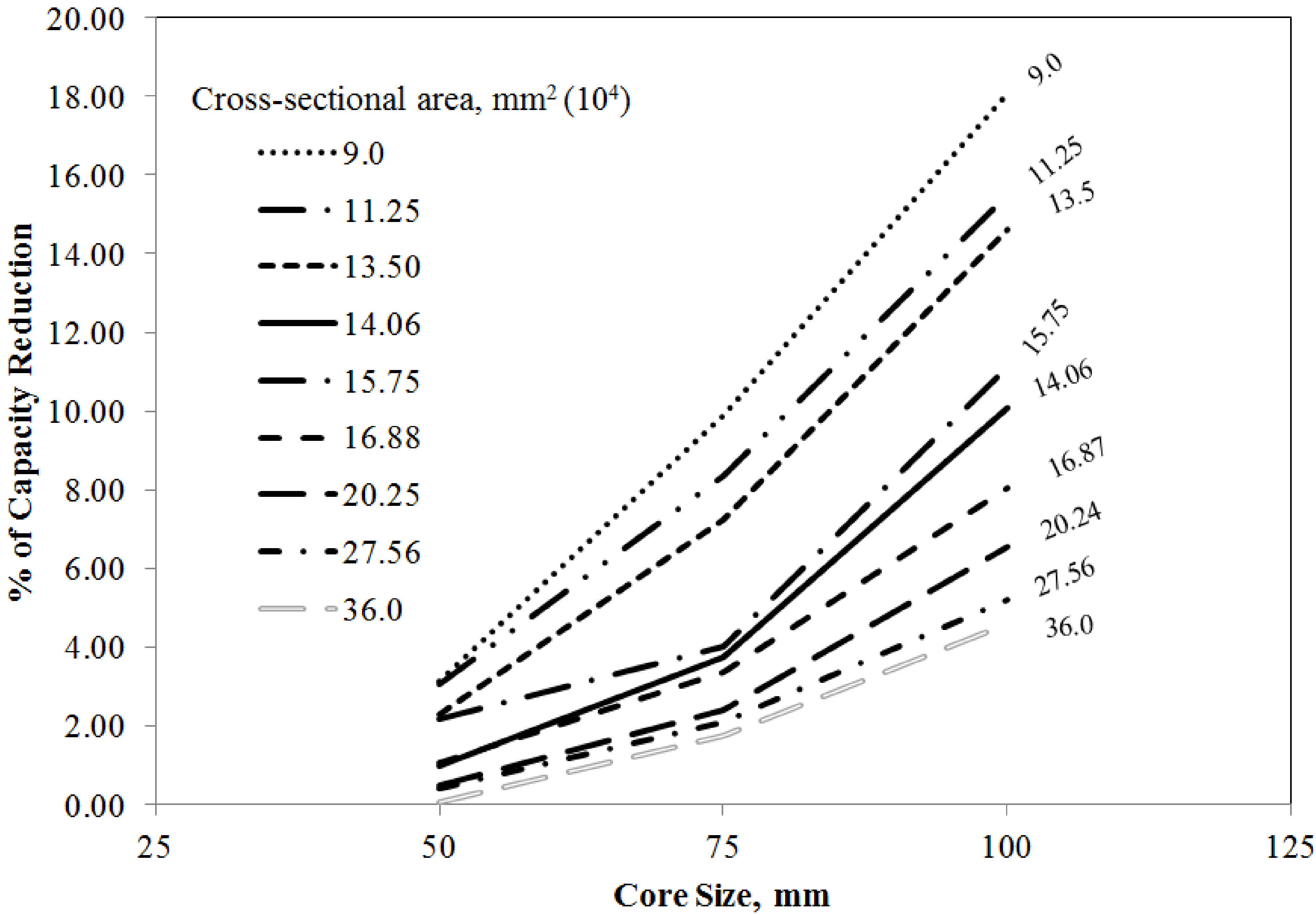


Figure 6-20 Core size Vs percentage of capacity reduction for 17.2 MPa concrete of tie bar spacing Category-III.

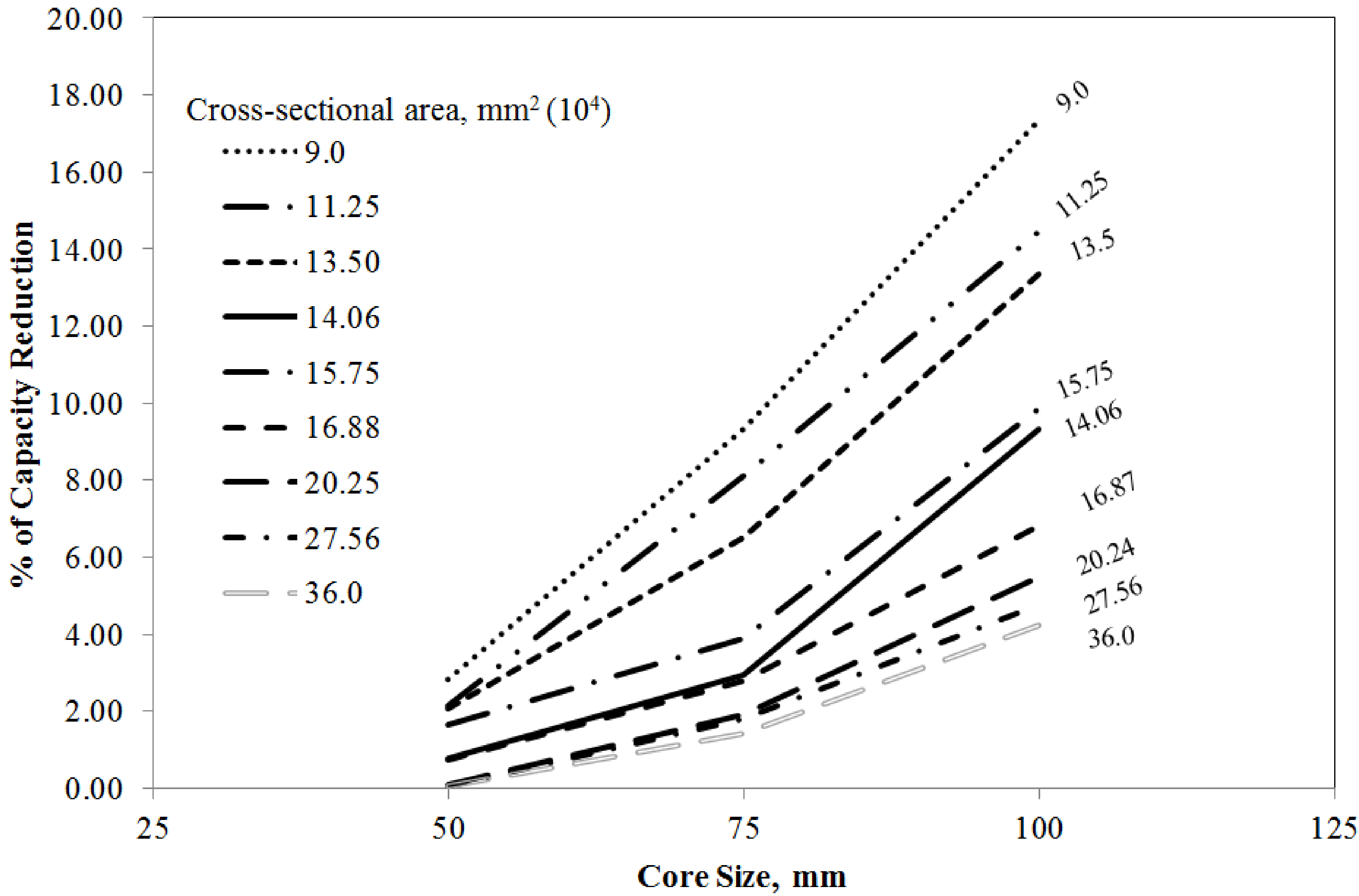


Figure 6-21 Core size Vs percentage of capacity reduction for 20.7 MPa concrete of tie bar spacing Category-III.

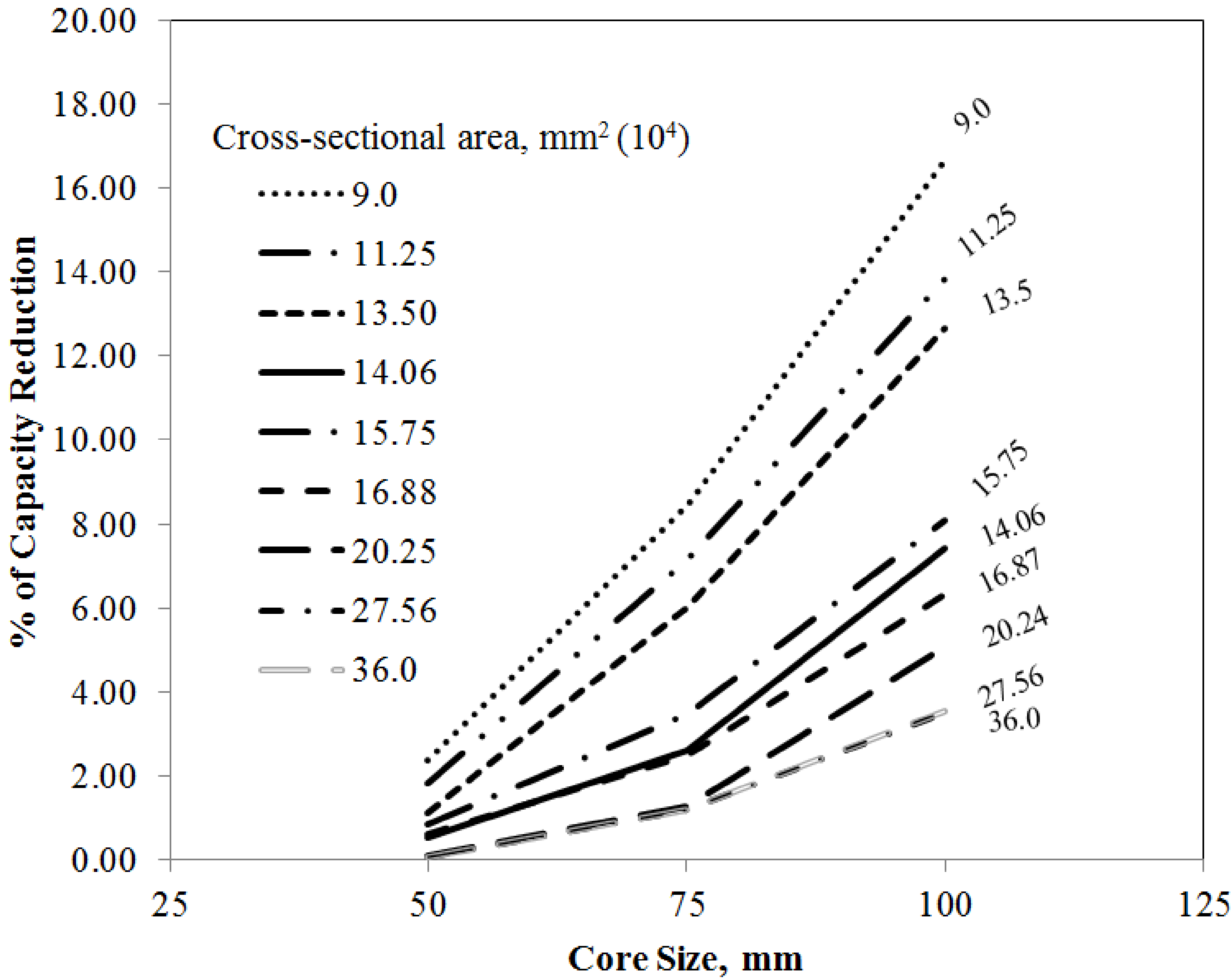


Figure 6-22 Core size Vs percentage of capacity reduction for 27.6 MPa concrete of tie bar spacing Category-III.

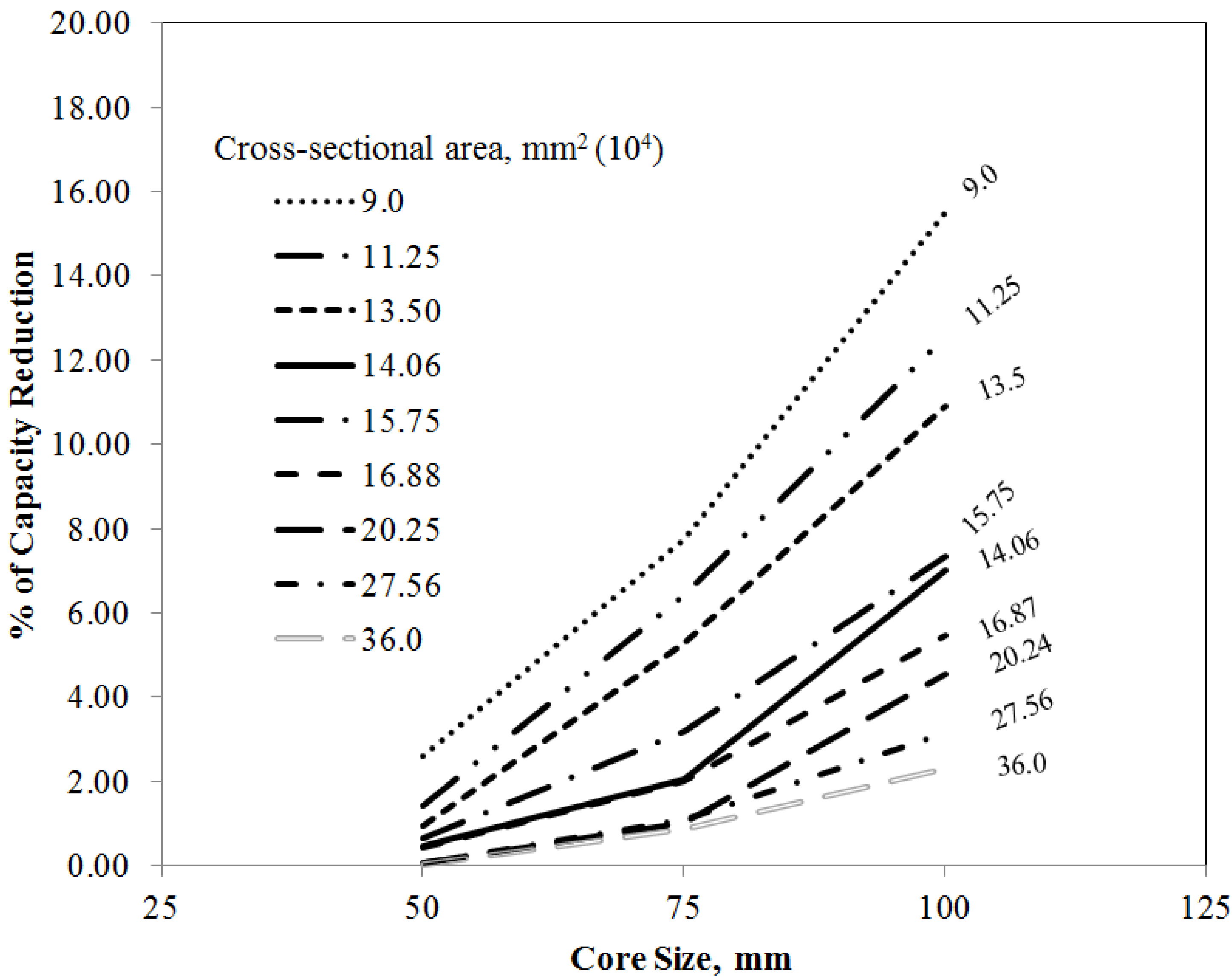


Figure 6-23 Core size Vs percentage of capacity reduction for 34.5 MPa concrete of tie bar spacing Category-III.

Chapter 7

CONCLUSION AND SUGGESTIONS

7.1 General

The significance of core drilling is optimal to determine the adequacy of in situ compressive strength of concrete. However, drilling out of core from a structural element results in capacity reduction which may lead a structure to an anticipated risk. In addition, the literature on the effect of load carrying capacity of column due to core drilling is not quite common. As a result, a detailed study aiming to develop a quantitative guideline to ensure safe core drilling procedure is of immense importance. Based on experimental and finite element analysis of each parameter on behavior of core drilled columns, a set of conclusions can be inferred.

7.2 Conclusion

i. From experimental results, it has been found that drilling out a core can significantly affect the axial capacity of a column depending on location of core along column length. Axial capacity of core drilled column can be reduced by an amount of more than 19% if core is cut in close proximity to the support. In the experimental analysis, it has been observed that columns with cores of 50 mm at mid height experienced a maximum of about 9% reduction in axial capacity than that of columns without any cores. Hence it is necessary to extract cores from mid height of a column in order to avoidhigher stress concentration at core location near support.

ii. It has been observed that the percentage of capacity reduction of CMD column has been found as 7.66% and 8.28% in FEA and experimental, respectively. On the other hand, for COD column, the percentage of capacity reduction was found as 18.39% and 19.24% in FEA and experimental analysis, respectively. It is evident that both experimental and finite element analysis yielded analogous ultimate capacity. In both cases it has been observed that FEA result shows lower value compared to experimental results. Moreover, similar crack pattern and load-displacement curves were observed for both cases. Therefore, it can be said that the assumptions of FEA method have been validated.

iii. From FEA, it has been observed that there is a linear reduction in axial capacity of core drilled columns with the increase in projected core area. Moreover, it has been found that axial capacity of smaller columns have been reduced significantly with the increase in projected core area in comparison with capacity reduction of larger size columns. It has also been observed that columns having dimension of 450 mm x 450 mm or more were relatively less affected by drilled cores than that of columns with cross-sectional dimension of 375 mm x 450mm or less.

iv. Column tie bar spacing has also some effect on the percentage of capacity reduction of core drilled column. For core size of 50mm diameter, the percentage capacity reduction varied between 10 to 23% and 33 to 46% for an increase in tie bar spacing fromType-1 to Type-2 and Type-2 to Type-3, respectively. Therefore, columns with higher lateral confinement will be less affected by core extraction.

v. In FEA simulation, it has been observed that concrete strength plays an important role on the capacity reduction of core drilled column. For concrete having lower compressive strength of 13.5 MPa, about 16% capacity reduction with 100mm diameter drilled core from column size of 300m mm x 300 mm was observed. On the other hand, about 4% reduction was obtained for higher strength concrete of 34.5 MPa for similar condition. Therefore, columns with lower strength are prone to higher risk due to core drilling and will require more caution.

vi. Restoration of core drilled column by different filler material showed different degree of capacity restoration. From experimental results, it is quite evident that restoration by non-shrinkage grout resulted in better performance than that of concrete lean mix. Hence, the non-shrinkage grout can be recommended to be used as filler material when available.

vii. On the basis of effect of different variables on the capacity of core drilled columns, several generalized graphs have been plotted as guidelines that represent the individual and combined effect of all the parameters on core extraction. The graphs cover almost all practical ranges of column dimension, concrete strength, lateral tie spacing and core size. The variables required to use the graphical charts can be found from design drawings or from field investigations. Utilization of charts will ensure confinement of capacity decline of a core drilled column within specified safety limits

7.3 Suggestions for Future Research

i. The experiment has been conducted for pure axial loading and no eccentricity was allowed in both experimental and FEA simulation. Other type of loading effect such as eccentric loading, lateral loading etc. can be considered to analyze the behavior of core drilled column.

ii. Further research can be conducted on column casting vertically since in this study columns were casted horizontally.

iii. Effect of aggregate size and type were not investigated in this study. Hence, future research can be conducted by varying aggregate size and type of concrete.

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APPENDIX

Table 7-1 Percentage of capacity reduction for lateral reinforcement of 13.5 and 17.2 MPa concrete for tie bar spacing Category-I (T1).

		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
300x300	NC	1646		1908	
	CMD (50mm)	1610	2.16	1864	2.33
	Core (75mm)	1499	8.92	1761	7.69
	CMD (100mm)	1379	16.22	1610	15.62
300X375	NC	2051		2246	
	CMD (50mm)	2002	2.39	2202	1.98
	Core (75mm)	1890	7.81	2091	6.93
	CMD (100mm)	1748	14.75	1935	13.86
300X450	NC	2469		2776	
	CMD (50mm)	2420	1.98	2727	1.76
	Core (75mm)	2295	7.03	2633	5.13
	CMD (100mm)	2131	13.69	2420	12.82

continue		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
375X375	NC	2518		2945	
	CMD (50mm)	2478	1.59	2909	1.21
	Core (75mm)	2424	3.71	2865	2.72
	CMD (100mm)	2246	10.78	2673	9.21
300X525	NC	2816		3283	
	CMD (50mm)	2780	1.26	3252	0.95
	Core (75mm)	2736	2.84	3212	2.17
	CMD (100mm)	2562	9.00	3016	8.13
375x450	NC	2900		3390	
	CMD (50mm)	2878	0.77	3372	0.52
	Core (75mm)	2820	2.76	3314	2.23
	CMD (100mm)	2664	8.13	3172	6.43

continue		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
450X450	NC	3799		4484	
	CMD (50mm)	3794	0.12	4479	0.10
	Core (75mm)	3728	1.87	4417	1.49
	CMD (100mm)	3576	5.85	4252	5.16
525X525	NC	4306		5538	
	CMD (50mm)	4301	0.10	5534	0.08
	Core (75mm)	4248	1.34	5453	1.53
	CMD (100mm)	4115	4.44	5289	4.50
600X600	NC	6330		7606	
	CMD (50mm)	6325	0.07	7602	0.06
	Core (75mm)	6259	1.12	7500	1.40
	CMD (100mm)	6081	3.94	7308	3.92

Table 7-2 Percentage of capacity reduction for lateral reinforcement of 20.7, 27.6 and 34.5 MPa concrete for tie bar spacing Category-I (T1).

		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
300x300	NC	2122		2491		2780	
	Core (50mm)	2077	2.10	2455	1.43	2762	0.64
	Core (75mm)	1962	7.55	2318	6.96	2616	5.92
	Core (100mm)	1793	15.51	2126	14.64	2406	13.44
300X375	NC	2544		2998		3180	
	Core (50mm)	2509	1.40	2967	1.04	3158	0.70
	Core (75mm)	2375	6.64	2825	5.79	3029	4.76
	Core (100mm)	2215	12.94	2638	12.02	2834	10.91
300X450	NC	3069		3692		4350	
	CMD (50mm)	3034	1.16	3665	0.72	4324	0.61
	Core (75mm)	2909	5.22	3514	4.82	4181	3.89
	CMD (100mm)	2705	11.88	3292	10.84	3968	8.79

continue		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
375X375	NC	3269		3803		4248	
	CMD (50mm)	3238	0.95	3781	0.58	4221	0.63
	Core (75mm)	3180	2.72	3719	2.22	4168	1.88
	CMD (100mm)	3003	8.16	3581	5.85	4017	5.45
300X525	NC	3559		4155		4889	
	CMD (50mm)	3536	0.63	4137	0.43	4866	0.45
	Core (75mm)	3492	1.88	4092	1.50	4826	1.27
	CMD (100mm)	3296	7.38	3928	5.46	4644	5.00
375x450	NC	3710		4510		5204	
	CMD (50mm)	3701	0.24	4502	0.20	5200	0.09
	Core (75mm)	3639	1.92	4457	1.18	5160	0.85
	CMD (100mm)	3505	5.52	4293	4.83	5009	3.76

continue		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
450X450	NC	4453		5605		6548	
	CMD (50mm)	4448	0.10	5600	0.08	6543	0.07
	Core (75mm)	4395	1.30	5565	0.71	6503	0.68
	CMD (100mm)	4270	4.10	5382	3.97	6330	3.33
525X525	NC	6027		7464		7847	
	CMD (50mm)	6023	0.07	7460	0.06	7842	0.06
	Core (75mm)	5952	1.25	7420	0.60	7798	0.62
	CMD (100mm)	5783	4.06	7264	2.68	7686	2.04
600X600	NC	8705		10231		11494	
	CMD (50mm)	8701	0.05	10226	0.04	11490	0.04
	Core (75mm)	8612	1.07	10169	0.61	11423	0.62
	CMD (100mm)	8394	3.58	9982	2.43	11312	1.59

Table 7-3 Percentage of capacity reduction for lateral reinforcement of 13.5 and 17.2 MPa concrete for tie bar spacing Category-II (T2).

		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
300x300	NC	1632		1890	
	CMD (50mm)	1588	2.72	1842	2.59
	Core (75mm)	1477	9.54	1721	8.71
	CMD (100mm)	1352	17.17	1575	16.71
300X375	NC	2033		2224	
	CMD (50mm)	1979	2.63	2175	2.20
	Core (75mm)	1864	8.32	2051	7.60
	CMD (100mm)	1717	15.54	1895	14.60
300X450	NC	2447		2749	
	CMD (50mm)	2393	2.18	2700	1.78
	Core (75mm)	2269	7.27	2567	6.63
	CMD (100mm)	2100	14.18	2375	13.59

continue		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
375X375	CMD (100mm)	2500		2922	
	NC	2451	1.96	2874	1.67
	CMD (50mm)	2402	3.91	2825	3.35
	Core (75mm)	2215	11.39	2629	10.05
300X525	NC	2793		3256	
	CMD (50mm)	2753	1.43	3216	1.23
	Core (75mm)	2705	3.18	3163	2.87
	CMD (100mm)	2518	9.87	2958	9.15
375x450	NC	2882		3363	
	CMD (50mm)	2856	0.93	3336	0.79
	Core (75mm)	2793	3.09	3274	2.65
	CMD (100mm)	2629	8.80	3114	7.41
450X450	NC	3777		4453	
	CMD (50mm)	3768	0.24	4444	0.20
	Core (75mm)	3692	2.24	4368	1.90
	CMD (100mm)	3532	6.48	4186	5.99

continue		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
525X525	NC	4288		5511	
	CMD (50mm)	4284	0.10	5507	0.08
	Core (75mm)	4199	2.07	5413	1.78
	CMD (100mm)	4079	4.88	5258	4.60
600X600	NC	6308		7580	
	CMD (50mm)	6303	0.07	7575	0.06
	Core (75mm)	6196	1.76	7464	1.53
	CMD (100mm)	6018	4.58	7264	4.17

Table 7-4 Percentage of capacity reduction for lateral reinforcement of 20.7, 27.6 and 34.5 MPa concrete for tie bar spacing Category-II (T2).

		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
300x300	NC	2095		2469		2758	
	CMD (50mm)	2046	2.34	2420	1.98	2713	1.61
	Core (75mm)	1922	8.28	2282	7.57	2571	6.77
	CMD (100mm)	1757	16.14	2091	15.32	2366	14.19
300X375	NC	2518		2967		3158	
	CMD (50mm)	2473	1.77	2931	1.20	3136	0.70
	Core (75mm)	2340	7.07	2780	6.30	2989	5.35
	CMD (100mm)	2175	13.60	2598	12.44	2802	11.27
300X450	NC	3038		3665		4324	
	CMD (50mm)	2998	1.32	3630	0.97	4288	0.82
	Core (75mm)	2865	5.71	3487	4.85	4146	4.12
	CMD (100mm)	2660	12.45	3256	11.17	3914	9.47

continue		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
375X375	CMD (100mm)	3247		3781		4226	
	NC	3203	1.37	3745	0.94	4199	0.63
	CMD (50mm)	3140	3.29	3674	2.82	4128	2.32
	Core (75mm)	2963	8.77	3505	7.29	3981	5.79
300X525	NC	3527		4128		4862	
	CMD (50mm)	3492	1.01	4097	0.75	4835	0.55
	Core (75mm)	3438	2.52	4052	1.83	4791	1.46
	CMD (100mm)	3234	8.32	3848	6.79	4608	5.22
375x450	NC	3683		4484		5178	
	CMD (50mm)	3661	0.60	4466	0.40	5169	0.17
	Core (75mm)	3603	2.17	4408	1.69	5120	1.12
	CMD (100mm)	3461	6.04	4266	4.86	4978	3.87
450X450	NC	4430		5578		6521	
	CMD (50mm)	4426	0.10	5574	0.08	6517	0.07
	Core (75mm)	4364	1.51	5516	1.12	6468	0.82
	CMD (100mm)	4190	5.42	5320	4.63	6281	3.68

continue		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
525X525	NC	5996		7437		7820	
	CMD (50mm)	5992	0.07	7433	0.06	7815	0.06
	Core (75mm)	5907	1.48	7375	0.84	7758	0.80
	CMD (100mm)	5743	4.23	7215	2.99	7638	2.33
600X600	NC	8683		10204		11472	
	CMD (50mm)	8678	0.05	10200	0.04	11467	0.04
	Core (75mm)	8576	1.23	10124	0.78	11401	0.62
	CMD (100mm)	8358	3.74	9933	2.66	11263	1.82

Table 7-5 Percentage of capacity reduction for lateral reinforcement of 13.5 and 17.2 MPa concrete for tie bar spacing Category-III (T3).

		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
300x300	NC	1588		1846	
	CMD (50mm)	1535	3.36	1788	3.13
	Core (75mm)	1415	10.92	1664	9.88
	CMD (100mm)	1290	18.77	1512	18.07
300x375	NC	1984		2180	
	CMD (50mm)	1917	3.36	2113	3.06
	Core (75mm)	1797	9.42	1997	8.37
	CMD (100mm)	1655	16.59	1842	15.51
300x450	NC	2402		2709	
	CMD (50mm)	2331	2.96	2647	2.30
	Core (75mm)	2189	8.89	2513	7.22
	CMD (100mm)	2033	15.37	2313	14.61

continue		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
375x375	NC	2487		2878	
	CMD (50mm)	2415	2.86	2816	2.16
	Core (75mm)	2340	5.90	2762	4.02
	CMD (100mm)	2171	12.70	2558	11.13
300x525	NC	2753		3216	
	CMD (50mm)	2696	2.10	3185	0.97
	Core (75mm)	2638	4.20	3096	3.73
	CMD (100mm)	2451	10.99	2891	10.10
375x450	NC	2838		3318	
	CMD (50mm)	2802	1.25	3283	1.07
	Core (75mm)	2736	3.61	3207	3.35
	CMD (100mm)	2571	9.40	3051	8.04
450x450	NC	3736		4413	
	CMD (50mm)	3714	0.60	4390	0.50
	Core (75mm)	3621	3.10	4306	2.42
	CMD (100mm)	3461	7.38	4123	6.55

continue		13.5 MPa		17.2 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
525x525	NC	4244		5467	
	CMD (50mm)	4226	0.42	5445	0.41
	Core (75mm)	4141	2.41	5351	2.12
	CMD (100mm)	4012	5.45	5182	5.21
600x600	NC	6268		7540	
	CMD (50mm)	6263	0.07	7535	0.06
	Core (75mm)	6139	2.06	7406	1.77
	CMD (100mm)	5952	5.04	7193	4.60

Table 7-6 Percentage of capacity reduction for lateral reinforcement of 20.7, 27.6 and 34.5 MPa concrete for tie bar spacing Category-III (T3).

		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
300x300	NC	2051		2429		2758	
	CMD (50mm)	1993	2.82	2371	2.38	2687	2.58
	Core (75mm)	1859	9.33	2224	8.42	2544	7.74
	CMD (100mm)	1695	17.35	2024	16.67	2331	15.48
300x375	NC	2464		2927		3118	
	CMD (50mm)	2411	2.17	2874	1.82	3074	1.43
	Core (75mm)	2264	8.12	2718	7.14	2918	6.42
	CMD (100mm)	2108	14.44	2522	13.83	2727	12.55
300x450	NC	2998		3621		4284	
	CMD (50mm)	2936	2.08	3581	1.11	4244	0.93
	Core (75mm)	2802	6.53	3403	6.02	4057	5.30
	CMD (100mm)	2598	13.35	3163	12.65	3817	10.90

continue		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
375x375	CMD (100mm)	3207		3745		4186	
	NC	3154	1.66	3714	0.83	4159	0.64
	CMD (50mm)	3083	3.88	3616	3.44	4052	3.19
	Core (75mm)	2891	9.85	3443	8.08	3879	7.33
300x525	NC	3487		4088		4822	
	CMD (50mm)	3461	0.77	4066	0.54	4800	0.46
	Core (75mm)	3385	2.93	3981	2.61	4724	2.03
	CMD (100mm)	3163	9.31	3785	7.40	4484	7.01
375x450	NC	3643		4448		5142	
	CMD (50mm)	3616	0.73	4422	0.60	5120	0.43
	Core (75mm)	3541	2.81	4337	2.50	5040	1.99
	CMD (100mm)	3394	6.84	4168	6.30	4862	5.45
450x450	NC	4390		5542		6485	
	CMD (50mm)	4386	0.10	5538	0.08	6481	0.07
	Core (75mm)	4306	1.93	5471	1.28	6419	1.03
	CMD (100mm)	4150	5.47	5262	5.06	6192	4.53

continue		20.7 MPa		27.6 MPa		34.5 MPa	
Column Size, mm	Column Type	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced	Column Capacity, KN	Percentage of capacity reduced
525x525	NC	5956		1664		32925	
	CMD (50mm)	5952	0.07	1663	0.06	32905	0.06
	Core (75mm)	5849	1.79	1644	1.20	32529	1.20
	CMD (100mm)	5671	4.78	1606	3.49	31777	3.49
600x600	NC	8643		2286		45252	
	CMD (50mm)	8638	0.05	2287	-0.04	45232	0.04
	Core (75mm)	8518	1.44	2260	1.14	44717	1.18
	CMD (100mm)	8278	4.22	2206	3.50	43649	3.54