ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

EFFECT OF USING HIGH STRENGTH CONCRETE COLUMNS ON THE STRUCTURAL BEHAVIORS OF BUILDING FRAME

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EFFECT OF USING HIGH STRENGTH CONCRETE COLUMNS ON THE STRUCTURAL BEHAVIORS OF BUILDING FRAME

BY

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DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other university and that all sources of material used for the thesis have been duly acknowledged.

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List of Symbols

A_s or A_{st}	*Area of reinforcement bar*
A_g or A_c	*Area of concrete*
b	*Width of section*
c	*Neutral axis depth*
C_c	*Force in compressed zone of concrete*
d	*Concrete section depth to the outermost tensile steel*
E_c	*Modulus of elasticity of concrete*
E_s	*Modulus of elasticity of reinforcing bar*
E_cI_e or EI_e	*Effective stiffness of section*
ϵ_{cu}	*Ultimate concrete compressive strain*
ϵ_{cm}	*Concrete axial strain*
ϵ_s	*Reinforcing bar strain*
ϵ_y or ϵ_{yd}	*Yield strain of reinforcement bar*
f_c' or f_{ck}	*Cylinder specified compressive strength of concrete*
f_{cu}	*Cubic specified compressive strength of concrete*
f_r	*Modulus of rupture of concrete*
F_s	*Force in reinforcing bar*
F_{cd}	*Design concrete strength*
f_s	*Stress in reinforcing bar*
f_y	*Yield strength of reinforcement bar*
f_{yk}	*Characteristic strength of reinforcing bar*
H	*Total building height from the base*
h	*Story height*
h	*Total depth of section*
I	*Section moment of inertia*
I_e	*Section effective moment of inertia*
M	*Moment*
M_n	*Nominal flexural resistance*
M_{xd}	*Column design moment in the X-direction*
M_{yd}	*Column design moment in the Y-direction*

List of Symbols (continued)

P	*Applied axial load of column*
P_n	*Column nominal axial resistance*
P_u	*Section axial load capacity*
T	*Building period*
α	*Concrete cylinder strength reduction factor*
β	*A factor relating depth of equivalent rectangular compressive strength of concrete to neutral axis depth*
Φ	*Curvature*
Φ_u	*Ultimate curvature*
φ	*Strength reduction factor*
Φ	*Diameter of reinforcing bar*
Δ	*Building top story deflection relative to base*
δ	*Story deflection relative to the the story just below it*
ρ	*Longitudinal reinforcing bar area ratio*
ρ_s	*Ratio of volume of spiral reinforcement to volume of concrete core*
σ	*Stress*

ABSTRACT

High strength concrete (HSC) has been used in the lower story columns of high rise buildings owing to its qualities over normal strength concrete (NSC) in many countries. But, the full structural qualities of the HSC were unable to be used because of insufficient information regarding the structural behavior of the material and its properties were not adequately addressed in building codes including EBCS. Analytical study was conducted at structure level to investigate the effect of using HSC column on the structural behavior of regular models of medium to high rise frame buildings under seismic lateral load in addition to gravity loads. Concrete strength variations of C30 to C90MPa were applied on the columns of the frame models. The proposed properties of the HSC class were incorporated in the analysis and design of the columns. The frames analysis was done using ETABS and columns were designed based on the EBCS column design procedure.

Columns moment- curvature curves were developed and maximum interstorey drifts were obtained for the different frame models with variation in columns concrete strength. The study shows that frames with HSC columns have got lower stiffness and performed well in satisfying ductility demand. The maximum interstorey drifts are slightly higher for frames with HSC columns, but the contribution of the concrete strength in resisting the lateral deformations was significant. Economic comparisons were also made and it was found that the most economical frame corresponds to frame with the highest columns concrete strength.

CHAPTER ONE

1 Introduction

1.1 General

In developing countries, the increasing reliance of employment on economic and social considerations is one of the reasons that lead to increasing rural-to-urban migration which in turn lead to increased demand on land use in large cities like Addis Ababa. Following this, more high rise structures are being constructed now than in the past. On the other hand, for the developed countries, the engineering challenge where by the two targets of boasting the longest bridge and the highest building have become serious considerations in the conceptual design of landmark projects is another stimulus for construction of high rise buildings. Thus, the need for higher buildings naturally leads to the conclusion that high strength construction materials will be increasingly used in the future. The following three performance criteria lend weight to the argument for the use of high strength concrete (HSC) for such high rise buildings. Firstly, column sizes should be kept at manageable dimensions in order to make more effective use of floor areas, especially in the lower story's of high rise structures. Secondly, increased wind, increase in seismic force in seismic prone areas and traffic vibration susceptibility dictates that the modulus of elasticity of the material should be as high as possible in order to limit small amplitude elastic deformations. And the third point is the need for rapid construction requires early age strength gain, a feature that may be offered readily by high strength concrete. The combined effect of the three above mentioned requirements renders high strength concrete economics rather appealing. On the other hand, HSC is the important development direction for the concrete material in civil structures for the advantages of higher-strength, earlier curing strength and smaller deformation in applications than that of normal strength concrete (NSC). It has been stated [8] that many performances of HSC structure are usually better than that of NSC structure except the relatively weak capacity of deformations (brittleness). But due to inadequate research regarding the properties of the HSC, effect related to design and construction of HSC building members were not sufficiently addressed in building codes including EBCS. As a result, the full structural qualities of the material were unable to be used in many countries. To bring the HSC into application, the development and the strength limit given to it has to be understood as stated in the following section.

1.1.1 Definition and Development of HSC

Although high-strength concrete is often considered a relatively new material, its development has been gradual over many years. As the development continued, the definition of high-strength concrete has changed with time and geographical location due to lack of a standard criterion for the strength that is required to qualify as a high strength concrete. In the 1950s, concrete with a compressive strength of 34 MPa was considered high strength. In the 1960s, concrete with 41 and 52 MPa compressive strengths were used commercially. In the early 1970s, 62 MPa concrete was being produced. More recently, compressive strengths approaching 138 MPa have been used in cast-in-place buildings [5]. Thus, different countries or standards give different strength limit to be considered as high strength concrete. For instance, concretes that were considered to be high strength according to ACI 363R-10 [5] are those that attain cylinder compressive strength of at least 41MPa at 28 days. But in the FIP/CEB (1990) [10] state-of-the-art report on high strength concrete, it is defined as concrete having a 28-day cylinder compressive strength of 60MPa. The following typical classifications based on strength are observed in some literatures.

Normal strength…………………. 20-50MPa
High strength……………………. 50-100MPa
Ultra high strength……………… 100-150MPa
Special strength ………………… >150MPa

The continuous increase in the compressive strength of concrete has been observed in the past decades, though with restricted availability. Today, high strength concrete in excess of 100MPa is already being used for high rise buildings, long-span bridges and offshore structures in many parts of the world due to its superior performance and strength especially in construction of columns of multistory buildings [28]. But in Ethiopia, even though the construction of high rise buildings are increasing in recent years in some cities of the country, the practice of using high-strength concrete (HSC) for such projects is very low. Thus, in addition for the performance criteria listed above, additional information regarding the structural behavior of HSC columns of multistory building is required for awareness creation in using HSC and for addressing the various advantages of the HSC over that of the NSC.

In this thesis, the effect of HSC columns on structural behavior of frame buildings will be investigated through analytical study. The economic and structural behavior issues of the HSC column for the model frame buildings have been given attention in the study. Therefore, concrete having the 28-day cylinder compressive strength greater than 40MPa is considered as HSC.

1.2 Statement of the problem

The application of high strength concrete has preceded research and therefore the behavior of the material cannot yet be predicted with reasonable accuracy. As a consequence, important issues related to design and construction of high strength concrete structures are not adequately addressed in building codes including EBCS. Structural designers are unable to take the full advantage of the material because of insufficient information. Therefore, research on reinforced HSC columns subjected to axial loading and bending moment is needed to provide designers and researchers with additional information on the structural behavior of reinforced HSC column and that of the frame building, and to provide data to evaluate preference of the HSC over the NSC for columns of medium to high rise buildings under both static and seismic loadings.

Several research projects have been done concerned with the experimental behavior of reinforced high strength concrete at material level. There are very few studies on high strength concrete at the structure level. This study is important for it provides detailed analytical investigations into structural behavior of frame buildings with HSC columns. Therefore, determining to what extent will the different concrete strength are sensitive to structural behavior gives information required on the need to control such high strength concrete.

1.3 Objective of the study

1.3.1 General objective

The objective of this thesis is to investigate the structural behavior of medium to high rise frame building with reinforced HSC columns subjected to seismic lateral load in addition to gravity loads. In light of this, the variation of the different structural responses due to change in columns concrete strength for regular moment resisting building frames will be studied. This will provide data which determines the need for using HSC columns over NSC for medium to high rise buildings and the HSC will be given attention by structural designers. The obtained data which is related to structural responses will act as a supportive document for possible decision to be made on the need for awareness creation in using HSC column and increase confidence that it can be used economically for high rise building frames.

1.3.2 Specific objective

This thesis has a specific objective of conducting analytical study on the structural response of moment resisting concrete framed buildings due to variation in columns concrete strength. The parameters used for comparisons includes, story maximum displacement, maximum inter-story drift, axial load level and curvature ductility of the columns. In addition, economic advantages of the HSC will be evaluated by comparisons of material's quantity and cost. It is aimed at comparing effects on the above parameters due to change in columns concrete strength and determining the suitability of HSC over NSC columns in resisting lateral loads. This will be done with the variation from NSC of 30MPa to HSC of 90MPa concrete columns to clearly visualize the effect.

1.4 Scope of the study

This study is limited to analysis of moment resisting concrete frame buildings with number of story not less than ten which was believed to represent medium high rise buildings. Sample regular moment resisting frame buildings with rectangular horizontal layout will be modeled using structural analysis program ETABS. The buildings frame will be loaded and analyzed based on EBCS 2, 1995, ACI building code and other relevant building code provisions. Concrete class variation will be applied on the columns of the frame. For different concrete strength of the columns, the same building frame will be analyzed. Keeping all other design parameters constant except the columns cross section dimension, the effect of alteration in columns concrete strength on structural behavior of the frames will be investigated. Because higher number of story dictates the use of other lateral load resisting system which is outside the scope of this thesis, the number of story is limited to not greater than twenty five.

CHAPTER TWO

2 Literature Review

2.1 General

Over the last few decades, the development in material technology, especially with the availability of mineral and chemical admixtures, led to the production of higher concrete strength grade. Since then, a series of research studies have been conducted on the behavior of such concrete [19]. Some previous studies confirmed improvements in the behavior of reinforced concrete columns and building frames under seismic loads as a result of using HSC. But a large number of studies have demonstrated the economy of using high-strength concrete in columns of high-rise buildings, as well as low to medium-rise buildings where it has been mostly applicable. In addition to reducing column size and producing a more durable material, the use of high-strength concrete has been shown to be advantageous with regard to lateral stiffness and axial shortening. Another economic advantage cited in the use of high-strength concrete columns is the reduction in the cost of formwork.

The economic advantages of using high strength in the columns of high-rise buildings have been known for many years. In simple terms, high strength concrete provides the most economical way to carry a vertical load to the building foundation [20]. The three major components contributing to the cost of a column are concrete, steel reinforcement and formwork. By utilizing high-strength concrete, the columns size is reduced. Consequently, less concrete and less formwork is needed which increases another advantage in space requirement.

In tall building structures, the dead load plays a very severe effect on structural members, especially the columns near the ground level which are required to resist a tremendous axial load which is mainly due to the accumulated dead load from all the floors above. In fact, in medium rise building (20-30 stories high), the size of the columns at the ground floor may have a diameter more than one meters when normal strength concrete is used. It can be imagined that there will be no space in the ground level if normal strength concrete is used for a very tall building (more than 60 story). Hence, it is a normal trend to adopt HSC in tall building construction due to its advantages, such as high strength, smaller size and high durability. It has been argued that, when judging the strength of a column, it is not only a matter of ensuring that stresses in the member are kept below a certain specified value, but also of preventing the peculiar state of unstable equilibrium [27]. Buckling has become more of a problem in recent years since the use of high-strength material requires less material for load support-structures and components have become generally more slender and buckle-prone. As a result, the slenderness limits based on high-strength concrete grades have to be implemented to make use of the merits of HSC for such member.

2.2 Material composition of HSC

The structural behaviors of reinforced concrete such as the stress-strain response of concrete, its compressive strength and its ultimate strain are functions of several parameters, including the mix proportions, type of cement and cementitious materials, type of admixtures, and grading of aggregates. The material selection and mix proportioning of HSC are more critical than those of

NSC. Therefore, it is important to review the composition and properties of the ingredient materials of HSC.

Concrete is composite consisting of aggregates enclosed in a matrix of cement paste which has two major components cement paste and aggregates. The strength of concrete depends upon the strength of these components, their deformation properties, and the adhesion between the paste and aggregate surface. With most natural aggregates, it is possible to make concretes up to 120MPa compressive strength by improving the strength of the cement paste, which can be controlled through the choice of water-content ratio and dosage of admixtures. However, with the recent advancement in concrete technology and the availability of various types of mineral and chemical admixtures, and special superplasticizer, concrete with a compressive strength of up to 100MPa can be produced commercially with an acceptable level of variability using ordinary aggregates. These developments have led to increased applications of high-strength concrete (HSC) all around the globe [26].

Production of HSC may or may not require special materials, but it definitely requires materials of highest quality and their optimum proportions. The choice of the constituents and of their mix proportions as to satisfy requirements concerning the properties of fresh concrete, the specified properties of the hardened concrete and durability taking account of the conditions of exposure. Particularly, the production of HSC that consistently meets requirements for workability and strength development places more stringent requirements on material selection than that for normal strength concrete. However, many trial batches are often required to generate the data that enables the researchers and professionals to identify optimum mix proportions for HSC.

2.2.1 Constituent materials for high strength concrete

2.2.1.1 Cement

Strength development of concrete will depend on both cement characteristic and cement content. Cement to be used shall be Portland or Portland-Pozzolana according to EBCS 2, 1995 [12].The choice of Portland cement for HSC is extremely important unless high initial strength is the objective, such as in prestressed concrete. In some conditions for example, when the temperature rise is expected to be a problem, Type-II low-heat-of-hydration cement can be used, provided it meets the strength-producing requirements [5].

For HSC containing no chemical admixture or fly ash, a high cement content of 10 to 12 sacks/m^3 (500 to 600 Kg/m^3) of concrete must be used. The optimum cement content depends on cement type. For instance, 13 sacks/m^3 for Type-I cement and 11 sacks/m^3 for Type-II cement are recommended by design standards. A sack of cement is equal to 50Kg bag [26].

2.2.1.2 Water and water-cement ratio

The single most important variable in achieving HSC is the water-cement ratio. HSC produced by conventional mixing technologies are usually prepared with water-cement ratios in the range of 0.22 to 0.40, and their 28 days compressive strength is about 60 to 130 MPa when normal density aggregates are used [26]. Fig.1 below shows the decrease in compressive strength as w/c ratio increases. The requirements for water quality for HSC are no more stringent than those for conventional concrete. Usually, water for concrete is specified to be of potable quality [5].

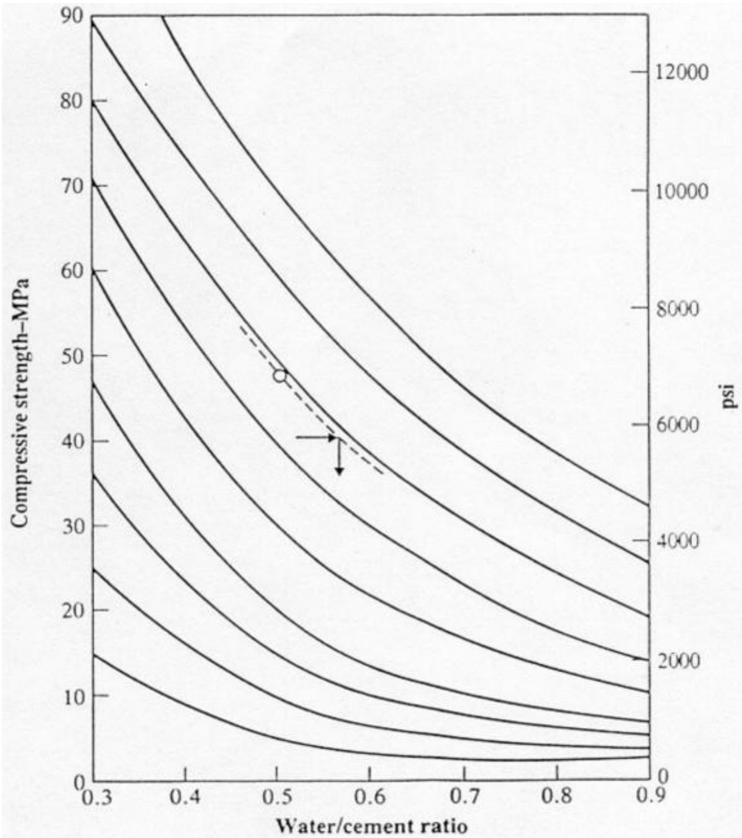


Figure 2.1: Relationship between concrete compressive strength and w/c ratio **[30]**

2.2.1.3 Aggregates

The properties of aggregates are decisive for the compressive strength and modulus of elasticity of HSC. Using the right type and quality of aggregates in the production of HSC should be emphasized. In normal strength concrete (NSC), the aggregate has a higher strength and stiffness than the cement paste. Failures in NSC are characterized by fractures in the cement paste and in the transition zone between paste and aggregate. Reduced water-cement ratio, therefore, causes a great improvement in compressive strength of cement paste and hence of high strength concrete.
The fine and coarse aggregates generally occupy 60% to 75% of the concrete volume (70% to 85% by mass) and strongly influence the concrete's freshly mixed and hardened properties, mixture proportions, and economy [26].

2.2.1.4 Coarse aggregate

In HSC the capacity of the aggregates can be the limiting factor. This may be either the result of the aggregate being weaker than the low water-cement matrix, or alternatively it is not sufficiently strong and rigid to provide the strengthening effect. This is mainly related to the coarse aggregates (CA) which consist of one or a combination of gravels or crushed stone with particles predominantly larger than 5 mm and generally between 9.5 mm and 37.5 mm. Obtaining HSC requires reduction of CA size distribution. For instance, laboratory sieve analysis done for grading of CA for C90 concrete shows the sizes are limited between 5mm and 19mm which are within the recommended range. The strength increases were caused by the reduction in average bond stress due to the increased surface area of the individual aggregate. Smaller aggregate sizes are also considered to produce higher concrete strengths because of less severe concentrations of stress around the particles, which are caused by differences between the elastic moduli of the paste and the aggregate. Many studies have shown that crushed stone produces higher strengths than rounded gravel. The most likely reason for this is the greater mechanical bond, which can develop with angular particles. However, accentuated angularity is to be avoided because of the attendant high water requirement and reduced workability [26]. The ideal CA should be clean, cubical, angular, 100% crushed aggregate with a minimum of flat and elongated particles [5].

Among the different crushed aggregates that have been studied – traprock, quartzite, limestone, greywacke, granite, and crushed gravel traprock tends to produce the highest concrete strength. Limestone, however, produces concrete strengths nearly as high as those achieved using traprock. Almost all of these materials are available in many parts of Ethiopia. Gradation of CA within ASTM limits makes very little difference in strength of HSC. Optimum strength and workability of HSC are attained with a ratio of CA to FA above that usually recommended for NSC [26].

2.2.1.5 Fine aggregate

Fine aggregates (FA) generally consist of natural sand or crushed stone with most particles smaller than 5 mm. A rounded particle shape and smooth texture have been found to require less mixing water in concrete and for this reason they are preferable in HSC [26]. HSC typically contain such high contents of fine cementitious materials that the grading of the FA used is relatively unimportant. However, it is sometimes helpful to increase the fineness modulus (FM) as the lower FM of FA can give the concrete a sticky consistency (i.e. making concrete difficult to compact) and less workable fresh concrete with a greater water demand. Therefore, sand with a FM of about 3.0 is usually preferred for HSC [5].

2.2.1.6 Admixtures

Admixtures are widely used in the production of HSC. These materials include air-entraining agents and chemical and mineral admixtures. Significant increases in compressive strength, control of rate of hardening, accelerated strength gain, improved workability, and durability are contributions that can be expected from the admixture or admixtures chosen. Reliable performance on previous work should be considered during the selection process [26].

2.2.1.6.1 Chemical admixtures

Chemical admixtures such as superplasticizers (high–range water reducer) increase concrete strength by reducing the mixing water requirement for a constant slump, and by dispersing cement particles, with or without a change in mixing water content, permitting more efficient hydration. The main consideration when using superplasticizers in concrete are the high fines requirements for cohesiveness of the mix and rapid slump loss. Neither is harmful for the production of HSC. HSC mixes generally have more than sufficient fines due to high cement contents. The use of retarders, together with high doses and redoses of superplasticizers at the plant or at the job site can improve strength while restoring slump to its initial amount. Even a super plasticized mix that appears stiff and difficult to consolidate is very responsive to applied vibration [26].

2.2.1.6.2 Mineral admixtures

Finely divided mineral admixtures, consisting mainly of fly ash and silica fume (SF), and slag cement has been widely used in HSC. Fly ash for HSC is classified into two classes. Class F fly

ash is normally produced from burning anthracite or bituminous coal and has pozzolanic properties, but little or no cementitious properties. Class C fly ash is normally produced from burning lignite or sub-bituminous coal, and in addition to having pozzolanic properties, has some autogenous cementitious properties [5]. When adding fly ash during concrete production, the workability is normally improved due to the 'lubricating' effect of the spherical particles.
Silica fume (SF) is a by-product of the melting process used to produce silicon metal and ferrosilicon alloys. The main characteristics of SF are its high content of amorphous SiO2 ranging from 85 to 98%, mean particle size of 0.1 – 0.2 micron (approximately 100 times smaller than the average cement particle) and its spherical shape [26]. Because of its extreme fineness and high silica content, SF is a highly effective pozzolanic material. The SF reacts pozzolanically with the lime during the hydration of cement to form the stable cementitious compound calcium silicate hydrate (CSH). Normal SF content ranges from 5 to 15 percent of Portland cement according to ACI 363R-10 [5]. The use of SF as replacement of a part of the cement gives considerable strength gain. For most binder combinations, the use of SF is the only way of producing concrete of normal workability with a strength level exceeding 80 MPa. To ensure a proper dispersion of the ultra-fine SF particles, plasticizers should be used in these mixtures.

2.2.1.7 Summary for requirements of ingredients for high strength concrete

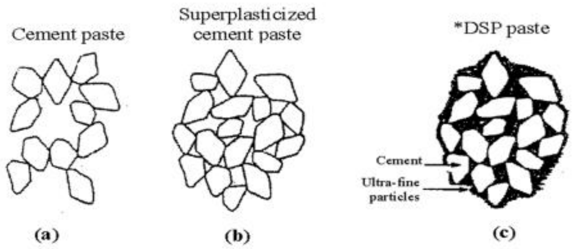
From the preceding discussions on information found from literature, the necessary requirement of different ingredient materials required for producing HSC can be summarized as stated in Table 2-1.

Material/issue	Requerements
Cement	- Portland cement - Higher content (10 to 12 sacks/m^3 of concrete)
Water	- Potable quality - w/c ratio 0.22 to 0.40
Fine Aggregate	- Sand with rounded particle shape - Higher FM (around 3.0) - Smaller sand content or coarser sand - Grading is not critical for concrete strength
Course Aggregate	- Smaller maximum size (10 – 20 mm) is preferred - Angular and crushed with a minimum flat and elongated particles - Type of aggregate depending on the concrete strength targeted - Gradation within ASTM limits has little effect on concrete strength - Higher CA/FA ratio than that for normal strength concrete
Admixtures (chemical & mineral)	- Type of admixture depends on the property of the concrete to be improved - Reliable performance on previous work can be considered during selection - Optimum dosage
Overall basic considerations	- Quality materials - Improved quality of cement paste as well as aggregates - Denser packing of aggregates and cement paste - Improved bond between aggregate surface and cement paste - Minimum numbers as well as smaller sizes of voids in the paste

Table 2-1: Requirements of ingredient-materials for high strength concrete **[**26**]**

2.3 Basic considerations for the mix design of high strength concrete

The performance of high-strength concrete is highly dependent on the properties of its individual components. Thus, ability to produce high strength concrete on a routine economic basis is crucial. Proper proportioning is required for all materials used. The proportioning procedure is meant to produce mixture proportions based on the performance of adjusted laboratory and field trial batches. This procedure further ensures that the properties and characteristics of the materials used in the trial mixtures are adequate to achieve the desired concrete compressive strength. The availability of such constituent materials with adequate properties for HSC has no doubt within Ethiopia. But, Table 8-1 in EBCS 2, 1995 [12] describes standard mix proportions only for ordinary structural concrete graded from C5 to C30. Therefore, for HSC, the procedures and steps in ACI 211-4R-10 [3] method of mix design for high strength concrete could be followed. Accordingly, several laboratory mix design preparation for HSC have been done by graduate students in AAiT of Addis Ababa University. For instance, a laboratory report on trial mix preparation for C25 and C90 concrete by 'Anteneh et al' [7] shows that it is possible to get the required strength using OPC cement and locally available constituent materials following the procedures and steps indicated by the standards. For the HSC C90, they followed the ACI procedures with the possible grading and quality of materials within the limit to attain the specified strength. Their test result showed C-90 concrete attained a 28-day compressive strength of 83.45MPa, which is 92.7% of the specified strength. This was not significantly lower than the specified value. Therefore, they suggested that the specified strength could be attainable with little modifications to the constituent materials within the limit without affecting the economy.
On the other hand, the mechanical properties and paste-aggregate bond of the concrete can be improved by adding ultra-fine Particles such as silica fume (SF). This effect might be explained by so-called 'DSP-concrete' (Densified Systems containing homogeneously arranged ultra-fine Particles) (Figure2.2). The size and spherical geometry of SF particles allow them to fill effectively the voids between the larger and angular cement grains. As a result, very low water demand is obtained and thus, pastes of very low water-cement ratios can be produced [26].



.
(*DSP – Densified Systems containing homogeneously arranged ultra-fine Particles)

Figure 2.2: The structure of the cement paste in fresh concrete

A dense cementitious matrix is not sufficient by itself to obtain HSC but the aggregate-matrix bond must be strong enough. Generally, the pillars of practical mix design for HSC are:

- Quality materials, recommended size limit and optimum proportions.
- Reduced water-cement ratio.
- Extensive use of admixtures.
- Application of cement with a high strength potential.
- Application of pozzolans and in particular SF.

2.4 Summary for production materials of HSC

HSC that consistently meets requirements for workability and strength development places more stringent requirements on material selection and mix proportioning than that for lower strength concrete. Therefore, the production of HSC may or may not require the special materials, but it definitely requires materials of highest quality and their optimum proportions. In the production of HSC, use of strong, sound and clean aggregates is essential. The basic trick then lies in reducing the capillary pores in the matrix and improving the bond strength between cement matrix and aggregate. These can be accomplished by using low water-cement ratio and incorporating ultra-fine particles (particles much smaller than the grains of cement, such as silica fume) in the concrete mix, but not at the expense of workability necessary to achieve adequate compaction. The requirements of ingredient materials and the basic considerations in producing HSC are described in Table 2-1. [26].

In general, several techniques can be used to produce HSC. However, the above discussions can be summarized into; adjustment to mix proportions, using high density materials and the use of admixtures are the major methods of producing high strength concrete.

2.5 Structural Design consideration of HSC

High-strength concretes have some characteristics and engineering properties that are different from those of normal strength concretes. The use of higher-strength concretes permits more efficient structural designs, allowing members to span longer distances, be smaller in cross section, and carry larger loads. The HSC members design are more likely to be controlled by serviceability and other practical design considerations instead of strength. As a result, special considerations may be required in the design of high-strength concrete structural members.

2.5.1 Flexural strength of HSC column

The use of high-strength concrete in construction industry has expanded in recent years for its superior strength and performance. However, many aspects of structural design for high-strength concrete members remain to be developed. Of fundamental importance is the development of equivalent **rectangular stress block** that is applicable to high-strength concrete. For very high strength concrete the idealized stress-strain curve is almost linear up to and beyond a strain of 0.003. As a result the idealized concrete stress block is triangular in shape. Figure 2.3 illustrates the variation in stress distribution for NSC and HSC [23]. To account for this variation, modifications in the stress block parameter, $\alpha_1$and β_1, have been suggested by several authors where α_1 is the concrete cylinder strength reduction factor, and $\beta_1$is a factor relating depth of equivalent rectangular compressive strength block to neutral axis depth.

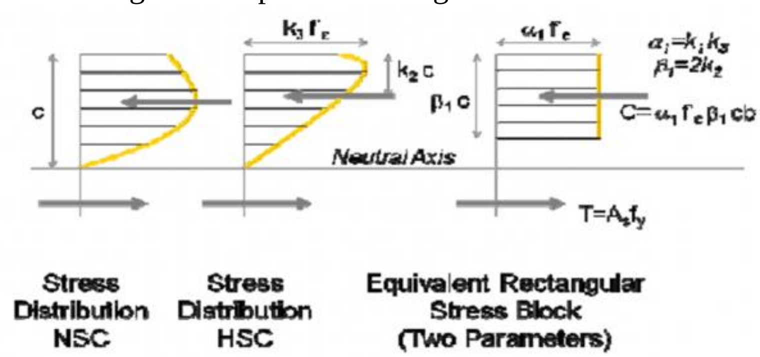


Figure 2.3: Stress Distribution in NSC and HSC **[23]**

A generalized stress block is defined by three parameters, k_1, k_2 and k_3 as indicated in figure 2.4 [18]. The design values of the stress block parameters are determined at the ultimate strain ε_{cu}, which corresponds to the maximum moment of the section. The k_1k_3 value and the k_2 value can be obtained from the equilibrium of the external and internal forces as follows:

$P_n=k_1k_3f_c'bc+A_sf_y+A_s'f_y$

$M_n= k_1k_3f_c'bc(d-k_2c)+A_s'f_y(d-d')$

The three-parameters generalized stress block can be reduced to a two-parameter equivalent rectangular stress block, by keeping the resultant of the compression force at the mid-depth of the assumed rectangular stress block as shown in fig.2.4.

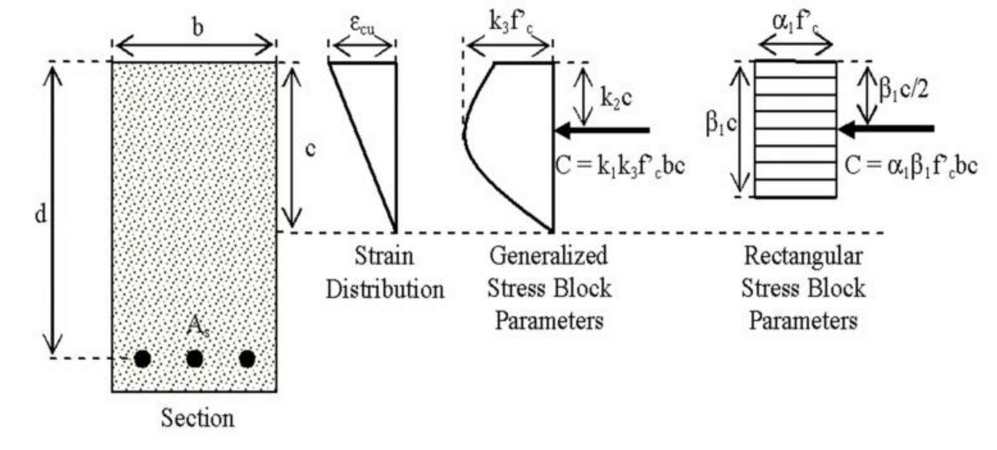


Figure 2.4: Stress Block Parameters for Rectangular Sections

The two parameters α_1 and β_1 can be defined as:

$\alpha_1=k_1k_3$

$\beta_1=2k_2$

The nominal axial and flexural resistance of the section can then be shown as: `

$P_n= \alpha_1f_c'A_g(A_g-A_s)+A_sf_y+A_s'f_y$

$M_n= \alpha_1f_c'A_g(d- \beta_1c/2)+A_s'f_y(d-d')$

For the equivalent rectangular stress block parameters required for the analysis of the columns cross section, the equivalent rectangular stress block model as given in some literatures can be implemented where α_1 and β_1 are the equivalent rectangular stress block parameters. Accordingly, the definitions and parametric equations to determine α_1 and β_1 for different concrete strength are described by some design codes and research publications as follows:

- ACI 318M-11 and LRFD defines the equivalent rectangular stress block having a width of $\alpha_1f_{c'}$ and depth of β_1c where c is the neutral axis depth. Factor β_1 shall be taken as 0.85 for concrete strengths $f_{c'}$ up to and including 30 MPa. For strengths above 30 MPa, β_1 shall be reduced continuously at a rate of 0.05 for each 7 MPa strength increase beyond 30 MPa. However, β_1 should not be taken as less than 0.65. Similarly, the factor α_1 shall be taken as 0.85 for concrete strengths $f_{c'}$ up to and including 55 MPa. For strengths in excess of 55 MPa, α_1 shall be reduced continuously at a rate of 0.015 for each 7 MPa strength increase beyond 55 MPa. However, α_1 shall not be taken as less than 0.7. The ultimate strain for concrete is given as ϵ_{cu}=0.003. The properties of the ERSB as defined by ACI 318M-11 [4] are summarized as:

  $\alpha_1=0.85$ for $f_c' \leq 55MPa$

  $\alpha_1=0.85-0.015(\underline{f_c'-55})/7 \geq 0.7$ for $f_c'>55MPa$

β_1=0.85 for f_c' ≤30MPa

β_1 =0.85-0.05(f_c'-30)/7 ≥0.65 for f_c'>30MPa

- The EC2-02 [15] treats the rectangular stress block parameters α_1 and β_1 for different concrete strength as:

  α_1=1 for f_{ck} ≤50MPa

  α_1=1-(f_{ck}-50)/200 for 50< f_{ck}≤90MPa

  β_1=0.8 for f_{ck} ≤50MPa

  β_1 =0.8-(f_{ck}-50)/400 for 50< f_{ck}≤90MPa

  The ultimate strain for concrete is given as:

  ϵ_{cu}=0.0035 for f_{ck} ≤50MPa and

  $\epsilon_{cu}=0.0026+0.035(\frac{90-fck}{100})^4$ for 50< f_{ck}≤90MPa

- The Canadian Code for Design of Concrete Structures [9] treats the flexural stress block for HSC in two ways. Design may be based on equations for the stress-strain curves of the concrete with peak stresses no greater than 0.9f_c. Alternatively, a modified rectangular stress block parameters are defined by:

  α_1 = 0.85 - 0.0015f'_c≥0.67 (f'_c in MPa)

  β_1 = 0.97 - 0.0025f'_c≥0.67 (f'_c in MPa)

  These two equations were further representing a stress-strain curve with peak stress not greater than 0.9f_c. Additionally, the Canadian code allows using 0.0035 as the maximum concrete strain.

- A comprehensive investigation assessing the applicability of the rectangular compression block for computing flexural strength of HSC columns is reported by Ibrahim and Macgregor [18]. Those authors also reported that for all specimens, the maximum concrete compressive strains before spalling were greater than 0.003 and concluded the following:
  - The rectangular stress block can be used to design HSC cross-sections with some modification to the parameters used to define the stress block.
  - They modified the compressive stress intensity factor α_1 as:

    α_1 = (0.85-0.00125 f'_c)≥0.725 (f'_c in MPa)
  - The distance from extreme compression face to centroid of the rectangular compression block (parameter $\beta_1 c/2$) is proposed as:

    β_1 = (0.95-0.0025 f'_c) ≥0.70 (f'_c in MPa)

But, James G. Macgregor recommended in his book [22] the following formula for β_1 without specifying weather to use for HSC, and it becomes less than 0.65 for C70 and C90.

β_1 = 1.09-0.008 f'_c ≥ 0.65 and less than 0.85

- The other researchers Ozbakkaloglu and Saatcioglu stated that the error introduced by using the ERSB of ACI 318-02 in analyzing sections of HSC may be small for beams and sections under low axial compression. However, the error becomes very substantial as the level of axial compression increases since in such cases the contribution of concrete to section behavior becomes more pronounced. Therefore, the rectangular stress block developed by Ozbakkaloglu and Saatcioglu (2003) [18] was also used to establish the moment–axial load interaction diagrams. The properties of ERSB's proposed by these researchers are summarized below. The width of the block is defined as $\alpha_1 f_c'$ and a depth as $\beta_1 c$.

α_1=0.85 for $f_c' \leq$ 30MPa
α_1=0.85-0.0014(f_c'-30)≥0.72 for f_c'>30MPa
β_1=0.85 for f_c' ≤30MPa
β_1 =0.85-0.0020(f_c'-30)≥0.67 for f_c'>30MPa

- EBCS 2, 1995 [12] uses 0.85 for α_1 and 0.8 for β_1 irrespective to the concrete compressive strength. The EBCS code allows using 0.0035 as the maximum concrete strain. In this code, concrete is graded as characteristic compressive cube strength ranging from C5 (lean concrete) to C60 (HSC). With conversion factor of 1.25, it was graded also as the characteristic cylinder compressive strength f_{ck} (equivalent to fc') ranging from f_{ck}=12 for C15 to f_{ck}=48 for C60. Where the number in the grade designation denotes the 28 days specified characteristic compressive strength in MPa.

For the purpose of this study, the values of α_1 and β_1 for concrete grades of C30, C50, C70 and C90 are computed based on the equations proposed from the different design codes and publications discussed above. The corresponding comparative values of α_1 and β_1 are shown in table 2-2.

Grades of concrete f_{cu} (MPa)	Equivalent cylindrical strength f_{ck} or fc' (MPa)	Parameters	ACI318M-11	EC2-02	Canadian Code	Ibrahim & Macgregor	Ozbakkaloglu & Saatcioglu
30	24	α_1	0.85	1	0.814	0.82	0.85
		β_1	0.85	0.8	0.91	0.89	0.85
		ϵ_{cu}	0.003	0.0035	0.0035	> 0.003	0.003
50	40	α_1	0.85	1	0.79	0.80	0.836
		β_1	0.78	0.8	0.87	0.85	0.83
		ϵ_{cu}	0.003	0.0035	0.0035	>0.003	0.003
70	56	α_1	0.848	0.97	0.766	0.78	0.814
		β_1	0.664	0.785	0.83	0.81	0.798
		ϵ_{cu}	0.003	0.002656	0.0035	0.003	0.003
90	75	α_1	0.814	0.89	0.742	0.756	0.79
		β_1	0.65	0.745	0.79	0.763	0.766
		ϵ_{cu}	0.003	0.0026	0.0035	0.003	0.003

Table 2-2: Values of proposed Rectangular Stress Block Parameters in different Design Codes and Publications

2.5.2 Axial Strength of HSC columns

Present design practice in calculating the nominal strength of an axially loaded member is to assume a direct addition law summing the strength of the concrete and that of the steel. The usual assumption is made that steel and concrete strains are identical at any load stage. The nominal axial strength of a reinforced concrete column is given by the expression:

$$P_n = \alpha_1 f_c'(A_c - A_{st}) + f_y A_{st} \quad \text{……………i}$$

in which f'_c is the specified concrete compressive strength , f_y is the yield strength of the longitudinal steel, A_c is the area of concrete section and A_{st} is the area of steel. In this expression, the concrete area displaced by the longitudinal reinforcement was taken into account. The factor

α_1 accounts for observed differences between the strength of concrete in columns and that of the control concrete cylinder of the same mix and varies depending on the concrete strength as discussed in section 2.5.1. In this equation, it is assumed that longitudinal steel bars in the column (if any) yield when concrete reaches its peak stress of f'_c. This assumption is generally justified, as the strain ε_{co} corresponding to the peak stress is usually about 0.002 [18].
For concentrically loaded columns with lateral steel reinforcement, P_n is taken as the first peak load that corresponds to the spalling of cover concrete.
For design purposes, the design strength of an axially loaded tied RC column is further reduced by two multipliers and is given by:

$$P_n = 0.8\phi\,[\,\alpha_1 f_c'(A_c - A_{st}) + f_y A_{st}] \quad \ldots\ldots\ldots\ldots ii$$

where ϕ(=0.70) is the strength reduction factor prescribed by the ACI Code and 0.80 is a factor that takes into account the effect of accidental eccentricities of loading not considered in the analysis. In the case of the computation of a HSC column capacity, some care shall be exercised with respect to equation (i). First, the ratio of column strength to cylinder strength α_1 should be used respective to the concrete compressive strength and as recommended by the design code under consideration (if any). This factor α_1 is somewhat smaller for HSC than for NSC columns as explained in the preceding section. The trend in table 2-2 shows, α_1 decreases as the concrete compressive strength increases, as proposed by researchers.
Additionally, in AS 3600, for concretes with cylinder strength up to about 65 Mpa, the peak stress reduction factor α_1 of 0.85 was specified in the standard. However, as the concrete cylinder strength moves beyond this point, the peak stress reduction factor should decrease to a lower value as shown in Figure 2.5. A limit of 0.72 has been suggested in some studies.

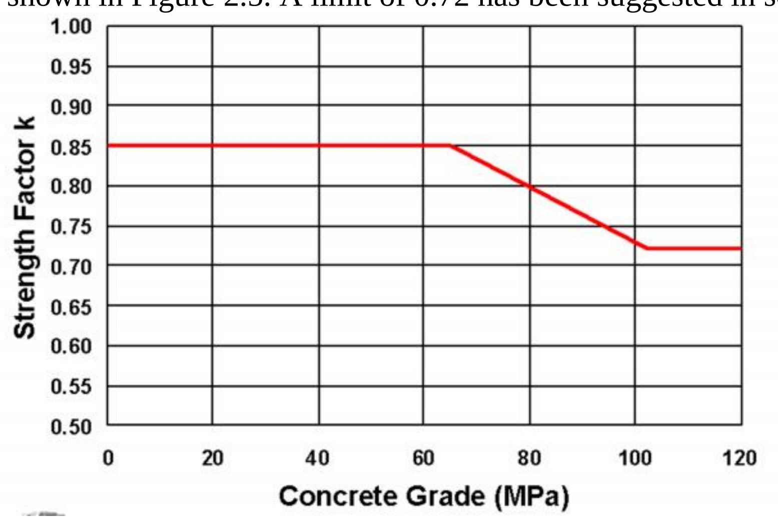


Figure 2.5: Strength factor α_1 verses concrete grade according to AS 3600.

2.5.3 Stress-strain and Ductility behavior of HSC and Steel reinforcement

2.5.3.1 Stress-strain curve of HSC

The use of High strength Concrete (HSC) elements (f'_c>41MPa) for concrete structures has been very popular with strengths of concrete over 100MPa used around the world. Many of the structural behavior of concrete depend on the axial stress-strain relationship of concrete as it varies with its strength. As shown in Figure 2.6, the ascending and descending portions of the curve become steeper with increasing strength [18]. The curves tend to become more linear for higher strength concretes which makes it less ductile compared to normal strength concrete. As a result, the equivalent rectangular stress block for high-strength concrete is expected to be different from that of normal-strength concrete as discussed in section 2.5.1. In order to use the main advantages of HSC which include higher strength and higher stiffness, improved durability, cost efficiency, reduced creep and drying shrinkage, better impact resistance and better resistance to abrasion, we need to consider its properties in the analysis and design of the structural element.

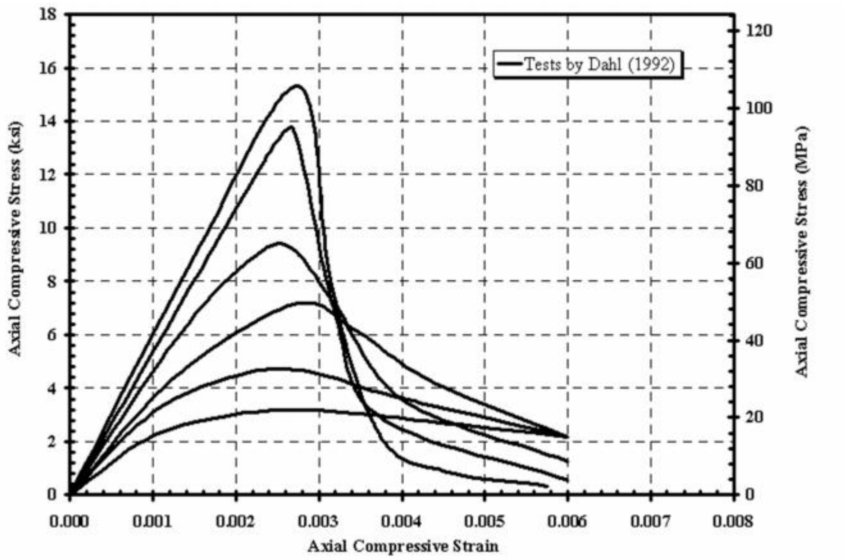


Figure 2.6: Axial compressive Stress-Strain curves for different strength of concrete

2.5.3.2 Stress-strain behavior of reinforcing steel

A simplified bilinear idealization of the steel stress-strain curve is assumed in which no strain hardening of the material is taken into account (figure 3.2) [12].

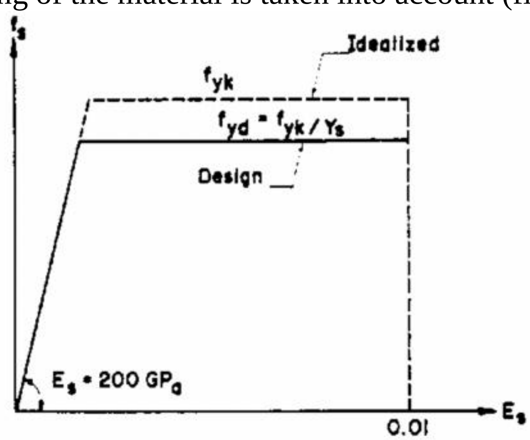


Figure 2.7: Simplified stress-strain diagrams for reinforcing steel

The evaluation of flexural strength of reinforced concrete columns by developing the moment-curvature curves generally depends on the assumed stress-strain curve of concrete and steel reinforcement. This simplified curve of the steel assumes the stress to remain at the yield stress for all strains exceeding the yield strain. But the ultimate tensile strength can be much higher than the yield stress if the stress-strain curve which considers strain hardening is used. In such case, the strain hardening develops only when the strain exceeds values of about 0.02 for high-yield steel which is much higher than the yield strain [29]. Hence, strain hardening may or may not develop even when reinforced concrete column fails completely. It is standard practice to ignore the strain hardening effect of steel reinforcement in structural design, as engineers generally believe that the flexural strength so evaluated is on the safe side. Considering this concept, the evaluations of flexural ductility of the designed column sections for the different concrete strength in this study are done through analysis of moment-curvature until the strain range of ultimate strength.

2.5.3.3 Ductility

Ductility is generally defined as the ability of a reinforced concrete member to undergo deformations without a substantial reduction in the flexural capacity of a member. It is an important structural property because it allows stress redistribution and gives warning of the imminence of failure. It is usually measured using the moment-curvature relationship for a given structural section. Research on HSC at material level shows that strength and ductility of concrete are inversely proportional. Thus, high- strength concrete columns are expected to exhibit brittle characteristics, developing sudden and explosive failures under concentric compression. Reinforced concrete columns which carry high axial loads can have a marked reduction in cross section size and the amount of longitudinal steel reinforcement can be substantially reduced when high strength concrete is used. Therefore, the main concern regarding the use of high strength concrete is the reduction in ductility with the increase in compressive strength as observed under uniaxial compression. To use the many advantages of the HSC, especially in the columns of high rise buildings, the properties of the HSC should be employed in the analysis and design of the column in order to have adequate curvature and displacement properties. Some researchers tried to investigate the structural behavior of HSC at structure level. Therefore it is important to review their findings with assumptions considered in the study.

One of the studies was by Kateinas summarized in Elnasha [8] which was the first detailed investigations into high strength concrete buildings. He used different concrete strength and steel yield, keeping the member size constant and changing the material properties. By keeping the section dimensions and ratio of reinforcement constant, many of the structures ended up with response typical of 'strength design' as opposed to 'ductility design' as well as sections being over reinforced. The approach used account comprehensively for the behavior observed. It was noted in that study that high strength concrete structures exhibit very low levels of ductility. Therefore, the study reported represents the expected behavior of high strength concrete structures.

The approach by Kateinas and Elnashai didn't use the advantages of HSC that member size would be reduced which contribute to economical design. On the other hand, keeping the member size constant led to over reinforced section for the HSC members. Also, the use of design expressions intended for normal strength concrete does not represent the properties of HSC. In this study, different approach to investigate the ductility level of HSC column in buildings was carried out considering the advantages of columns size reduction, the use of design expressions and properties of HSC.

Another analytical study carried out by 'A. Bourouz et al' using computer program which was based on fiber element model (FEM) for selected model was performed to investigate the effects of different parameters on the behavior of moment curvature curves. One of the parameter was variation of concrete strength from 30MPa to 60MPa. A constant square 40cm x 40cm reinforced concrete column section was used as a specimen to carry out the moment curvature analysis with a 2.5cm cover on all faces. In addition, steel ratios and axial load levels were kept constant. Using an iterative procedure, moments and the corresponding curvatures at failure for an assumed concrete strain were computed. From the graph of moment-curvature curve behavior, they concluded that the curvature ductility decreases with the increase of the concrete strength. However, the flexural strength of the section increases with the increase in concrete strength of the section [1].

Similar to the approach by Kateinas and Elnashai, they made section dimension constant for the different concrete strength. They also selected a single section model with fixed amount and distributions of steel which did not represent the actual members of the structure while concrete strength changes. Therefore, their result only shows the effect of concrete strength on a single section model. However, to use the advantages of HSC, a study at structure level which can represent the actual condition and makes use of the properties of the material is needed. Generally, reinforced HSC columns in moment-resisting frames constructed in areas of high seismicity should be proportioned to have adequate curvature and displacement properties, so that they are capable of inelastic response without appreciably losing load-carrying capacity. This could also be done by providing sufficient lateral reinforcement which was discussed in the following section.

2.6 Level of confinement in HSC column

Lateral reinforcement in columns, in the form of continuous spirals or tied, has two beneficial effects on column behavior: (1) it greatly increases the strength of the core concrete inside the spiral by confining the core against lateral expansion under load; and (2) it increases the axial strain capacity of the concrete, permitting a more gradual and ductile failure, that is a tougher column. This relationship for confined columns is illustrated in Figure 2.8 [2]. Confinement is less effective for HSC columns due to reduced lateral expansion of the concrete core. This reduced effectiveness can be attributed to less lateral expansion of the concrete core, which leads to lower stresses in the spirals at the peak load in high-strength concrete columns. Therefore, lateral confinement pressure required for HSC columns may be significantly higher than that for NSC columns to achieve a satisfactory deformability (ductility) level. The higher level of confinement pressure may be achieved using higher grades of lateral steel to avoid congestion of the reinforcement cage.

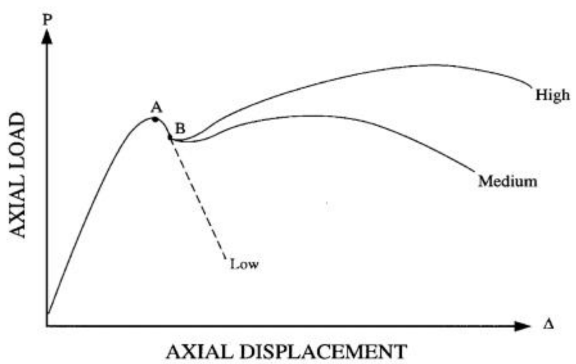


Figure 2.8: Schematic behavior of HSC columns subjected to concentric axial loads, incorporating low, medium, and high amounts of transverse reinforcement [2]

Generally, parameters that affect the confinement of concrete include yield strength, spacing, size, distribution, shape, and effectiveness of the confinement reinforcement, as well as distribution and yield strength of the longitudinal reinforcement, spalling of cover concrete, and level of axial load on the section.

Accordingly, the ACI 318 [4] equation for minimum volumetric ratio of spiral is given by:

$$\rho s = \frac{0.45\left(\frac{Ag}{Ac}-1\right)fc}{fy}$$

where:
ρ_s = ratio of volume of spiral reinforcement to volume of concrete core;
Ag = gross area of concrete section;
Ac = area of concrete core;
f'c = specified compressive strength of concrete; and
fy = yield strength of spiral steel.
Experimental data suggests similar trends for tied high-strength concrete columns as for high-strength concrete columns with spirals.
While column ductility can be increased through confinement, it has a negligible impact on the ultimate column strength. For design purpose, the above minimum confinement requirements for HSC which have been revised in ACI 318 were applicable. However, even though the effect of lateral reinforcements on structural behavior of columns is significant, for simplicity, their contributions were not taken into account in this study.

2.7 Concrete Modulus of elasticity and Modulus of rupture

2.7.1 Modulus of elasticity

Modulus of elasticity is defined as the ratio of normal stress to corresponding strain for tensile or compressive stresses below the proportional limit of a material. It is a key factor influencing the structural performance of reinforced concrete structures and is particularly important as a design parameter in predicting the deformation of tall buildings, lateral stiffness of columns and it is a fundamental factor for determining modular ratio, n, which is used for the design of section of members subjected to flexure. The modulus of elasticity is strongly influenced by the concrete materials especially the aggregate and proportions used. It increases with increase in concrete strength, however, at somewhat lower rate. Many design codes and researchers have established several empirical equations for predicting the elastic modulus of concrete [11]. Based on the property of modulus of elasticity of concrete that it is proportional to the square root of compressive strength in the range of normal concrete strength, the American code ACI 318 specified the elastic modulus Ec as $4700\sqrt{f'_c}$ and Indian code IS as $5000\sqrt{f'_{ck}}$ MPa for normal weight concrete.
However, these formulas do not make any distinction between high-strength and lower-strength concretes. In view of test results from Cornell University indicating that the moduli of elasticity of concrete with very high levels of strength might be lower than values given by the ACI 318 provision, the following formula proposed by Cornell researchers was endorsed by ACI Committee 363R-10 state of the art report [28].
($Ec = 3{,}320\sqrt{f'c} + 6{,}900$ MPa for 21 MPa $< f'c <$ 83 MPa)

2.7.2 Concrete Modulus of Rupture

The modulus of rupture of concrete represents its flexural tensile strength. It affects cracking moment for concrete members, and therefore, influences the minimum flexural reinforcement that is required to prevent sudden failure of the member under flexural loads. It also affects strain limits in prestressed concrete members. There is wide variability in the values of the modulus of rupture reported in literature. The value reported by various investigators for the modulus of

rupture range from 0.33 to 1.0√f'c MPa. For HSC, some literatures suggest values for the modulus of rupture in the range of 0.62√*f'c* to 0.97√*f'c* MPa [25].
According to ACI 318, the modulus of rupture of normal-weight concrete is (fr = 0.62√f'c).
Again, this formula does not make any distinction between high-strength and normal-strength concretes, although there are indications from Cornell research that the constant relating fr and f'c might be higher for concretes with very high strength levels. Accordingly, ACI Committee 363R-10 state of the art report recommends fr = 0.94√f'c for 21 MPa < f'c < 83 MPa [5].
From the above formulas, the following values are obtained for concrete grades between 30 and 90MPa which are used in this study.

Concrete grades f_{cu} (MPa)	Equivalent cylindrical strength of concrete f_{ck} or fc' (MPa)	Modulus of elasticity Ec (MPa)	Modulus of Rupture fr (MPa)
30	24	23,164.6	4.605
50	40	27,897.5	5.945
70	56	31,744.6	7.034
90	75	35,652.043	8.141
100	83.33	37,207.254	8.581

Table 2-3: computed values of Ec and fr from the corresponding concrete strength

2.8 Transmission of column loads through floor system of HSC column

It has been proposed in this study that beam/slab concrete strength to be used is NSC of C30 and kept constant for all models while columns concrete strength vary from C30 to C90. Based on the ratios of columns concrete strength to that of beam/slab concrete strength, the load transmission of the frame system could be affected. Accordingly, ACI 318 [4] recommends if fc′ of a column is greater than 1.4 times that of the floor system, transmission of load through the floor system shall be provided by the following three guidelines.

i) Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 0.6m into the slab from face of column. Column concrete shall be well integrated with floor concrete. It requires the placing of two different concrete mixtures in the floor system. The lower-strength mixture should be placed while the higher-strength concrete is still plastic and should be adequately vibrated to ensure the concretes are well integrated. This requires careful coordination of the concrete deliveries and the possible use of retarders. In some cases, additional inspection services will be required when this procedure is used. It is important that the higher-strength concrete in the floor in the region of the column be placed before the lower-strength concrete in the remainder of the floor to prevent accidental placing of the low-strength concrete in the column area. It is the responsibility of the licensed design professional to indicate on the drawings where the high- and low-strength concretes are to be placed.
ii) Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.
iii) For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength

plus 35 percent of floor concrete strength. In this case, the ratio of column concrete strength to slab concrete strength shall not be taken greater than 2.5 for design.

In this study, transmission of column loads through floor system were assumed to be based on one of the above guidelines, since all the f_c' of HSC considered for columns are greater than 1.4 times the f_c' of NSC of the slabs and beams.

CHAPTER THREE

3 Analysis and Design of Sample Building Frames

3.1 Description of Sample building models

Rectangular four 12-storey and four 18-storey building models having five bays in the X-direction and three bays in the Y-direction were considered. The structural plan layout and 3D ETABS model are shown in figure 3.1. The bay dimensions are the same in all models having 5.5m,5m,5m,5m,5.5m in the X-direction and 5.5m,5m,5.5m in the Y-direction. The entire story's height is assumed to be 3m. These are very common class of regular buildings, whose structural systems generally consists of a rectangular lattice-work of columns and beams (the frame), together with the relatively rigid floor slabs. The four models for each 12 and 18-storey buildings are all the same in overall dimensions and geometry with difference only in all columns concrete strength. Table 3-1 describes how the building models are designated.

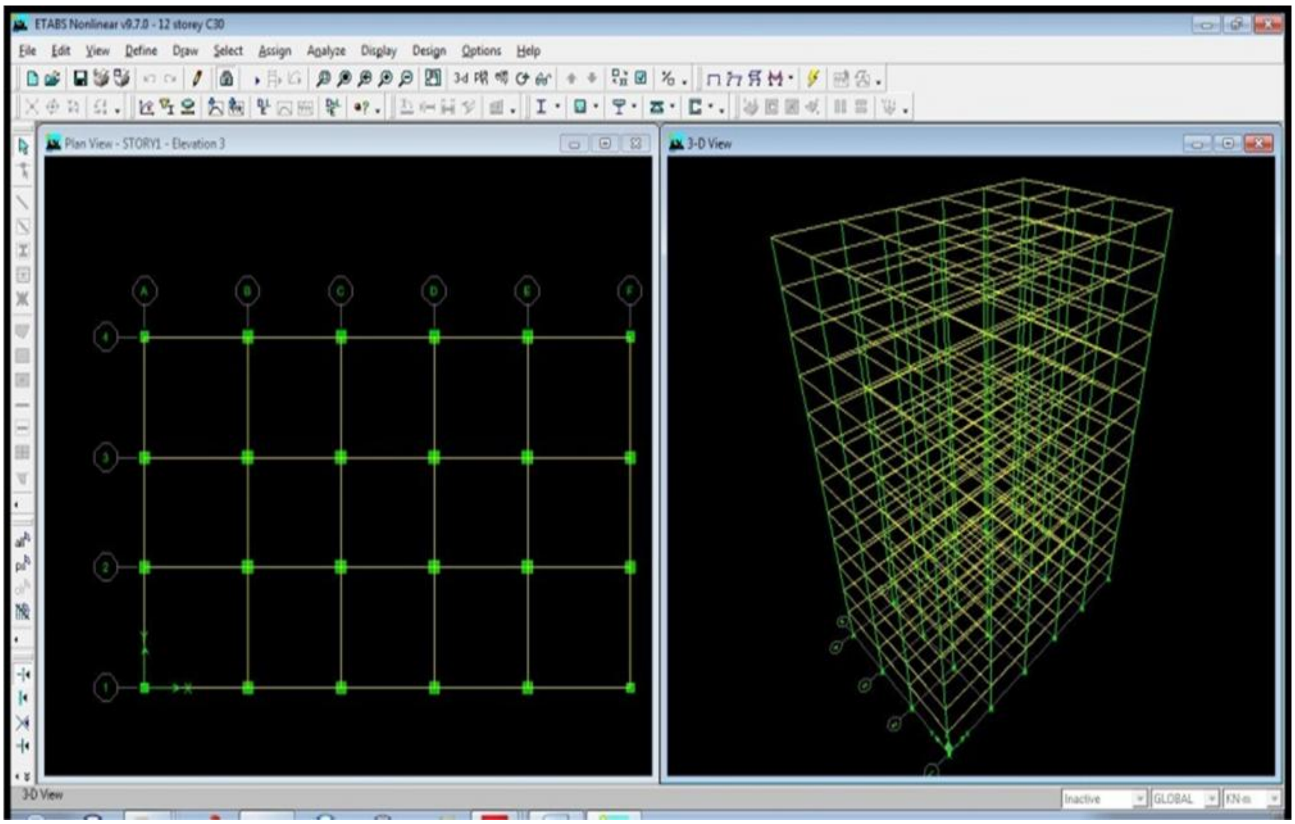


Figure 3.1: Structural plan layout and 3D ETABS model of the building.

Building models	Concrete strength used for all beams & slabs	Concrete strength used for all columns	Frame model designation
12-storey	C30	C30	FC30
	C30	C50	FC50
	C30	C70	FC70
	C30	C90	FC90
18-storey	C30	C30	FC30
	C30	C50	FC50
	C30	C70	FC70
	C30	C90	FC90

Table 3-1: Building frames designation

The modelings of the building frames for analysis have been done by ETABS v9.7.0 (fig.3.1) and the model contains the following features:

- ✓ The geometry, connectivity, member types and sizes for the building are correctly defined in the computer model.
- ✓ All other relevant data, group properties and characteristics are correctly defined.
- ✓ All restrains are defined so that the stiffness of the building members correctly transfers the vertical and lateral loads.

Assumptions in structural modeling:

i. The sizes of the beams are uniform throughout the height of the buildings, while the sizes of the columns change every four and six storeys for 12 and 18-storey respectively. That is columns size from base to 6^{th} floor, from 7^{th} to 12^{th} floor and from 13^{th} to 18^{th} floor are of the same size for the 18-storey buildings for each of frame model designated in table 3-1 and similarly, columns size from base to 4^{th} floor, from 5^{th} to 8^{th} floor and from 9^{th} to 12^{th} floor are of the same size for the 12-storey buildings for each of the frame model labeled in the same table.
ii. Stiffness of floor slabs is very small relative to beams and columns of the frame, so that their contribution to overall structural stiffness can be ignored.
iii. The frames have been modeled as rigid frames.

3.2 Building's columns and stories categorization

To simplify the design of columns, the stories of each frame are categorized into three levels as Lower storey, Middle storey and Upper storey. Columns are grouped as Corner columns (C1), Exterior columns (C2) and Interior columns (C3). Based on this categorization, columns designations are described in tables 3-2.

Columns category	Stories category	No of floors	Columns nomination	No of columns
Corner columns (C1)	Lower storey	Base-6^{th}	CL1	4*6=24
	Middle storey	7^{th} -12^{th}	CM1	4*6=24
	Upper storey	13^{th} -18^{th}	CU1	4*6=24
Exterior columns (C2)	Lower storey	Base-6^{th}	CL2	12*6=72
	Middle storey	7^{th} -12^{th}	CM2	12*6=72
	Upper storey	13^{th} -18^{th}	CU2	12*6=72
Interior columns (C3)	Lower storey	Base-6^{th}	CL3	8*6=48
	Middle storey	7^{th} -12^{th}	CM3	8*6=48
	Upper storey	13^{th} -18^{th}	CU3	8*6=48

i) 18-storey building model

Columns category	Stories category	No of floors	Columns nomination	No of columns
Corner columns (C1)	Lower storey	Base-4^{th}	CL1	4*4=16
	Middle storey	5^{th} -8^{th}	CM1	4*4=16
	Upper storey	9^{th} -12^{th}	CU1	4*4=16
Exterior columns (C2)	Lower storey	Base-4^{th}	CL2	12*4=48
	Middle storey	5^{th} -8^{th}	CM2	12*4=48
	Upper storey	9^{th} -12^{th}	CU2	12*4=48
Interior columns (C3)	Lower storey	Base-4^{th}	CL3	8*4=32
	Middle storey	5^{th} -8^{th}	CM3	8*4=32
	Upper storey	9^{th} -12^{th}	CU3	8*4=32

ii) 12-storey building model

Table 3-2: Category and nominations of columns

3.3 Loading and Load Combinations

The loading on the model buildings have been done according to EBCS for the general building category. Accordingly, in addition to the building's member self-weight, the following gravity loads are considered for the analysis of the frames.

Roof: 0.5KN/m^2 (flat roof assumed).

Floors: 1.8KN/m^2 imposed dead load and 3KN/m^2 live load.

Perimeter beams: 12.5KN/m imposed dead load which accounts for wall cladding.

Since the buildings are assumed to be located in high seismic area zone 4 as per EBCS 8, the lateral forces from earthquake and wind are obtained using UBC1997 and ASCE 7-05 based methodology incorporated in ETABS respectively.

To determine the design values of the effect of actions, the following load combinations are used in the analysis.

COMB 1: 1.3DL + 1.6LL

COMB 2: 0.8(1.3DL + 1.6LL + 1.6WL)

COMB 3: 0.8(1.3DL + 1.6LL – 1.6WL)

COMB 4: 0.75(1.3DL + 1.6LL) + EQx

COMB 5: 0.75(1.3DL + 1.6LL) – EQx

COMB 6: 0.75(1.3DL + 1.6LL) + EQy

COMB 7: 0.75(1.3DL + 1.6LL) – EQy

3.4 Assumptions and Analysis considerations

3.4.1 Bases for the behavioral difference between HSC and NSC

Based on the results of different design codes and publications, this analysis and design has taken into account the major behavioral differences between NSC and HSC. These differences include the following:

- The stress-strain relationship of HSC is almost linear throughout loading as compared to NSC which is significantly more nonlinear at high stresses.
- The ascending and descending portions of the stress-strain curve are approximately steeper for HSC in contrast with NSC which can sustain significantly larger strains beyond the peak stress.
- HSC is more brittle and unpredictable than NSC.

These differences are incorporated into the analysis and design through the modification of the rectangular stress block parameters.

3.4.2 Properties of proposed rectangular stress block

To account for the near linear stress-strain relationship and nearly triangular stress distribution of HSC as shown in Figure 2.3, an equivalent rectangular stress block parameters α_1 and β_1 are modified. Accordingly, the values of α_1 0.85,0.85,0.78 and 0.756 are used for the corresponding C30,C50,C70 and C90 based on the computed values in table 2-2 of ACI318-11 and that of Ibrahim and Macgregor. Similarly, values of β_1 0.85, 0.85, 0.81 and 0.76 are employed for the corresponding C30, C50, C70 and C90. Any contribution of the concrete tensile stress is neglected. Even though the ACI and some other building codes uses 0.003 as a limiting maximum concrete strain and some other codes including EBCS uses 0.0035, many experimental investigations show that the maximum concrete strain decreases as concrete strength increases even though the strain at maximum stress is slightly higher for HSC. Taking into considerations this proposed trend, the values of $\epsilon_{cu}=0.0035$ for concrete strength C30 and C50 and $\epsilon_{cu}=0.003$ for C70 and C90 are considered in the analysis and design of the column sections.

3.4.3 Materials

Concrete: The 28 days specified characteristic cube compressive strength (f_{cu}) of C30, C50, C70 and C90 according to EBCS-2 [12] grading of concrete strength is used where the number in the designations is in MPa.

The design concrete strength (F_{cd}) is computed from the formula:

$$F_{cd} = \alpha f_{ck}/1.5$$

Where : α is the corresponding values of α_1 in the above section;
f_{ck} is the characteristics cylindrical compressive strength of concrete which is $f_{cu}/1.25$.
1.25 is the conversion factor.
1.5 is the partial safety factor for concrete.

Accordingly, the corresponding values of F_{cd} for C30, C50, C70 and C90 are 13.600, 21.333, 29.120 and 37.800 in MPa respectively.

Modulus of elasticity of concrete (Ec):- the values of Ec in Table 3 are used in the analysis for each corresponding concrete grade.

The differences in the stress-strain curve of the different concrete strength were reflected through the modification of the rectangular stress block parameters as discussed under section 2.5.

Reinforcing steel: High yield steel of S-460 with the design strength f_{yd} of 400MPa both in tension and compression is chosen. The modulus of elasticity Es=200GPa.

3.5 Concrete member Stiffness requirement

In building structures, the flexural stiffness reduction of beams and columns due to concrete cracking plays an important role in the nonlinear load-deformation response of reinforced concrete structures under service loads. Since concrete cracking amplifies the lateral deflection of the building, it is important to appropriately model the cracked stiffness of the beams and columns when analyzing a moment frame building as this stiffness determines the resulting building periods, base shear, story drifts, and internal force distributions of the building.

The main parameters affecting the stiffness's of the cracked concrete elements are modulus of elasticity (E_c) and the effective moment of inertia (I_e). The recommendations for these parameters vary significantly mainly due to different interpretations of test data and different behavior models [25]. The equation recommended and used for this study for modulus of elasticity (Ec) was described in section 2.8 and the corresponding values in table 2-3 for each concrete strength have been used in the analysis.

3.5.1 Effective moment of inertia (I_e)

The cracking of concrete is a dominant component in RCC buildings response. The member moment of inertia, I, used in the analysis should incorporate the degree of cracking, as tension cracks are inevitable when the imposed loads produce bending moments in excess of the cracking moment. Thus, the concept of effective moment of inertia (I_e) to reflect the concrete cracking was developed by some investigators and the equations and modification factors recommended in literature as well as in different country standards to introduce the non-linearity of concrete were applicable.

For this study, the recommendation by ACI 318-11, the effective moment of inertia (I_e) values for beams and columns at ultimate limit state were introduced as follows [4].

For Beams.. 0.35 I_g

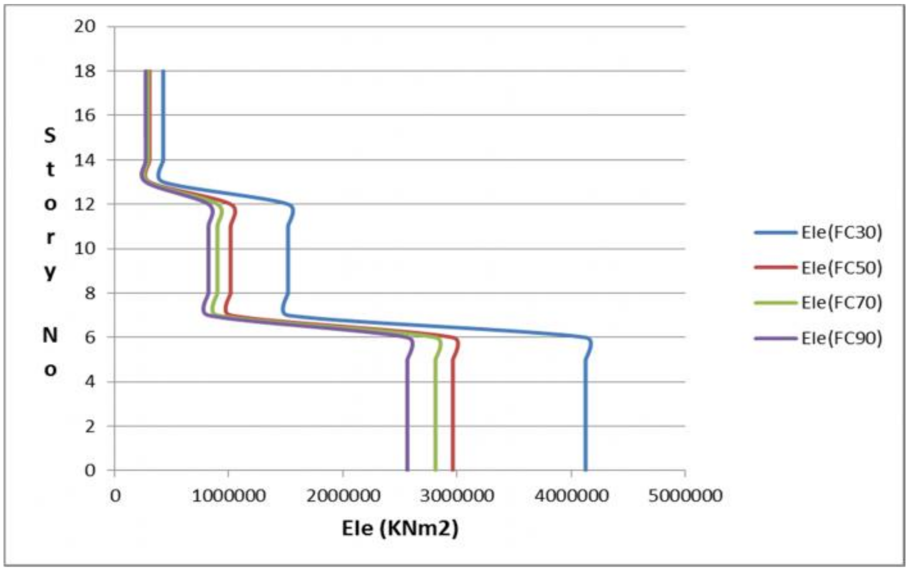
For Columns... 0.70 I_g

Where I_g is the gross member moment of inertia.

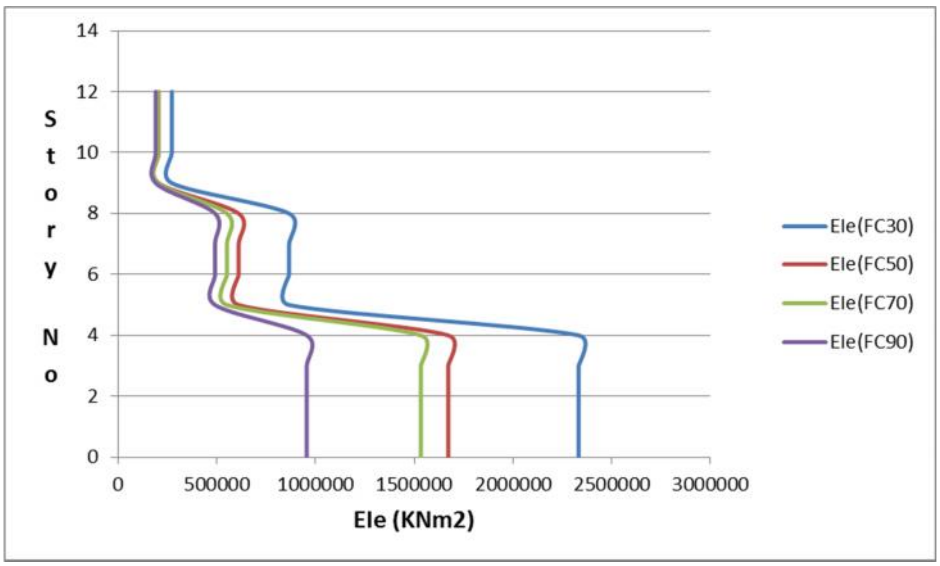
Based on this, the stiffness reduction factor of 0.35 for beams and 0.7 for columns were adopted in the analysis.

3.5.2 Effective stiffness of columns and stories (E_cI_e)

The effective stiffness of members which determines the overall stiffness of the structure directly affects buildings lateral deformation. For HSC members, the stiffness increases as it is directly related to the modulus of elasticity of concrete. On the other hand, HSC members have less cross-sectional dimensions than NSC members. The lesser cross-sectional dimensions give smaller moments of inertia which is the major parameter for computing effective stiffness of the element as well as the overall structure. In this study, the contribution of both parameters, modulus of elasticity (E_c) and moment of inertia (I) to obtain the effective stiffness (E_cI_e) of the stories have been analyzed and the moment of inertia (I) was obtained to be the most influencing parameter. Therefore, building models designed by higher columns concrete strength were obtained to have smaller stories stiffness. Accordingly, the stories of FC30 are the stiffest and that of FC90 are the least. For each frame model and corresponding stories, the effective stiffness were computed and the comparative results have been shown graphically in figure 3.2 for both 18-storey and 12-storey building models. The details were tabulated in table A 3-1of appendix A.



i) 18-storey



ii) 12-storey

Figure 3.2: Graphical illustration of stories effective stiffness (E_cI_e)

3.6 Structural analysis

The structural analysis are carried out to determine, deformations (displacements and drifts) of the frames and the column reaction forces (Axial force and Bending moments) by the linear elastic-static analysis using commercial package ETABS. Since the model buildings are assumed to be located at high seismic area, the Uniform Building Code 1997(UBC97) which is very similar with EBCS 8, 1995 was used for the equivalent static analysis of seismic load. The following conditions and corresponding parametric values are considered in the analysis.

- Seismic zone factor is 0.4 which is high seismic area (equivalent with zone 4 as per EBCS 8, 1995).
- Importance factor I=1.2 for building category II.

- Soil profile type S_D which is equivalent to soil type B of EBCS 8, 1995.
- Building period (T) computed from the following formula [14].

  $$T_1 = C_1 H^{3/4}$$

  Where: C_1=0.075 (for RC frames)

  H is height of the buildings above base in m.

  Accordingly, T= 1.1 and 1.5 in seconds are defined for the buildings model of twelve and eighteen story respectively.

For the case of wind load analysis, ASCE 7-05 was employed with exposure from extents of rigid diaphragms. The coefficients employed are:

- Wind speed 50mph (equivalent to 22m/s as per EBCS 1, 1995)
- Exposure type C (equivalent with type II as per EBCS 1,1995)
- Importance factor I =1
- Topographic factor = 1
- Gust factor = 0.85
- Directionality factor = 0.85
- Windward coefficient Cp=0.8
- Leeward coefficient Cp=0.5

In the structural analysis of the model buildings, the columns cross-sectional dimensions are assumed depending on story levels, columns category and columns concrete strength while that of the beams are based on the location i.e whether they are perimeter or interior beams. Accordingly, all the model frames are analyzed under the same loading. The actions effect (Axial load and Bending moments) were an outputs of the analysis and used for the design of columns. These outputs were selected for the governing load combination. The governing load combination in this case was found to be COMB 7 which was expected because, the buildings location was assumed to be in high seismic area and thus earthquake load governs. In addition, the building's plan geometry is shorter in the Y- direction which was therefore less stiff than the other direction and made the earthquake load to be higher in this direction. The detail analysis frame columns cross-sectional assumptions and output results are tabulated in table A 3-2 in appendix A.

3.7 Design of columns

The governing load combination COMB 7 was due to earthquake load acting in the negative Y-direction with the gravity load. The distributions of these lateral seismic forces along the elevation of the buildings are linear with minimum value at base and maximum value at the top floor. The contributions of seismic forces acting on a moment frame to the axial load at interior columns might be lesser than that of exterior and corner columns. Thus, attention should be given to the axial load in the exterior and corner columns because the seismic forces might be large in comparison with the gravity loads. For this study, all the three columns categories were designed for each frame model to investigate the effect of concrete strength on the structural behavior of building frame. Notice that the comparison study is intended only for the columns of the buildings. Therefore, detail structural design of beams and slabs were not computed. But, beams cross sectional dimensions have been checked for strength and obtained to be satisfactory while slabs were only modeled as shell components to provide lateral rigidity to the buildings.

3.7.1 Design procedure

Columns are compression members which are integral parts of a structure but which are considered to be isolated for design purposes. Thus, the limit state design procedures described in EBCS-2 [12] for design of isolated columns have been followed for each column designed in this study. Accordingly, the limits of slenderness have been checked for all sections to be designed and all have been found to be not slender. Therefore, the second-order effects were ignored for all sections. The design moments (M_{xd} and M_{yd}) were then obtained from the total eccentricity which includes first-order eccentricity from the first-order moments and an additional eccentricity recommended by the code. The columns were thus designed under biaxial bending with an axial force (P) and design bending moments M_{xd} and M_{yd}. EBCS-2, part 2 [13] biaxial design charts and a design chart prepared for HSC by Dr.Girma Zerayohannes [17] are used to provide reinforcements and optimum cross-sections.

In the design process, the effects of different parameters, such as steel ratio (ρ), longitudinal steel distributions and shape of columns cross-section are minimized by considering constant properties throughout the study. Accordingly, a constant reinforcement ratio between 1.65 to 1.69% was considered as this value is recommended for economical design. Most of the column sections are designed for ρ value of 1.67%. Since square column sections are intended to be designed, uniform distributions of longitudinal steel were selected on four faces for all sections. Therefore, the design was carried out by changing the sections as necessary until the values of the provided steel ratios (ρ) between 1.65 to 1.69% were maintained. The design results of the required dimensions and steel reinforcement of the columns have been displayed in table A 3-3 in appendix A.

3.8 Result discussions

3.8.1 Axial load level

The axial load level is one of the variables affecting curvature ductility of reinforced concrete structures. It can be defined as the ratio of applied axial load (P) to the ultimate section axial load capacity (P_u). As the axial load level on reinforced concrete column increases, a smaller curvature is achieved and as a result, the ductility capacity of the column progressively reduces. Under this investigation, as column concrete strength increases, the axial load level was observed to decrease. This was predominantly due to increase in column section capacity (P_u) since the difference between the applied axial loads (P) for the same column and different concrete strength was insignificant. For both 18-storey and 12-storey building models, the axial load levels were reduced as frame columns concrete strength increases. In other words, columns with more HSC had less axial load level (columns of FC90 in this case). The detail numerical comparison was tabulated in table A 3-4 in appendix A. It can be observed that, reduction in axial load level brought improvement to curvature ductility and reduced strength which were discussed under section 3.8.2.

3.8.2 Moment-curvatures

One of the ways in which building codes ensures ductility level in columns is by specifying the moment-curvature relation of columns. Thus, the designed column sections were analyzed for the computation of moment-curvature relations. All the column sections are symmetrically reinforced as shown in Figure 3.3 and all have a breadth b equal to total depth h of square section with reinforcement provided at depth d, d_2, d_3 for sections of Figure3.3 (a) and at depth d, d_2, d_3, d_4 for sections of Figure 3.3(b). Since concrete cover primarily depends on exposure to earth or weather, concrete cover of 25mm on all faces was considered. Then, for a given concrete axial deformation (ϵ_{cm}), the neutral axis depth(c) was assumed and the strains for each layer of steel were computed. Stresses in steel were then calculated by multiplying the strains with the modulus of elasticity of steel. Concrete stresses are obtained from the modified equivalent rectangular stress block by employing the two parameters α_1 and β_1 for the corresponding concrete strength. The limiting value of the maximum steel stress both in tension and compression was the yield stress of steel which was 400MPa.The corresponding forces in steel and concrete which satisfy the equilibrium conditions were predicted by an iterative procedure. The associated section resisting moments and curvatures for each level of concrete deformation (ϵ_{cm}) were computed. The procedure is repeated by changing the concrete strain ϵ_{cm} until the ultimate limit state is reached.

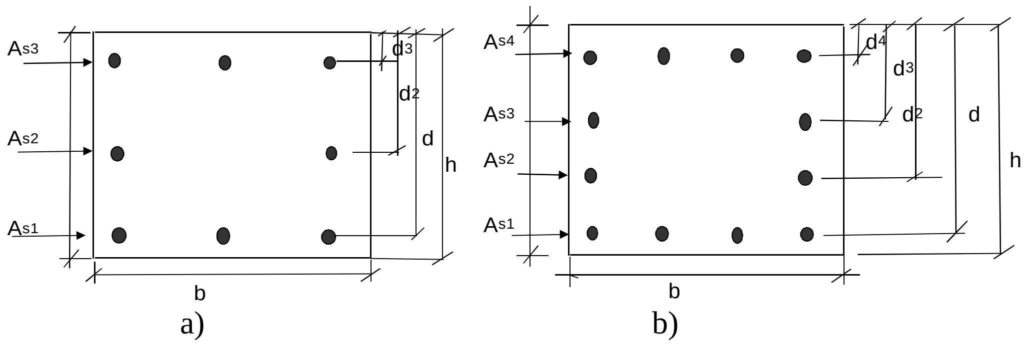


Figure 3.3: Designed column sections and longitudinal reinforcement distributions

The ultimate limit state is reached when either of the following conditions met.

- The strain in the extreme compressive concrete fiber reaches the ultimate value.
- The strain in steel reaches the ultimate value.
- The bending moment is limited to a post peak value of $0.8M_{max}$.
- The strain in the extreme compressive concrete fiber reach the peak strain in case of pure axial loading.

In all the sections analyzed for the moment-curvature, the ultimate limit state was found to reach for the bottom outer most layer steel strain which implies all the designed column sections were under reinforced and therefore, ductility was ensured. But the level of this ductility was obtained to be affected by variation in concrete strength.

3.8.2.1 Sample moment-curvature computation

The design results of column CL1 of 18-storey FC90 due to applied combined axial load and bending moments are illustrated.

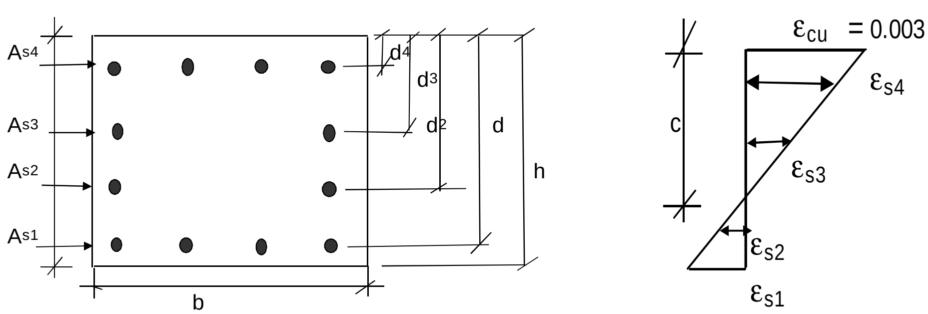


Figure 3.4: Sample designed section and assumed strain distributions

Specimen properties:

Column ID: CL1

Section properties: b=h=385mm, d_4=40mm, d_3=142mm, d_2=243mm, d=345mm

Concrete: C90, f_{cd}=37.8MPa, ϵ_{cu}=0.003

Steel: f_{yd}=400MPa, Es=200GPa,

A_{s1}=4Φ16 (=804.248mm^2),

A_{s2}=2Φ16 (=402.124mm^2),

A_{s3}=2Φ16 (=402.124mm^2),

A_{s4}=4Φ16 (=804.248mm^2)

Equations for steel strain:

$\epsilon_{s1} = \frac{c-d}{c}(0.003)$ $\epsilon_{s3} = \frac{c-d3}{c}(0.003)$

$\epsilon_{s2} = \frac{c-d2}{c}(0.003)$ $\epsilon_{s4} = \frac{c-d4}{c}(0.003)$

The maximum (ultimate) steel strain $\epsilon_{s,max}$= 0.01 and the yield strain ϵ_{yd} = 0.002 was considered.

Steel stresses (fs):

$fs_1 = Es^* \epsilon_{s1}$ $fs_2 = Es^* \epsilon_{s2}$ $fs_3 = Es^* \epsilon_{s3}$ $fs_4 = Es^* \epsilon_{s4}$ where $fs_i \leq \pm$ 400MPa

Forces in Concrete:

From equivalent rectangular stress block,

a=βc where β = 0.763 for C90

$C_c = abf_{cd}$

Forces in Steel:

$Fs_1 = fs_1{*}A_{s1}$ $Fs_2 = fs_2{*}A_{s2}$ $Fs_3 = fs_3{*}A_{s3}$ $Fs4 = fs_4{*}A_{s4}$

Sign conventions:

a) Compressive stress in concrete is taken as positive.
b) Compressive stress (fs_3 and fs_4) and strain (ϵ_{s3} and ϵ_{s4}) in compression reinforcement are positive.
c) Tensile stress (fs_1 and fs_2) and strain (ϵ_{s1} and ϵ_{s2}) in tension reinforcement are positive.

Notice that the labeling as compression and tension reinforcement here depends on the stresses carried at particular instant.

Then, for a value of ϵ_{cm}, assume a neutral axis depth (c) and iterate by changing the assumed neutral axis depth (c) until force equilibrium met. Then, the moment about neutral axis for each force was computed. The algebraic sum of the moments (M) was obtained. The corresponding curvature (Φ) was then Φ=ϵ_{cm}/c.

For this sample, table 3-3 shows the values obtained from the computation.

Concrete strain (ϵ_{cm})	Neutral axis depth (c) in (m)	Bottom outer most layer steel strain (ϵ_{s1})	Sum of moments (M) in (KNm)	Curvature (Φ)
0.00015	0.03231	0.0014515	86.938	0.0046419
0.0003	0.04521	0.0019893	107.893	0.0066356
0.00045	0.04937	0.0026946 (ϵ_{yd})	112.294	0.0091147 (Φ_y)
0.0006	0.05179	0.0033971	114.347	0.0115859
0.00075	0.05276	0.0041545	114.621	0.014216
0.0009	0.05363	0.0048902	114.901	0.0167831
0.00105	0.05441	0.0056078	115.186	0.0192979

0.0012	0.05512	0.0063103	115.476	0.0217691
0.00135	0.05528	0.0070759	115.913	0.0244229
0.0015	0.05481	0.0079417	116.501	0.0273672
0.00165	0.05437	0.0088207	117.065	0.03035
0.0018	0.05394	0.0097124	117.605	0.0333693
0.00195	**0.05354**	**0.0106158 (>$\epsilon_{s,max}$)**	**118.119**	**0.0364226 (Φ_u)**
0.0021				
0.00225				

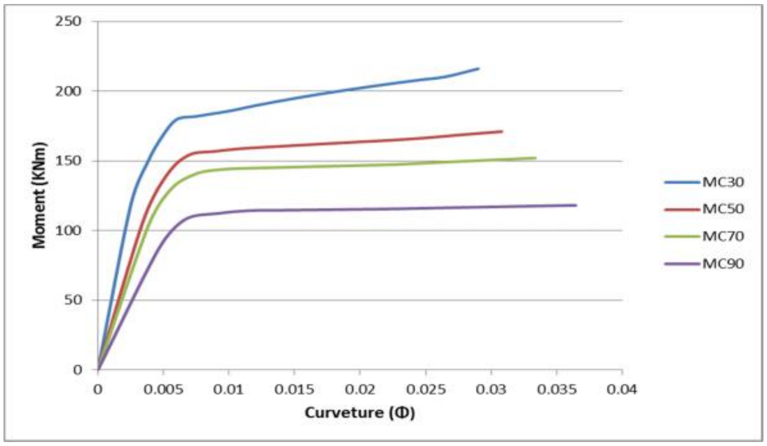
Table 3-3: Sample moment (M) and Curvature (Φ) values computed

For this sample, the ultimate limit state was reached for an assumed value of ϵ_{cm} =0.00195 where the bottom outer most layer steel strain (ϵ_{s1}) was greater than the limiting maximum value. The corresponding ultimate moment and curvature were **118.119KNm** and **0.0364226rad/m.** Similar procedures and techniques have been employed using excel sheet for the rest of designed column sections and the following points are observed for all.

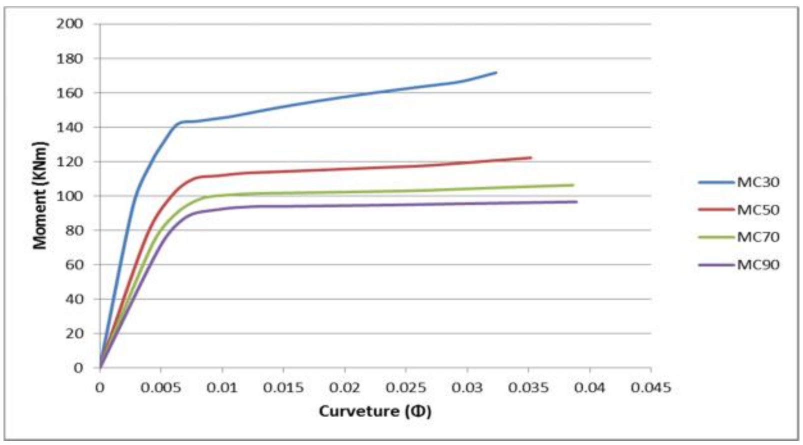
- All layers of tensile steel strain keeps increasing with curvature.
- The neutral axis depth (c) gradually increases as curvature increases until all layers of tensile steel strain reaches the yield value. However, after yielding of all layers of tensile steel strain, it starts to decrease as curvature keeps on increasing until outermost layer tensile steel strain reaches maximum value which is an ultimate value.
- The ultimate value reached at lower value of concrete strain (ϵ_{cm}) as column concrete strength increases.
- The moment increases at faster rate until the outermost layer of tensile steel strain reaches the yield value. But, after entering the post-yield stage, it becomes gradual and keeps on increasing until the ultimate value. This can be also observed from the graph of moment-curvature curves illustrated in the next section.

3.8.2.2 Graphs of moment-curvature

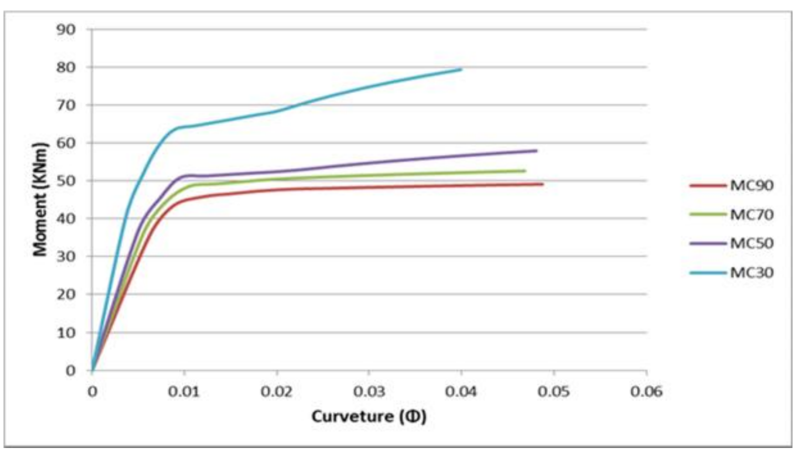
For all designed column sections, moment-curvature relations were computed and the graphs are plotted. From each of the graph of figure 3.5 and 3.6, difference in moment-curvature curve properties were observed for the same column type due to variation in column concrete strength. But similar curve trend exists between the different columns for the different concrete strength of both 18-storey and 12-storey frame models. Some of the graphs for some column types were illustrated in Figure 3.5 for 18-storey and in Figure 3.6 for 12-storey and all the rests were shown in figure B 3.1 and B 3.2 in appendix B.



a)

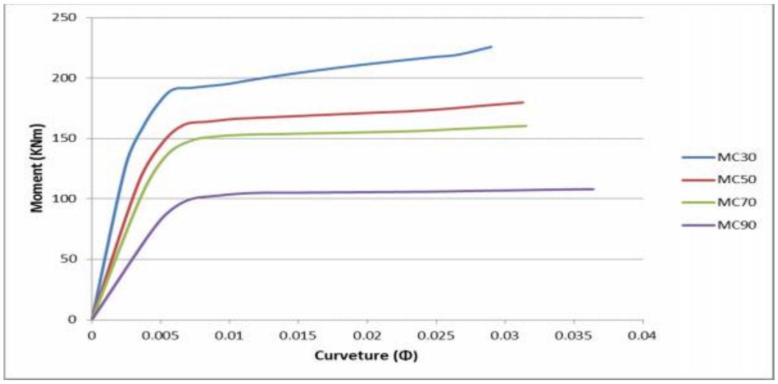


b)

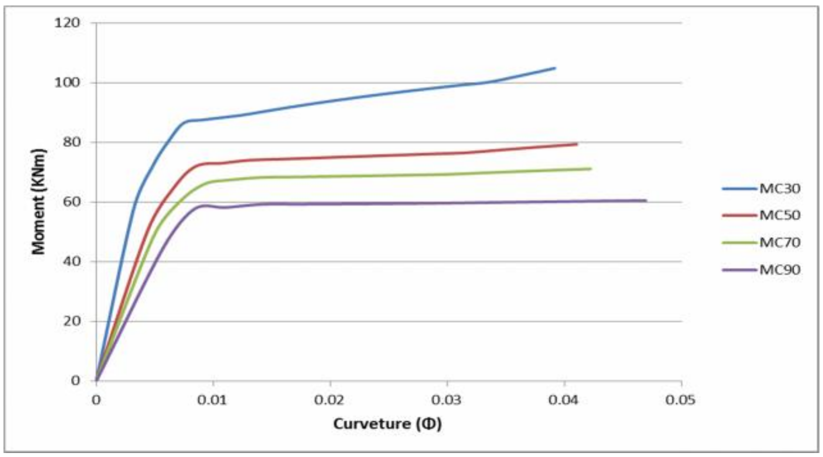


c)

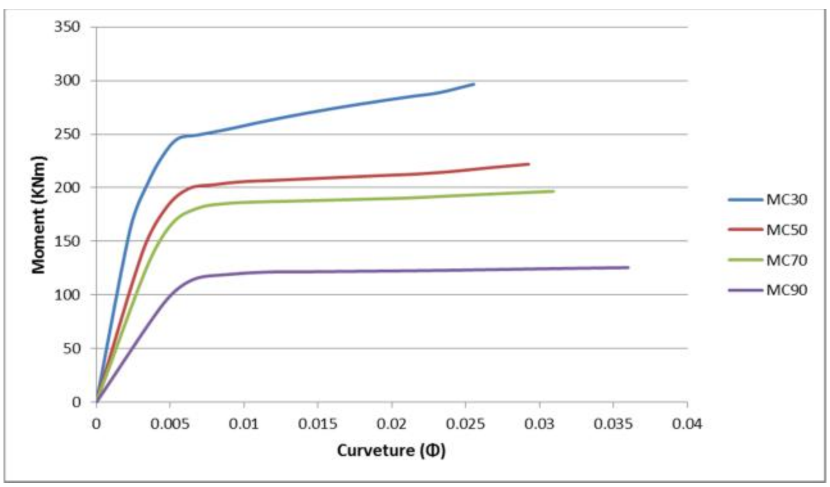
Figure 3.5: Graphs of Moment-Curvature curves for columns of 18-storey. (a) Column CL1, (b) Column CM2, (c) Column CU3



a)



b)



c)

Figure 3.6: Graph of Moment-Curvature curves for columns of 12-storey. (a) Column CL2, (b) Column CM2, (c) Column CL3

Figure 3.5 and 3.6 show the effect of concrete strength on the ultimate range moment-curvature curves for the different column sections designed of different concrete strength. Each family of curves follow slightly different slope initially until yielding of the outermost tensile steel. The slope or steepness increases with increasing effective stiffness of the columns, which is from higher to lower column concrete strength. With further increase in curvature and after yielding of the outermost tensile steel, stiffness degradation with slope reductions are observed. Column sections with higher concrete strength have a plateau moment-curvature curve with larger but comparable ultimate strength to that of yield strength and the ultimate strengths for the NSC are significantly larger than the yield strength. These indicate the stiffening of the NSC column sections after yielding of the outermost tensile steel. The length of plateau which is the measure of ductility is slightly greater for the higher strength concrete columns. Therefore, it can be concluded that, the use of HSC column lead to smaller section and a reduced longitudinal reinforcement which has led to a lower flexural strength and slightly higher curvature at failure as we can observe from the graph. In other words, the concrete strain at ultimate was slightly less and the neutral axis depth was significantly smaller for higher strength concrete columns.

In conclusion, from the moment-curvature curve relations, it can be observed that the curvature ductility slightly enhanced with higher concrete strength of the columns. However, the decrease in flexural strength of the section with the increase in concrete strength is noticeable. In addition, the ultimate strength is substantially higher than the yield strength for NSC. Therefore, it can be noted that as column concrete strength increases, the ultimate strength becomes comparable to the yield strength of the section. Thus, in this study, frames with HSC column have got lower stiffness and performed well in satisfying ductility demand.

3.9 Lateral deformation and Interstorey Drift Analysis

The design of high rise frame building under lateral load is usually governed by serviceability criteria. Serviceability of the buildings has to be checked under lateral loads by evaluation of the lateral frame deformations. Serviceability limit states which define functional performance and behavior under load which has been used to prevent damage to collateral building materials, such as cladding and partitions, and also to control the perception of building motion. Various national building codes tend to specify similar, although not identical, limits on horizontal building displacements for medium and high-rise buildings. These lateral building deflections are evaluated as total building drift or as interstory drift which are explained as follows [24].

Lateral frame deflections are usually evaluated for the buildings,

i. as a whole, where the applicable parameter is *total building drift,* defined as the lateral frame deflection at the top-most occupied floor (Δ) divided by the height from grade to the uppermost floor (Δ/H); and
ii. for each floor of the building, where the applicable parameter is *interstory drift,* defined as the lateral deflection of a floor relative to the one immediately below it divided by the distance between floors ($(\delta n - \delta n-1)/h$).

In such serviceability check, typical values of the limit specified by some building codes to these parameters (commonly called *drift index*) are H / 100 to H / 600 for total building drift (i above) and h / 200 to h / 600 for interstory drift (ii above) depending on building type and materials used. In this particular study, the limits given by UBC 1997 and EBCS-8 for interstorey drift have been taken into consideration.

In accordance with UBC 1997 code, for the buildings with fundamental period, T less than 0.7 seconds, the inelastic drifts are limited to a maximum 0.025 times the storey height (h). For buildings with natural periods 0.7 seconds or greater, the limitation for interstorey drift is 0.020 times the storey height (h) [24]. On the other hand, the limitation for interstorey drift given by EBCS-8 [13] for buildings having non-structural elements fixed in a way as not to interfere with structural deformations is 0.015 times the storey height. Since the values of period, T in this study were 1.5 and 1.1 seconds for 18-storey and 12-storey model respectively, and the storey height (h) is 3m, the limitation for interstorey drift is 60mm and 45mm according to UBC 1997 and EBCS-8 respectively.

These lateral deformations or drifts in building frames are a result of flexural and shear mode contributions, due to the column axial deformations, beam deformations and beam-column joint deformations. Therefore, the structural analysis must then capture all significant components of potential frame deflections which consist of:

1. Flexural deformation of beams and columns.
2. Axial deformation of columns.
3. Shear deformation of beams and columns.
4. Beam-column joint deformation.
5. Effect of member joint size.
6. P-Δ effect.

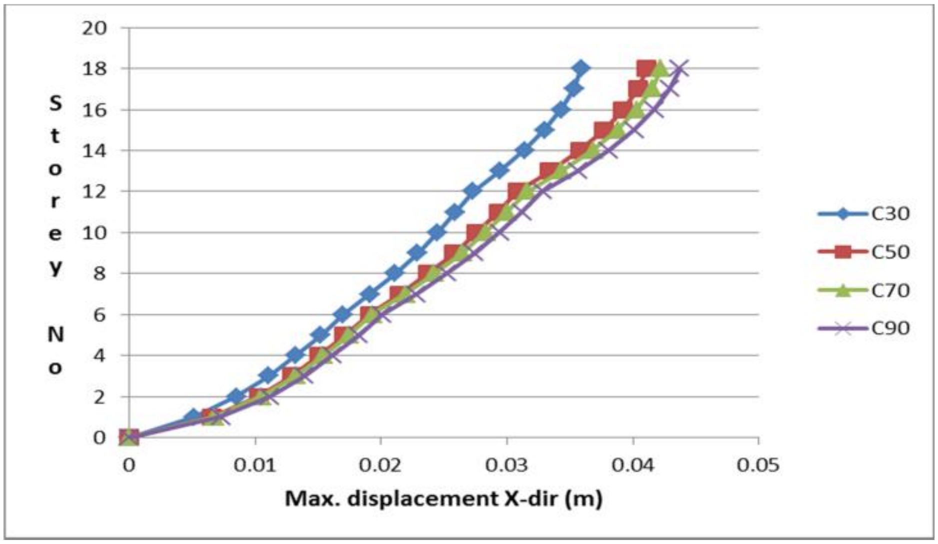
The behavioral knowledge of each of the above effects on frame deflection has to be sufficiently understood to permit a reasonably accurate prediction of the contribution to the total response.

In medium to high rise structures, the higher axial forces and deformations in the columns, and the accumulation of their effects over a greater height, cause the flexural component of displacement to become dominant [24].

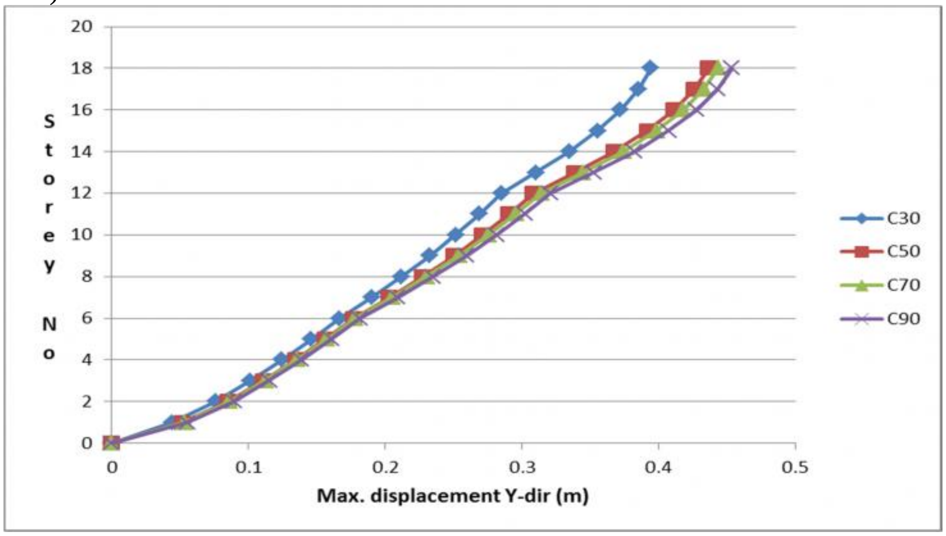
Computer programs and analytical models are now within reach of most engineers to afford consideration of all of the above effects. One of such computer programs is ETABS which was used in this study for analysis of the building models. The frame deflection responses (output from the ETABS) which consider the contributions of the above mentioned frame deflections have to be compared for the four frame types with different columns concrete strength. The influences of building height and column bay geometry on frame deflection were minimized by keeping them constant for each frame model. As a result of analysis, the X-direction and Y-direction deflections (story maximum displacements and maximum interstorey drifts) due to COMB 7 (the governing load combination) were taken for comparison.

3.9.1 Story maximum displacements

The graphs in figure 3.7 represent the results for maximum displacements at all storey levels in x-direction and y-direction for both models and for all the frame types due to the governing lateral load. As it can be observed from the maximum displacement graphs of 18-storey, maximum displacements of stories increase as the frame column concrete strength increases (i.e. from FC30 to FC90). This can be explained as reduced stiffness of the stories as a result of reduction in columns sizes because, displacement is directly affected by stiffness of the members. The difference is significant at upper stories than the lower stories between frame with NSC column (FC30) and the other frames with HSC column as it can be observed from the graphs. For instance, at 18th and 1st stories for the Y-direction, displacements were increased by 59.5mm and 11mm respectively for FC90 compared with FC30. But the incremental differences are insignificant between the HSC column frames.



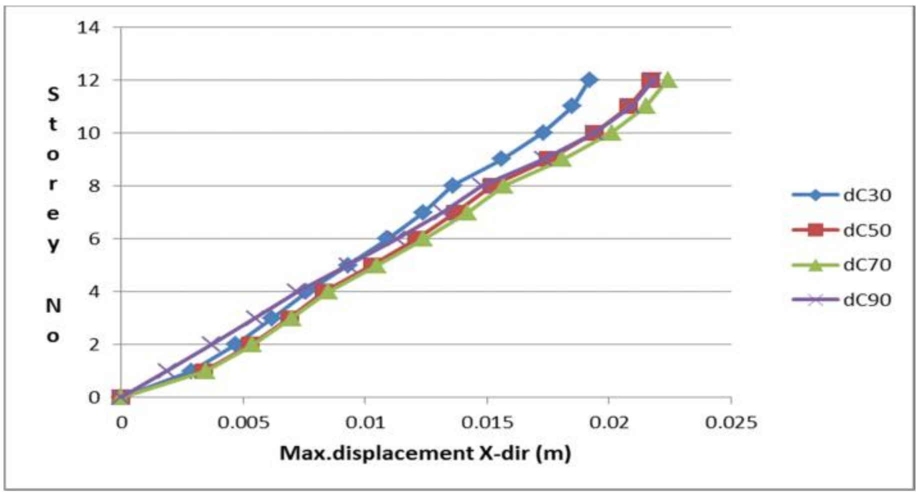
a) X-direction



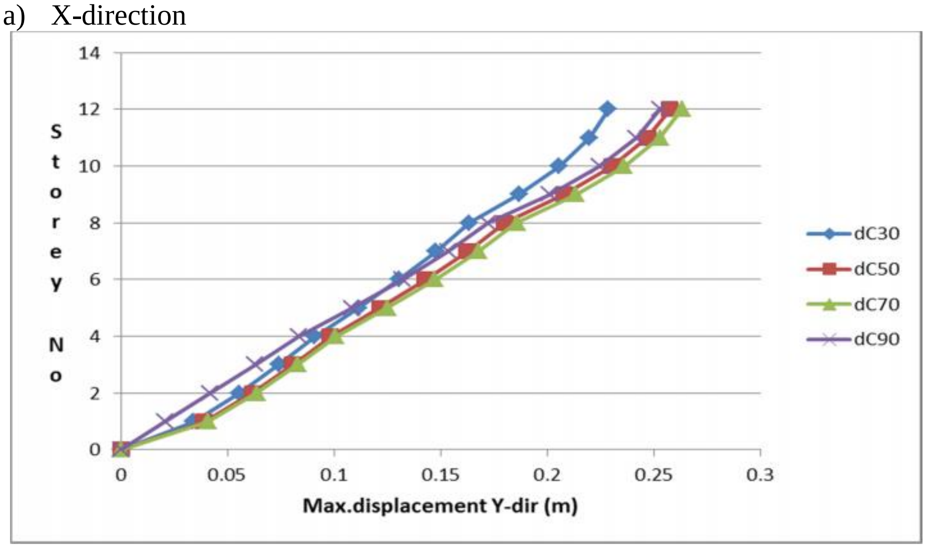
b) Y-direction

Figure 3.7: Graphs of maximum displacement for 18-storey

For 12-storey models, like that of 18-storey, maximum displacements of upper stories increase as the frame column concrete strength increase (i.e from FC30 to FC90). But in this case, FC90 has less maximum displacements for all stories compared to that of FC50 and FC70. As we can observe from the graphs of figure 3.8, less maximum stories displacement are obtained for FC90 below 5th story in comparison to all the other frames. This shows the contribution of HSC column in resisting the frame lateral displacement is more readable for the 12-storey frames. Here, FC70 is the frame with largest maximum stories displacement for all stories. In this case, at 12th and 1st stories for the Y-direction, displacements were increased by 34.5mm and 6.7mm respectively for FC70 compared with FC30. But the incremental differences at upper stories are insignificant between the HSC column frames.



a) X-direction



b) Y-direction

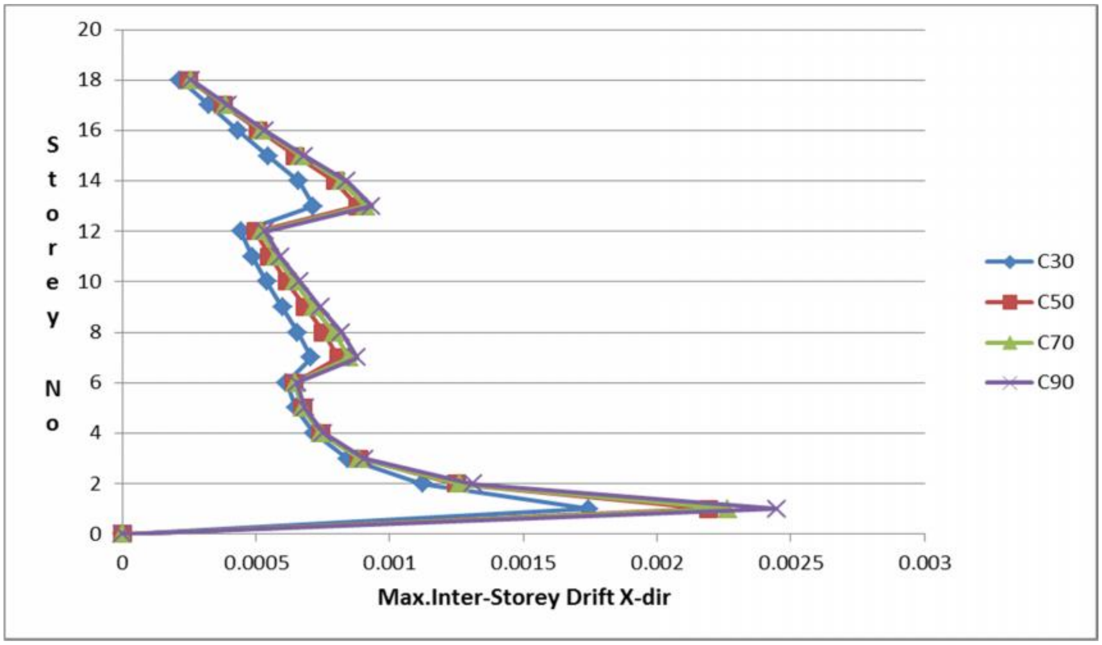
Figure 3.8: Graphs of maximum displacement for 12-storey

3.9.2 Maximum interstorey drift

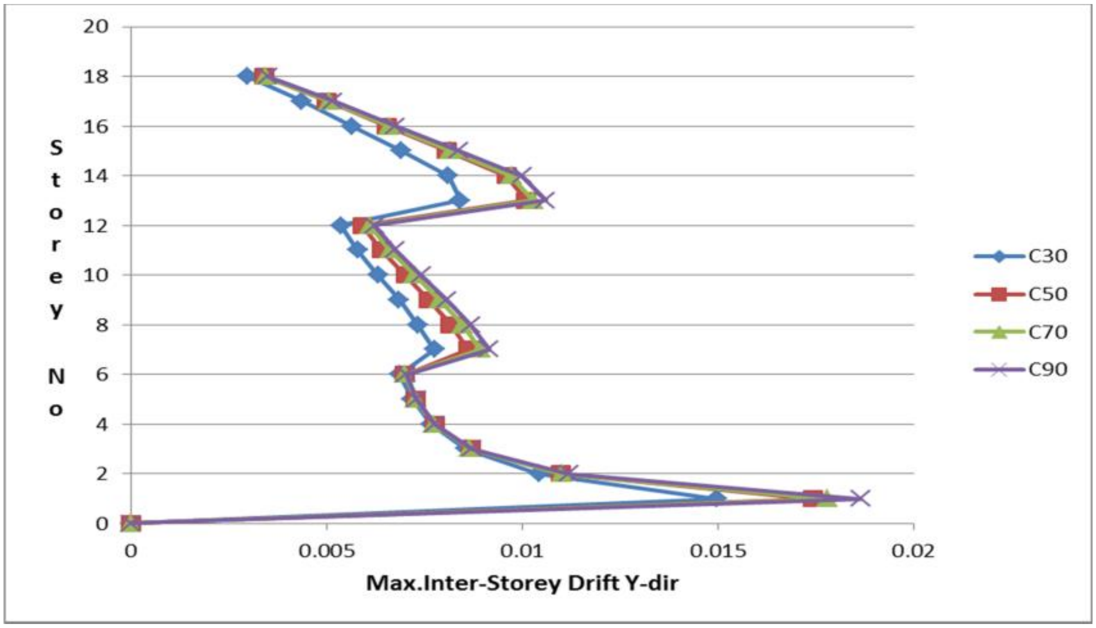
The inter-storey drifts at all levels are not exceeding the interstorey drift limits described in section 3.9 above. This result mean that the horizontal movement of columns joint are below the limitation and acceptable for design purposes even for high seismic load.

From the graphs in figure 3.9 and 3.10, it can be observed that at the same level, the values of interstorey drift increases slightly as column concrete strength increases. This could be due to the higher the columns concrete strength, the lesser is the stiffness because of smaller columns section dimensions. Since maximum displacements of each story level of each frame type was caused by the action of lateral load to the building, it is the same result for interstorey drift. For instance, maximum values of interstorey drifts in the y-direction for 18-storey frames are 14.973, 17.435, 17.802, 18.642mm respectively for each frame type FC30, FC50, FC70 and FC90. Comparing these interstorey drift values with that of FC30, the increamental differences are 16.44%, 18.89% and 24.5% respectively for FC50, FC70 and FC90 frames at 1st story. The maximum interstorey drifts were occurred at 1st story for all cases of frame type and for both x and y directions except FC90 of 12-storey which has at 9th story. The corresponding reductions in storey effective stiffness (E_cI_e) as compared to that of FC30 are 28.18%, 31.88% and 37.84% respectively for FC50, FC70 and FC90 frames for the 1st story. Thus, it can be said that the interstorey drift for the higher columns concrete strength of the frame increases relatively less than the decrease in columns effective stiffness.

For 18-storey, as we can observe from the graphs, the interstorey drift decreased until the 6th story where there is change of story columns cross-section dimensions. For the middle level stories, it starts from maximum at 7th floor and decreased until 12th story again where there is change of columns cross-section dimensions. Finally, for the upper level stories, it decreased from 13th floor to 18th story. These show that the relative lateral displacements (drifts) for both x and y direction decreases toward the top for each story level and it is minimum at 18th story. The rapid shift in interstorey drift curves between story 6th and 7th as well as between 12th and 13th indicate that the MRF interstorey drift variation is more sensitive to the changes in story stiffness of columns with respect to lateral displacements. The same behavior was observed for 12-storey frames except maximum drift occurred at 9th story for FC90 frame but at 1st story for the others. This is due to greater relative displacement between 9th and 8th stories than any other stories for this particular frame. It is also noticed that the 1st story drift for FC90 frame was smaller than that of the others with larger for FC70 frame.

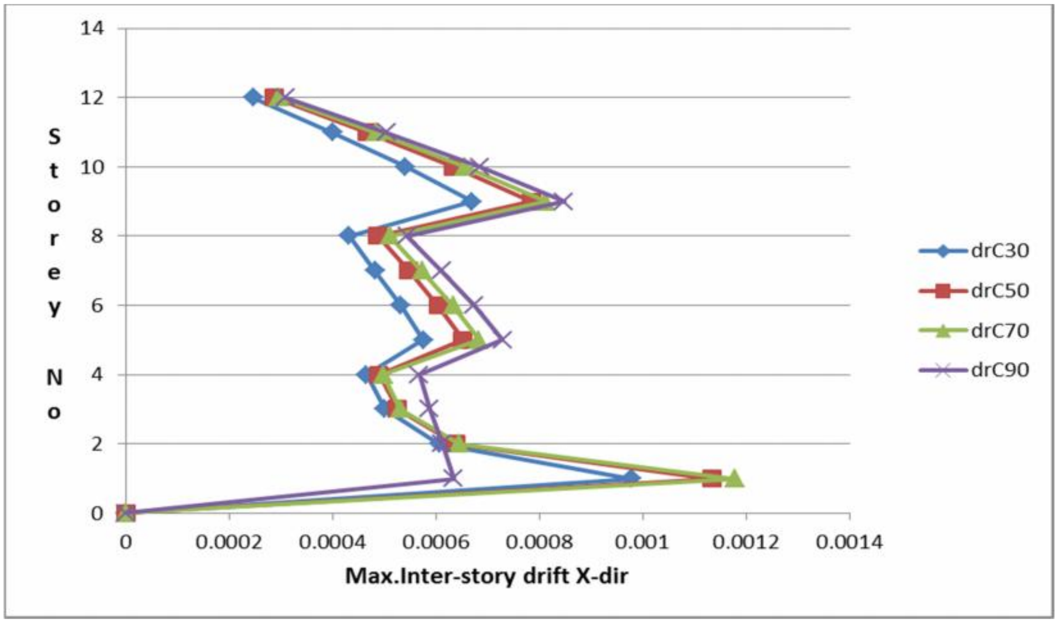


a) X-direction

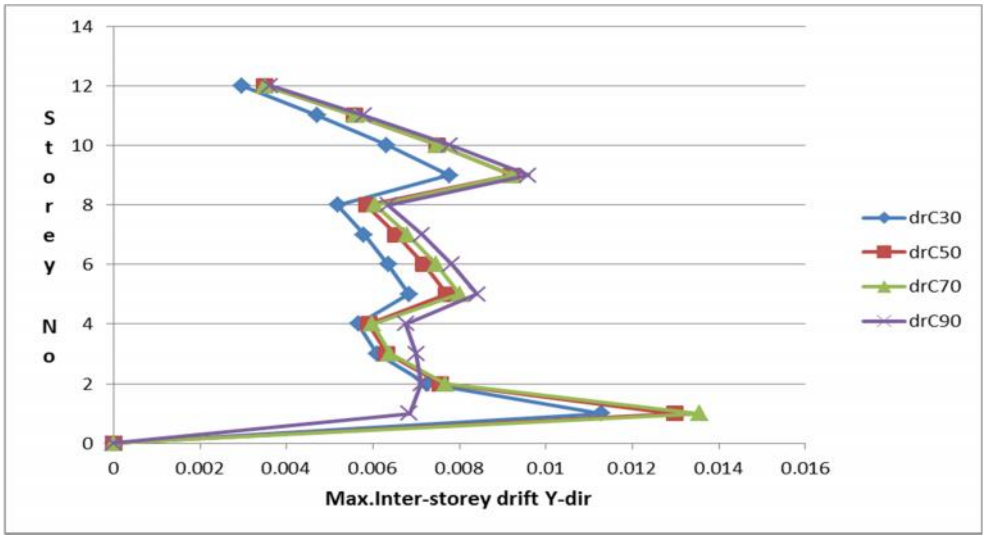


b) Y-direction

Figure 3.9: Graphs of maximum inter-storey drift for 18-storey frame models



a) X-direction



b) Y-direction

Figure 3.10: Graphs of maximum inter-storey drift for 12-storey frame models

In conclusion, building structure must satisfy serviceability limit states in addition to strength limit state in which functional performance of the building should be given consideration. This is because, exceeding a serviceability limit state in a building usually means that its function is disrupted or impaired because of local minor damage, deteriorations, or because of occupant discomfort. The present study result shows that even though lesser stiffness due to smaller section dimensions are obtained, the maximum interstorey drifts are within the limit but slightly higher for frame with HSC columns. Therefore, the contributions of the HSC column in resisting the lateral deformation of the frame and its structural performances have been obtained to be substantial.

3.10 Economic analysis

Reinforced concrete is a composite building material produced by a combination of concrete and steel. In designing reinforced concrete elements, the most important factor is the strength and durability properties of concrete and steel materials especially in multi-story buildings.

Rationally, using high-strength concrete in columns achieves economical and technical advantages. The main economic advantages of high strength concrete are its higher strength per unit cost, strength per unit weight and stiffness per unit cost. The unit price of concrete increases relatively less than the increase in available strength. This gives high strength concrete to be economical in terms of strength per unit cost [8]. Moreover, the unit weight of concrete only increases slightly with increasing compressive strength. This gives significant advantage in seismic areas, since earthquake forces are directly proportional to the weight of the structure. The results of the present study confirms the premise that using HSC for building columns resulted in reduced weight and hence reduced seismic lateral loads to stories. Accordingly, the analysis result for the distribution of seismic lateral load along the elevation of the model frames shows reduction of lateral loads to stories as frame's columns concrete strength increases. These comparisons are shown in table 3-4 which presents seismic lateral load reduction of the HSC columns frames with respect to the NSC columns frame (FC30) of 18-storey frame models in percentage.

Stories category	Seismic lateral load reduction of HSC frames with respect to FC30 in (%)			Remark
	FC50	FC70	FC90	The reduction values become more significant toward the lower stories
Upper stories	2.66	3.42	4.12	
Middle stories	4.20	5.60	6.62	
Lower stories	5.50	7.12	8.70	

Table 3-4: comparison of reduction in seismic lateral load to stories for HSC columns frames with respect to FC30.

In this study, in addition to the lesser seismic lateral load to stories, reducing total columns cost and increasing the usable floor space would be the essential benefits of utilizing HSC in columns. In construction projects, overall cost of the structure consists of several factors such as, direct costs, indirect costs, initial cost, maintenance, insurance and any possible costs of repair during its design service life. These are certainly a function of materials cost, labor cost and the time necessary for erection cost, of which materials cost have the largest percent.

3.10.1 Materials quantity and cost estimation

The quantity and cost comparison involves only the three main materials that are needed for reinforced concrete construction; concrete, reinforcing steel and formwork.

On the premise that relatively less unit price than the increase in compressive strength of concrete, only comparison in terms of quantity has been done for columns concrete but for longitudinal reinforcing steel and columns concrete formwork, cost estimation were included. Concerning the cost, unit cost for the HSC is difficult to obtain since the use of HSC in building projects was not practiced in Ethiopia. The cost of longitudinal steel was based on the prevailing prices of commercially available grades in Addis Ababa. For the formwork, unit cost was based on the current prices in practice by some engineering consultancy firms in the country. Quantity estimates were analyzed in cubic meters for concrete, kilograms for steel and square meters for formwork. The materials and cost estimates for the different model buildings are summarized in Table 3-5. The detail materials quantity computations for concrete, longitudinal steel and formwork have been tabulated in table A 3-5 to A 3-8 in appendix A.

Frame model	Column Concrete Quantity (m^3)		Percentage reduction of columns concrete quantity compared to FC30 (%)	
	12-storey	18-storey	12-storey	18-storey
FC30	146.888	289.959	0.00	0.00
FC50	113.667	221.187	22.62	23.72
FC70	102.203	199.531	30.42	31.19
FC90	83.927	181.537	42.86	37.39

a) Comparisons of columns concrete quantity

Frame model	Longitudinal Reinforcing steel Quantity (Kg)		Percentage reduction of columns steel quantity compared to FC30 (%)		Cost estimation (birr)		
	12-storey	18-storey	12-storey	18-storey	Unit rate (birr/Kg)	12-storey	18-storey
FC30	24722.128	42516.672	0.00	0.00	30	741663.84	1275500.16
FC50	18794.144	32296.56	23.98	24.04	30	563824.32	968896.80
FC70	16879.008	29229.528	31.73	31.25	30	506370.24	876885.84
FC90	12884.896	26590.08	47.88	37.46	30	386546.88	797702.40

b) comparisons of quantity and total costs for longitudinal reinforcement

Frame model	Columns formwork area required (m^2)		Percentage reduction of columns formwork compared to FC30 (%)		Cost estimates (birr)		
	12-storey	18-storey	12-storey	18-storey	Unit rate (birr/m^2)	12-storey	18-storey
FC30	1214.72	2085.41	0.00	0.00	130	157613.60	271103.30
FC50	1069.96	1820.83	11.92	12.69	130	139094.80	236707.90
FC70	1017.84	1728.23	16.21	17.13	130	132319.20	224669.90
FC90	926.69	1648.85	23.71	20.93	130	120469.70	214350.50

c) comparisons of quantity and total costs for columns formwork

Table 3-5: summary of materials quantity and cost estimations for the different building models (a, b, and c)

It is noted that the volume of concrete and the area of formwork decrease with increasing concrete strengths. This is expected because, the columns sizes decrease with increasing concrete strengths. The quantified numerical difference and percentage reductions of columns formwork

as concrete strength increases are shown in table 3-5 (c). These numbers are significant and hence the reduction in column size has important part in maximizing the useable floor space. This comparison of floor space can be obtained from table A3-8 in appendix A which shows the total space occupied by columns of the different frames types. For instance, the total space covered by columns of the lower stories of FC30 is $51.03m^2$ and that of FC90 is $32.49m^2$ for the 18-storey frames. The difference gives floor space gain due to the use of HSC for the columns and it is $18.54m^2$ which is equivalent to adding a square floor area with 4.3m width.
The amount of longitudinal steel reinforcement also decreases with increasing column concrete strength. There is a substantial differences in the amount of steel reinforcement between frames with the normal strength concrete columns (FC30) compared with the other frames with high strength concrete columns. For instance, comparing FC30 and FC90 for the amount of steel, 47.88% and 37.46% reductions are obtained respectively for 12-storey and 18-storey frames. These can be attributed to the fact that in frame FC30, the high axial loads have been resisted by a combination of the concrete and a large amount of steel reinforcement. On the other hand, in the high strength concrete columns frame FC90, the high axial loads have been resisted primarily by the concrete and small amount of steel reinforcement. It can be concluded that the contribution of longitudinal reinforcement in resisting the high axial load of medium to high rise building becomes smaller as columns concrete strength increases. The percentage reduction values for amount of steel above also holds true for reduction in steel cost. This emphasizes the economic advantage of using high strength concrete columns in medium to high rise buildings.

3.10.2 Results of cost comparison

The total costs and the corresponding savings for steel and formwork of the different building models are summarized in table 3-6.

Frame model	Total costs (in thousands birr)				Savings compared to FC30 (in thousands birr)			
	Steel		formwork		Steel		formwork	
	12-storey	18-storey	12-storey	18-storey	12-storey	18-storey	12-storey	18-storey
FC30	741.664	1275.500	157.614	271.103	0.00	0.00	0.00	0.00
FC50	563.824	968.897	139.095	236.708	177.840	306.603	18.519	34.395
FC70	506.370	876.886	132.319	224.670	235.294	398.614	25.294	46.433
FC90	386.547	797.702	120.470	214.351	355.117	477.798	37.144	56.753

Table 3-6: Summary of total costs and the corresponding savings for steel and formwork

The unit cost of concrete components slightly increases with increasing concrete strength but the total costs of the HSC columns are generally less than that of normal strength concrete columns.
It can be observed that the use of HSC for columns of medium to high rise building results in substantial cost savings from both steel and formwork. Even though the cost saving contributions of concrete and formwork are significant, this cost comparison suggests that the cost effectiveness of a HSC building depends to a large extent on the cost component attributable to the steel reinforcements. From table 3-6 for instance, only from steel, 477,797.76 birr will be

saved if FC90 instead of FC30 18-storey is to be constructed. Therefore, in this cost analysis, the most cost effective building model was FC90 and the least was FC30.
In conclusion, it has been shown that the use of HSC columns in medium to high rise building not only saves the valuable floor area, but also resulted in the most economical column corresponding to the use of sufficient amount of longitudinal steel with the highest available concrete compressive strength.

3.10.3 Effect of columns stiffness variation on beams moment in frame

The magnitude of bending moments in beams and columns of frame depend upon their relative rigidity in which beams are made of the same dimensions in all floors, while the dimensions of columns vary from storey to storey. Columns have smallest dimensions at the top and largest dimensions at the bottom. Due to this reason, the ratio of the rigidity of the beam to that of the column is larger in the upper floors than in the lower floors. The span bending moments in the beams increase with decrease of the rigidity of the columns, while the support bending moments in them increase with the increase in the rigidity of the columns. Due to this, the span bending moments are largest in the upper stories where the columns are least rigid and the support bending moments are maximum in the lower stories where the columns are rigid [12]. In this study, since stories were classified as lower, middle and upper based on variation in columns dimensions, these behaviors have been observed for all the frames under consideration in which maximum beams support bending moments are highest at the lower stories as shown in table 3-7.

On the other hand, increasing the columns concrete strength has reduced the stiffness of the columns despite increased modulus of elasticity because of reduction in size as explained in section 3.5. It has been tried to investigate if the reduced stiffness of columns for the higher concrete strength might have an effect on distribution of moments between beams and columns. Accordingly, exterior and interior beams maximum negative moments at different story levels for the four frame models have been compared. The result shows insignificant difference in maximum negative beam moments for frames FC30, FC50, FC70 and FC90 having beams of the same dimensions with the same concrete strength but variation in columns concrete strength which resulted in varying columns stiffness. Table 3-7 shows these results for some of exterior and interior beams at selected stories.

Frame models	Beam location	Story	Beam ID	Maximum support moment (KNm)			
				FC30	**FC50**	**FC70**	**FC90**
12-Story	Exterior beam	1st	1A-1B	50.094	49.478	49.161	42.132
			2A-3A	156.956	154.575	155.463	113.150
		5th	1A-1B	43.416	42.731	41.801	40.037
			2A-3A	104.723	102.282	101.212	98.038
		9th	1A-1B	36.983	36.669	36.447	36.536
			2A-3A	71.283	71.580	67.457	68.296
	Interior beam	1st	2A-2B	10.065	10.000	10.075	9.297
			2B-3B	88.082	88.111	86.962	59.368
		5th	2A-2B	9.389	9.078	9.278	9.555
			2B-3B	54.320	52.709	52.408	53.700
		9th	2A-2B	8.082	7.845	7.793	7.787
			2B-3B	34.776	32.811	35.292	35.107
18-Story	Exterior beam	1st	1A-1B	61.868	63.345	63.350	62.291
			2A-3A	209.797	211.053	210.517	211.538
		7th	1A-1B	46.841	46.133	45.758	43.098
			2A-3A	129.437	126.120	124.766	124.548
		13th	1A-1B	38.682	37.562	37.713	36.125
			2A-3A	108.657	109.146	106.446	108.257
	Interior beam	1st	2A-2B	11.638	11.794	11.807	11.884
			2B-3B	124.061	127.031	126.677	127.789
		7th	2A-2B	10.825	10.418	10.407	10.267
			2B-3B	69.444	68.345	67.953	67.829
		13th	2A-2B	10.268	9.616	9.442	9.319
			2B-3B	59.097	58.409	59.410	58.164

Table 3-7: Maximum beam moments for the different frames for selected beams

It can be concluded from these result that for a given loading, the variation in relative stiffness of beams to columns due to variation in columns dimensions along the stories have significant effect on maximum beams support bending moments. But the effects are insignificant for similar frames with difference in columns concrete strength which resulted in variation of columns stiffness. That is frames with identical beams stiffness but varying columns stiffness due to variation in columns concrete strength develops almost equal beams support moments at the joints. Therefore, in this thesis the economic influence of beams design for the frames with different columns concrete strength are observed to be insignificant.

CHAPTER FOUR

4 Conclusion and Recommendations

4.1 Conclusion

This study was conducted to investigate the effect of using HSC column on the structural behavior of regular rectangular models of medium to high rise frame buildings subjected to seismic lateral load in addition to normal use gravity loads. The structural behaviors; level of ductility and interstorey drifts were the main focus of the study. To clearly visualize the effect, the numbers of variables were limited to variation in columns concrete strength and optimizing the columns cross sectional dimensions. Columns longitudinal steel; yield strength, ratios and distributions were kept constant for all frame models to minimize their effect. In addition, square column sections were utilized for all the frame models. The properties of the HSC class were incorporated in the analysis and design of the columns based on the proposed equations by ACI and some researchers. The frames analysis was done using ETABS and columns were designed based on the EBCS-2 column design procedures using excel spread sheet.

Columns moment- curvature curves were developed to look into the ductility levels of the different concrete strength columns. It was found that frames with HSC columns have got lower stiffness and performed well in the level of columns ductility. The maximum stories displacement and interstorey drifts have been obtained from the analysis output and graphical comparison were made between the frames with varied columns concrete strength. The result showed that the maximum interstorey drifts are within the limit and slightly higher for frames with HSC columns, but the contribution of the concrete strength in resisting the lateral deformation was obtained to be substantial. Economic comparisons were also made and it was found that the most economical frame corresponds to the highest available columns concrete strength which uses small but sufficient amount of longitudinal reinforcement.

The following additional observations are noted:

- Curvature ductility slightly enhanced with higher concrete strength of the columns.
- Flexural strength of the column section decreases with the increase in column concrete strength.
- The ultimate strength is substantially higher than the yield strength for NSC column section, but it becomes comparable to the yield strength of the section as column concrete strength increases.
- The columns concrete deformation at ultimate decreases with increasing columns concrete strength.
- Lesser stories stiffness due to smaller column sections dimensions are obtained for frames with higher column concrete strength.

- Maximum displacements of stories increased as the frame column concrete strength increases.
- The maximum interstorey drifts were within the limit, but slightly higher for frame with HSC columns.
- The cost effectiveness of a HSC columns building depends to a large extent on the cost component attributable to the steel reinforcements.
- The use of HSC for columns of medium to high rise building frame contributed to increased floor space.

From the study under taken, the above resulted conclusions are indications of structural qualities of the HSC over that of the NSC in columns of medium to high rise frame buildings. Therefore, for the increasing demand on land use in large cities like Addis Ababa, the observations and conclusions made in this thesis could help to bring the HSC into applications especially for columns of high rise buildings which will be advantageous in structural response in addition to significant economic save.

4.2 Recommendations

The study undertaken is intended for regular frame buildings representing medium to high rise structures. The assumptions made and the study approach limited the number of variables that could affect the structural behavior of HSC. Therefore, with different approach which could consider different variables, further research regarding structural responses of reinforced high strength concrete columns of medium to high rise buildings is essential.

The MRF structural system was considered for the model buildings representing medium high rise structures under the present study. But, if the number of stories are to be increased beyond the limitation for MRF or for very high rise buildings, further research could also possible which utilizes different structural system.

The structural performance and economic advantages of HSC are attracting designers to use HSC for columns of buildings. Thus, even though the design of HSC members preceded the research to be undertaken, the importance of continued research on HSC at the structure level is crucial in providing sufficient knowledge of the performance of HSC structures so that the safety of design practices will be ensured.

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APPENDIX A

Concrete grade	Column type and corresponding effective stiffness (E_cI_e) in KNm^2								
	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
C30	84448.866	175124.376	210797.86	34587.064	63174.497	77897.917	9557.251	18063.755	21468.951
Storey E_cI_e	**4125670.856**			**1519625.556**			**426745.672**		
C50	60986.725	123633.351	154409.873	23062.863	41653.757	53097.312	6881.755	13181.569	15028.941
Storey E_cI_e	**2962826.096**			**1016875.032**			**305937.376**		
C70	57619.623	117594.696	146104.522	20659.068	36620.571	47397.862	6673.032	12217.227	14999.324
Storey E_cI_e	**2810451.02**			**901266.02**			**293293.444**		
C90	44950.667	108608.275	135171.149	17702.854	33608.664	43551.329	6240.557	11700.430	13497.475
Storey E_cI_e	**2564467.86**			**822526.016**			**273347.188**		

a) 18-storey

Concrete grade	Column type and corresponding effective stiffness (E_cI_e) in KNm^2								
	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
C30	50646.834	95054.026	123649.498	20277.470	34592.469	46197.175	7181.194	10233.629	15075.615
Storey E_cI_e	**2332431.632**			**865796.908**			**272133.244**		
C50	37647.878	66731.692	90042.809	15028.957	24420.483	32181.564	5863.358	8025.379	10736.470
Storey E_cI_e	**1671714.288**			**610614.136**			**205649.740**		
C70	34705.120	63308.274	79365.588	13097.206	21960.509	29410.341	5647.524	7233.470	11710.706
Storey E_cI_e	**1533444.472**			**551197.660**			**203077.384**		
C90	19205.756	41142.457	48094.606	11052.133	18713.757	27790.767	4870.069	6898.670	11052.133
Storey E_cI_e	**955289.356**			**491099.752**			**190681.380**		

b) 12-storey

Table A 3-1: Stories Effective Stiffness (E_cI_e) for building models

Frame model	Analysis result for columns 12-storey											
FC30	Columns			CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m^2)			0.4×0.4	0.5×0.5	0.55×0.55	0.35×0.35	0.4×0.4	0.45×0.45	0.3×0.3	0.3×0.3	0.35×0.35
	Action effects M in (KNm)	P(KN)		2510.26	3603.01	3670.73	1566.89	2246.86	2399.67	737.01	1049.33	1179.63
		M_{22}	T	167.08	273.47	469.87	74.01	85.04	169.68	57.83	53.10	108.21
			B	0	0	0	-102.68	-115.50	-218.76	-80.96	-70.97	-140.07
		M_{33}	T	3.41	35.96	15.51	-5.05	14.91	6.39	-5.11	8.85	4.84
			B	0	0	0	5.50	-19.28	-8.01	5.59	-11.50	-5.94
FC50	X-sections (b×h) (m^2)			0.38×0.38	0.45×0.45	0.5×0.5	0.32×0.32	0.37×0.37	0.42×0.42	0.3×0.3	0.3×0.3	0.32×0.32
	Action effects M in (KNm)	P(KN)		2521.42	3568.72	3630.66	1576.99	2232.05	2374.00	749.97	1046.85	1156.60
		M_{22}	T	182.24	266.74	468.29	71.38	83.78	168.40	62.64	56.18	96.01
			B	0	0	0	-96.57	-114.60	-219.90	-89.75	-76.84	-121.03
		M_{33}	T	4.40	34.40	15.36	-5.42	14.00	6.55	-6.45	8.87	4.16
			B	0	0	0	5.96	-18.37	-8.27	8.13	-11.74	-5.06
FC70	X-sections (b×h) (m^2)			0.35×0.35	0.42×0.42	0.45×0.45	0.3×0.3	0.35×0.35	0.4×0.4	0.25×0.25	0.25×0.25	0.3×0.3
	Action effects M in (KNm)	P(KN)		2472.65	3558.21	3611.95	1538.16	2225.73	2384.07	725.25	1042.69	1177.61
		M_{22}	T	182.59	278.87	447.66	68.87	83.21	168.24	55.50	51.04	109.57
			B	0	0	0	-93.15	-112.24	-223.63	-78.50	-69.06	-144.37
		M_{33}	T	4.72	34.93	14.21	-5.15	13.90	5.84	-3.92	7.86	4.66
			B	0	0	0	5.90	-18.31	-7.57	4.01	-10.31	-5.77
FC90	X-sections (b×h) (m^2)			0.3×0.3	0.4×0.4	0.43×0.43	0.25×0.25	0.3×0.3	0.35×0.35	0.2×0.2	0.22×0.22	0.25×0.25
	Action effects M in (KNm)	P(KN)		2314.15	3517.84	3596.30	1465.89	2242.03	2374.02	694.76	1054.99	1165.70
		M_{22}	T	63.83	83.60	171.88	60.45	82.94	169.86	46.56	56.81	104.93
			B	-81.25	-150.12	-292.37	-84.25	-114.22	-229.61	-65.29	-78.99	-138.16
		M_{33}	T	-3.54	15.59	6.84	-1.87	14.69	6.70	-0.61	8.80	4.67
			B	-1.1	-14.80	-10.64	1.51	-19.09	-8.65	0.17	-11.78	-5.84

(a) 12-storey models

Frame model	Analysis result for columns 18-storey											
FC30	Columns			CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m^2)			0.4×0.4	0.5×0.5	0.55×0.55	0.35×0.35	0.4×0.4	0.45×0.45	0.3×0.3	0.3×0.3	0.35×0.35
	Action effects M in (KNm)	P(KN)		3710.16	5331.41	5374.27	2284.68	3288.31	3452.87	1050.96	1521.42	1602.34
		M_{22}	T	-44.72	-71.53	-25.13	-16.95	-8.07	-3.53	-11.05	-5.35	-1.04
			B	0	0	0	22.34	11.76	5.31	13.66	8.98	1.88
		M_{33}	T	165.94	340.38	580.81	60.58	78.81	159.40	40.27	57.81	87.73
			B	0	0	0	-84.68	-109.83	-209.12	-56.92	-94.63	-119.73
FC50	X-sections (b×h) (m^2)			0.38×0.38	0.45×0.45	0.5×0.5	0.32×0.32	0.37×0.37	0.42×0.42	0.3×0.3	0.3×0.3	0.32×0.32
	Action effects M in (KNm)	P(KN)		3708.75	5284.56	5359.71	2280.79	3259.29	3457.85	1054.32	1508.67	1610.43
		M_{22}	T	-51.94	-82.25	-26.96	-17.26	-9.03	-3.46	-11.76	-4.89	-0.54
			B	0	0	0	22.61	12.99	5.28	14.91	8.16	1.18
		M_{33}	T	179.12	357.95	622.94	59.22	78.39	159.40	39.92	53.55	84.25
			B	0	0	0	-81.38	-109.00	-210.44	-57.24	-89.32	-116.56
FC70	X-sections (b×h) (m^2)			0.35×0.35	0.42×0.42	0.45×0.45	0.3×0.3	0.35×0.35	0.4×0.4	0.25×0.25	0.25×0.25	0.3×0.3
	Action effects M in (KNm)	P(KN)		3724.83	5299.19	5280.81	2276.88	3258.11	3428.82	1056.78	1505.88	1604.56
		M_{22}	T	-62.67	-100.91	-25.46	-17.46	-9.76	-5.11	-12.41	-4.75	-0.94
			B	0	0	0	22.73	13.86	7.45	16.12	7.80	1.58
		M_{33}	T	208.84	418.54	661.66	58.14	79.71	162.12	39.96	51.78	82.87
			B	0	0	0	-78.32	-108.55	-216.86	-58.01	-87.24	-115.64
FC90	X-sections (b×h) (m^2)			0.3×0.3	0.4×0.4	0.43×0.43	0.25×0.25	0.3×0.3	0.35×0.35	0.2×0.2	0.22×0.22	0.25×0.25
	Action effects M in (KNm)	P(KN)		3552.94	5388.71	5270.95	2174.28	3299.96	3437.18	1026.87	1513.37	1613.77
		M_{22}	T	-60.01	-134.61	-24.58	-13.30	-8.99	-5.88	-10.84	-1.80	0.12
			B	0	0	0	17.50	13.62	8.86	14.80	2.76	-0.16
		M_{33}	T	169.26	476.42	709.05	51.37	84.42	173.33	38.32	49.30	81.60
			B	0	0	0	-70.04	-115.32	-234.91	-57.39	-82.90	-114.45

(b) 18-storey models

Table A 3-2: Analysis cross-section dimensions and output action effects (a) and (b)

Frame model	Designed data (12-story)										
FC30	Columns		CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m^2)		0.44×0.44	0.515×0.515	0.55×0.55	0.35×0.35	0.4×0.4	0.43×0.43	0.27×0.27	0.295×0.295	0.325×0.325
	Design Actions	P_d (KN)	2510.26	3603.01	3670.73	1566.89	2246.86	2399.67	737.01	1049.33	1179.63
		M_{xd} (KNm)	92.68	137.89	142.79	53.29	75.37	85.61	22.91	32.46	39.17
		M_{yd} (KNm)	150.45	236.14	355.34	72.41	91.15	135.50	47.12	49.37	79.62
	Total reinforcement area (mm^2)	Calc.	3027.90	4238.30	4808.24	1811.77	2477.92	2828.97	1182.29	1310.77	1666.34
		Prov.	3198.14	4429.65	5007.70	2035.75	2626.37	3091.33	1231.5	1420	1768.72
	Steel ratio	ρ(%)	1.65	1.67	1.66	1.66	1.64	1.67	1.69	1.65	1.67
	Axial load capacity	P_u(KN	3869	5319	6049	2453	3191	3709	1467	1732	2120
FC50	X-sections (b×h) (m^2)		0.39×0.39	0.45×0.45	0.485×0.485	0.31×0.31	0.35×0.35	0.375×0.375	0.245×0.245	0.265×0.265	0.285×0.285
	Design actions	P_d (KN)	2521.42	3568.72	3630.66	1576.99	2232.05	2374.00	749.97	1046.85	1156.60
		M_{xd} (KNm)	90.82	132.40	136.98	52.36	73.19	82.25	22.47	31.80	37.81
		M_{yd} (KNm)	159.77	231.42	353.59	70.17	90.48	135.44	50.90	51.67	71.54
	Total reinforcement area (mm^2)	Calc.	2425.45	3283.15	3888.99	1248.00	1780.31	2054.96	923.57	992.50	1212.94
		Prov.	2519.56	3411.77	3901.86	1602.21	2035.75	2318.50	986.46	1149.82	1325.75
	Steel ratio	ρ(%)	1.66	1.68	1.66	1.67	1.66	1.65	1.65	1.65	1.64
	Axial load capacity	P_u(KN	4199	5612	6496	2657	3384	3878	1654	1934	2235
FC70	X-sections (b×h) (m^2)		0.37×0.37	0.428×0.428	0.455×0.455	0.29×0.29	0.33×0.33	0.355×0.355	0.235×0.235	0.25×0.25	0.282×0.282
	Design actions	P_d (KN)	2472.65	3558.21	3611.95	1538.66	2225.73	2384.07	725.25	1042.69	1177.61
		M_{xd} (KNm)	88.17	130.60	134.33	50.59	72.21	82.04	22.53	31.78	38.26
		M_{yd} (KNm)	159.01	238.49	340.84	68.03	89.41	137.13	45.91	48.48	81.30
	Total reinforcement area (mm^2)	Calc.	1993.26	2667.16	3165.00	1282.65	1664.86	1995.48	743.77	889.53	1157.87
		Prov.	2271.37	3053.63	3411.77	1398.00	1806.42	2082.88	904.78	1027.30	1325.75
	Steel ratio	ρ(%)	1.66	1.67	1.65	1.66	1.66	1.65	1.64	1.64	1.67
	Axial load capacity	P_u(KN	4829	6467	7294	2967	3841	4442	1944	2201	2807
FC90	X-sections (b×h) (m^2)		0.31×0.31	0.374×0.374	0.39×0.39	0.27×0.27	0.308×0.308	0.339×0.339	0.22×0.22	0.24×0.24	0.27×0.27
	Design actions	P_d (KN)	2314.15	3517.84	3596.30	1465.89	2242.03	2374.02	694.76	1054.99	1165.70
		M_{xd} (KNm)	80.46	119.78	126.80	49.03	71.57	80.63	22.83	31.63	37.60
		M_{yd} (KNm)	78.78	130.40	188.87	63.02	90.53	139.32	40.01	52.70	78.58
	Total reinforcement area (mm^2)	Calc.	1507.52	2029.01	2300.00	1136.69	1344.70	1840.76	667.77	914.46	1150.47
		Prov.	1602.21	2318.50	2519.56	1218.94	1561.37	1941.50	801.12	945.62	1190.66
	Steel ratio	ρ(%)	1.67	1.66	1.66	1.67	1.65	1.68	1.66	1.64	1.64
	Axial load capacity	P_u(KN	4213	6127	6662	3197	4151	5034	2120	2520	3187

Table A 3-3 (a) Columns section designed data for 12-storey

Frame model	Designed data (18-story)										
FC30	Columns		CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m^2)		0.5×0.5	0.6×0.6	0.63×0.63	0.4×0.4	0.465×0.465	0.49×0.49	0.29×0.29	0.34×0.34	0.355×0.355
	Design Actions	P_d (KN)	3710.16	5331.41	5374.27	2284.68	3288.31	3452.87	1050.96	1521.42	1602.34
		M_{xd} (KNm)	173.77	310.85	455.97	79.56	109.69	152.71	43.79	68.28	79.94
		M_{yd} (KNm)	140.99	212.19	216.61	75.93	119.66	129.18	31.54	51.29	56.29
	Total reinforcement area (mm^2)	Calc.	4088.50	5942.52	6436.35	2420.80	3558.20	3706.18	1298.17	1863.01	1874.62
		Prov.	4165.75	6056.99	6540.80	2626.37	3650.53	4005.53	1420.00	1928.94	2108.01
	Steel ratio	ρ(%)	1.67	1.68	1.66	1.65	1.68	1.67	1.69	1.67	1.67
	Axial load capacity	P_u(KN	5010	7236	7891	3191	4351	4813	1692	2318	2528
FC50	X-sections (b×h) (m^2)		0.44×0.44	0.525×0.525	0.555×0.555	0.344×0.344	0.4×0.4	0.425×0.425	0.255×0.255	0.3×0.3	0.31×0.31
	Design actions	P_d (KN)	3708.75	5284.56	5359.71	2280.79	3259.29	3457.85	1054.32	1508.67	1610.43
		M_{xd} (KNm)	181.65	320.46	480.96	78.17	108.78	153.33	43.98	65.90	78.83
		M_{yd} (KNm)	136.99	203.19	208.98	73.36	114.37	125.35	30.71	49.94	55.47
	Total reinforcement area (mm^2)	Calc.	2914.58	4262.93	4879.04	1798.68	2594.09	2745.46	971.02	1363.18	1560.64
		Prov.	3198.14	4561.59	5139.65	1941.50	2679.78	3000.22	1080.71	1514.25	1608.50
	Steel ratio	ρ(%)	1.65	1.65	1.67	1.65	1.67	1.66	1.66	1.68	1.67
	Axial load capacity	P_u(KN	5360	7607	8517	3260	4428	4989	1796	2493	2659
FC70	X-sections (b×h) (m^2)		0.42×0.42	0.502×0.502	0.531×0.531	0.323×0.323	0.375×0.375	0.4×0.4	0.245×0.245	0.286×0.286	0.296×0.296
	Design actions	P_d (KN)	3724.83	5299.19	5280.81	2276.88	3258.11	3428.82	1056.78	1505.88	1604.56
		M_{xd} (KNm)	199.80	357.11	502.61	76.86	108.58	155.32	44.34	65.01	78.35
		M_{yd} (KNm)	136.18	201.56	203.62	72.28	112.57	122.08	30.58	49.52	54.71
	Total reinforcement area (mm^2)	Calc.	2619.75	3845.72	4515.89	1575.99	2047.50	2352.90	742.87	1196.90	1218.28
		Prov.	2893.41	4165.75	4706.12	1724.73	2318.50	2679.78	986.46	1372.87	1467.12
	Steel ratio	ρ(%)	1.65	1.65	1.67	1.65	1.65	1.67	1.65	1.68	1.67
	Axial load capacity	P_u(KN	6210	8883	9956	3678	4955	5653	2114	2891	3096
FC90	X-sections (b×h) (m^2)		0.385×0.385	0.48×0.48	0.507×0.507	0.305×0.305	0.358×0.358	0.382×0.382	0.235×0.235	0.273×0.273	0.286×0.286
	Design actions	P_d (KN)	3552.94	5374.25	5270.95	2174.28	3299.96	3437.80	1026.87	1513.37	1613.77
		M_{xd} (KNm)	172.62	393.34	530.85	71.50	112.13	162.72	43.49	63.43	78.06
		M_{yd} (KNm)	127.66	202.29	200.96	69.63	113.37	120.98	29.79	50.60	55.06
	Total reinforcement area (mm^2)	Calc.	2220.15	3636.06	4202.36	1301.26	1937.84	2040.89	686.27	1077.58	1143.99
		Prov.	2412.74	3901.86	4297.70	1520.53	2130.00	2412.74	904.78	1231.50	1372.88
	Steel ratio	ρ(%)	1.64	1.69	1.67	1.65	1.66	1.65	1.64	1.65	1.68
	Axial load capacity	P_u(KN	6477	10122	11273	4044	5616	6390	2415	3263	3589

Table A 3-3 (b) Columns section designed data for 18-storey

Frame model	Column	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
FC30	Design axial load (P_d) (KN)	2510.26	3603.01	3670.73	1566.89	2246.86	2399.67	737.01	1049.33	1179.63
	Section axial load capacity (P_u) (KN)	3869	5319	6049	2453	3191	3709	1467	1732	2120
	Axial load level (P_d/0.8 P_u)	**0.811**	**0.847**	**0.758**	**0.798**	**0.880**	**0.808**	**0.628**	**0.757**	**0.695**
FC50	Design axial load (P_d) (KN)	2521.42	3568.72	3630.66	1576.99	2232.05	2374.00	749.97	1046.85	1156.60
	Section axial load capacity (P_u) (KN)	4199	5612	6496	2657	3384	3878	1654	1934	2235
	Axial load level (P_d/0.8 P_u)	**0.751**	**0.795**	**0.698**	**0.742**	**0.824**	**0.765**	**0.567**	**0.676**	**0.646**
FC70	Design axial load (P_d) (KN)	2472.65	3558.21	3611.95	1538.66	2225.73	2384.07	725.25	1042.69	1177.61
	Section axial load capacity (P_u) (KN)	4829	6467	7294	2967	3841	4442	1944	2201	2807
	Axial load level (P_d/0.8 P_u)	**0.640**	**0.688**	**0.619**	**0.648**	**0.724**	**0.671**	**0.466**	**0.592**	**0.524**
FC90	Design axial load (P_d) (KN)	2314.15	3517.84	3596.30	1465.89	2242.03	2374.02	694.76	1054.99	1165.70
	Section axial load capacity (P_u) (KN)	4213	6127	6662	3197	4151	5034	2120	2520	3187
	Axial load level (P_d/0.8 P_u)	**0.686**	**0.717**	**0.675**	**0.573**	**0.675**	**0.589**	**0.409**	**0.523**	**0.457**

a) 12-storey

Frame model	Column	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
FC30	Design axial load (P_d) (KN)	3710.16	5331.41	5374.27	2284.68	3288.31	3452.87	1050.96	1521.42	1602.34
	Section axial load capacity (P_u) (KN)	5010	7236	7891	3191	4351	4813	1692	2318	2528
	Axial load level (P_d/0.8 P_u)	**0.926**	**0.921**	**0.851**	**0.895**	**0.945**	**0.897**	**0.776**	**0.821**	**0.792**
FC50	Design axial load (P_d) (KN)	3708.75	5284.56	5359.71	2280.79	3259.29	3457.85	1054.32	1508.67	1610.43
	Section axial load capacity (P_u) (KN)	5360	7607	8517	3260	4428	4989	1796	2493	2659
	Axial load level (P_d/0.8 P_u)	**0.865**	**0.868**	**0.787**	**0.875**	**0.920**	**0.866**	**0.734**	**0.756**	**0.757**
FC70	Design axial load (P_d) (KN)	3724.83	5299.19	5280.81	2276.88	3258.11	3428.82	1056.78	1505.88	1604.56
	Section axial load capacity (P_u) (KN)	6210	8883	9956	3678	4955	5653	2114	2891	3096
	Axial load level (P_d/0.8 P_u)	**0.750**	**0.746**	**0.663**	**0.774**	**0.822**	**0.758**	**0.625**	**0.651**	**0.648**
FC90	Design axial load (P_d) (KN)	3552.94	5374.25	5270.95	2174.28	3299.96	3437.80	1026.87	1513.37	1613.77
	Section axial load capacity (P_u) (KN)	6477	10122	11273	4044	5616	6390	2415	3263	3589
	Axial load level (P_d/0.8 P_u)	**0.686**	**0.664**	**0.585**	**0.672**	**0.735**	**0.673**	**0.531**	**0.580**	**0.562**

b) 18-storey

Table A 3-4: Axial load levels comparison

Frame model	Column concrete quantity (18-story)									
FC30	Columns	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m×m)	0.5×0.5	0.6×0.6	0.63×0.63	0.4×0.4	0.465×0.465	0.49×0.49	0.29×0.29	0.34×0.34	0.355×0.355
	Gross concrete area (m^2)	0.25	0.36	0.3969	0.16	0.216225	0.2401	0.0841	0.1156	0.126025
	Long. Steel area provided (m^2)	0.00416575	0.00605699	0.00654080	0.00262637	0.00365053	0.00400553	0.00142000	0.00192894	0.00210801
	Net concrete area (m^2)	0.24583425	0.35394301	0.3903592	0.15737363	0.21257447	0.23609447	0.08268	0.11367106	0.12391699
	Number of columns	24	72	48	24	72	48	24	72	48
	Total net concrete area (m^2)	5.9	25.484	18.737	3.777	15.305	11.333	1.984	8.184	5.948
	Total net concrete volume (m^3)	17.7	76.452	56.212	11.331	45.916	33.998	5.953	24.553	17.844
	Over all concrete volume (m^3)	**289.959**								
FC50	X-sections (b×h) (m×m)	0.44×0.44	0.525×0.525	0.555×0.555	0.345×0.345	0.4×0.4	0.425×0.425	0.255×0.255	0.3×0.3	0.31×0.31
	Gross concrete area (m^2)	0.1936	0.275625	0.308025	0.119025	0.16	0.180625	0.065025	0.09	0.0961
	Long. Steel area provided (m^2)	0.00319814	0.00456159	0.00513965	0.00194150	0.00267978	0.00300022	0.00108071	0.00151425	0.00160850
	Net concrete area (m^2)	0.19040186	0.27106341	0.30288535	0.1170835	0.15732022	0.17762478	0.06394429	0.08848575	0.0944915
	Number of columns	24	72	48	24	72	48	24	72	48
	Total net concrete area (m^2)	4.57	19.517	14.538	2.81	11.327	8.526	1.535	6.371	4.536
	Total net concrete volume (m^3)	13.709	58.55	43.615	8.43	33.981	25.578	4.604	19.113	13.607
	Over all concrete volume (m^3)	**221.187**								
FC70	X-sections (b×h) (m×m)	0.42×0.42	0.502×0.502	0.53×0.53	0.323×0.323	0.375×0.375	0.4×0.4	0.245×0.245	0.285×0.285	0.295×0.295
	Gross concrete area (m^2)	0.1764	0.252004	0.2809	0.104329	0.140625	0.16	0.060025	0.081225	0.087025
	Long. Steel area provided (m^2)	0.00289341	0.00416575	0.00470612	0.00172473	0.00231850	0.00267978	0.00098646	0.00137287	0.00146712
	Net concrete area (m^2)	0.17350659	0.24783825	0.27619388	0.10260427	0.1383065	0.15732022	0.05903854	0.07985213	0.08555788
	Number of columns	24	72	48	24	72	48	24	72	48
	Total net concrete area (m^2)	4.164	17.844	13.257	2.463	9.958	7.551	1.417	5.749	4.107
	Total net concrete volume (m^3)	12.492	53.533	39.772	7.387	29.874	22.654	4.251	17.248	12.32
	Over all concrete volume (m^3)	**199.531**								
FC90	X-sections (b×h) (m×m)	0.385×0.385	0.48×0.48	0.507×0.507	0.305×0.305	0.358×0.358	0.382×0.382	0.235×0.235	0.273×0.273	0.285×0.285
	Gross concrete area (m^2)	0.148225	0.2304	0.257049	0.093025	0.128164	0.145924	0.055225	0.074529	0.081225
	Long. Steel area provided (m^2)	0.00241274	0.00390186	0.00429770	0.00152053	0.00213000	0.00241274	0.00090478	0.00123150	0.00137288
	Net concrete area (m^2)	0.14581226	0.22649814	0.2527513	0.09150447	0.126034	0.14351126	0.05432022	0.0732975	0.07985212
	Number of columns	24	72	48	24	72	48	24	72	48
	Total net concrete area (m^2)	3.499	16.3078661	12.132	2.196	9.074	6.888	1.304	5.277	3.833
	Total net concrete volume (m^3)	10.498	48.924	36.396	6.588	27.223	20.666	3.911	15.832	11.499
	Over all concrete volume (m^3)	**181.537**								

Table A 3-5: (a) Columns concrete quantity for 18-storey

Frame model	Column concrete quantity (12-story)									
FC30	Columns	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m×m)	0.44×0.44	0.515×0.515	0.55×0.55	0.35×0.35	0.4×0.4	0.43×0.43	0.27×0.27	0.295×0.295	0.325×0.325
	Gross concrete area (m^2)	0.1936	0.265225	0.3025	0.1225	0.16	0.1849	0.0729	0.087025	0.105625
	Long. Steel area provided (m^2)	0.00319814	0.00442965	0.00500770	0.00203575	0.00262637	0.00309133	0.00123150	0.001420	0.00176872
	Net concrete area (m^2)	0.19040186	0.26079535	0.2974923	0.12046425	0.15737363	0.18180867	0.0716685	0.085605	0.10385628
	Number of columns	16	48	32	16	48	32	16	48	32
	Total net concrete area (m^2)	3.046	12.518	9.52	1.927	7.554	5.818	1.147	4.109	3.323
	Total net concrete volume (m^3)	9.139	37.555	28.559	5.782	22.662	17.454	3.44	12.327	9.97
	Over all concrete volume (m^3)	**146.888**								
FC50	X-sections (b×h) (m×m)	0.39×0.39	0.45×0.45	0.485×0.485	0.31×0.31	0.35×0.35	0.375×0.375	0.245×0.245	0.265×0.265	0.285×0.285
	Gross concrete area (m^2)	0.1521	0.2025	0.235225	0.0961	0.1225	0.140625	0.060025	0.070225	0.081225
	Long. Steel area provided (m^2)	0.00251956	0.00341177	0.00390186	0.00160221	0.00203575	0.00231850	0.00098646	0.00114982	0.00132575
	Net concrete area (m^2)	0.14958044	0.19908823	0.23132314	0.09449779	0.12046425	0.1383065	0.05903854	0.06907518	0.07989925
	Number of columns	16	48	32	16	48	32	16	48	32
	Total net concrete area (m^2)	2.393	9.556	7.402	1.512	5.782	4.426	0.945	3.316	2.557
	Total net concrete volume (m^3)	7.18	28.669	22.207	4.536	17.347	13.277	2.834	9.947	7.67
	Over all concrete volume (m^3)	**113.667**								
FC70	X-sections (b×h) (m×m)	0.37×0.37	0.428×0.428	0.455×0.455	0.29×0.29	0.33×0.33	0.355×0.355	0.235×0.235	0.25×0.25	0.282×0.282
	Gross concrete area (m^2)	0.1369	0.183184	0.207025	0.0841	0.1089	0.126025	0.055225	0.0625	0.079524
	Long. Steel area provided (m^2)	0.00227137	0.00305363	0.00341177	0.001398	0.00180642	0.00208288	0.00090478	0.00102730	0.00132575
	Net concrete area (m^2)	0.13462863	0.18013037	0.20361323	0.082702	0.10709358	0.12394212	0.05432022	0.0614727	0.07819825
	Number of columns	16	48	32	16	48	32	16	48	32
	Total net concrete area (m^2)	2.154	8.646	6.516	1.323	5.14	3.966	0.869	2.951	2.502
	Total net concrete volume (m^3)	6.462	25.939	19.547	3.97	15.421	11.898	2.607	8.852	7.507
	Over all concrete volume (m^3)	**102.203**								
FC90	X-sections (b×h) (m×m)	0.31×0.31	0.375×0.375	0.39×0.39	0.27×0.27	0.308×0.308	0.34×0.34	0.22×0.22	0.24×0.24	0.27×0.27
	Gross concrete area (m^2)	0.0961	0.140625	0.1521	0.0729	0.094864	0.1156	0.0484	0.0576	0.0729
	Long. Steel area provided (m^2)	0.00160221	0.00231850	0.00251956	0.00121894	0.00156137	0.00194150	0.00080112	0.00094562	0.00119066
	Net concrete area (m^2)	0.09449779	0.1383065	0.14958044	0.07168106	0.09330263	0.1136585	0.04759888	0.05665438	0.07170934
	Number of columns	16	48	32	16	48	32	16	48	32
	Total net concrete area (m^2)	1.512	6.639	4.786	1.147	4.478	3.637	0.762	2.719	2.295
	Total net concrete volume (m^3)	4.536	19.916	14.36	3.441	13.436	10.911	2.285	8.158	6.884
	Over all concrete volume (m^3)	**83.927**								

Tables A 3-5: (b) Columns concrete quantity for 12-storey

FC30	Longitudinal steel provided (12-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	20	6	2.465	14.79	25.09	3.3	82.797	16	1324.752
	18	2	1.996	3.992					
	16	4	1.577	6.308					
CL2	26	8	4.165	33.32	47.516	3.3	156.803	48	7526.544
	24	4	3.549	14.196					
CL3	24	8	3.549	28.392	39.286	3.3	129.644	32	4148.608
	22	2	2.982	5.964					
	20	2	2.465	4.93					
CM1	16	4	1.577	6.308	15.972	3.3	52.708	16	843.328
	14	8	1.208	9.664					
CM2	20	10	2.465	24.65	28.642	3.3	94.518	48	4536.864
	18	2	1.996	3.992					
CM3	20	6	2.465	14.79	24.252	3.3	81.319	32	2602.208
	16	6	1.577	9.462					
CU1	14	8	1.208	9.664	9.664	3.3	31.891	16	510.256
CU2	16	4	1.577	6.308	11.14	3.3	36.762	48	1764.576
	14	4	1.208	4.832					
CU3	18	3	1.996	5.988	13.873	3.3	45.781	32	1464.992
	16	5	1.577	7.885					
Sum									**24722.128**

(a) FC30

FC50	Longitudinal steel provided (12-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	18	2	1.996	3.992	19.762	3.3	65.215	16	1043.44
	16	10	1.577	15.77					
CL2	22	12	2.982	35.784	35.784	3.3	118.087	48	5668.176
CL3	22	2	2.982	5.964	30.614	3.3	101.026	32	3232.832
	20	10	2.465	24.65					
CM1	14	6	1.208	7.248	12.57	3.3	41.481	16	663.696
	12	6	0.887	5.322					
CM2	18	5	1.996	9.98	21.019	3.3	69.363	48	3329.424
	16	7	1.577	11.039					
CM3	16	10	1.577	15.77	18.186	3.3	60.014	32	1920.448
	14	2	1.208	2.416					
CU1	14	2	1.208	2.416	7.738	3.3	25.535	16	408.56
	12	6	0.887	5.322					
CU2	14	6	1.208	7.248	9.022	3.3	29.773	48	1429.104
	12	2	0.887	1.774					
CU3	16	2	1.577	3.154	10.402	3.3	34.327	32	1098.464
	14	6	1.208	7.248					
Sum									**18794.144**

(b) FC50

FC70	Longitudinal steel provided (12-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	16	9	1.577	14.193	17.817	3.3	58.796	16	940.736
	14	3	1.208	3.624					
CL2	22	6	2.982	17.892	32.682	3.3	107.851	48	5176.848
	20	6	2.465	14.79					
CL3	20	6	2.465	14.79	26.766	3.3	88.328	32	2826.496
	18	6	1.996	11.976					
CM1	14	1	1.208	1.208	10.965	3.3	36.185	16	578.96
	12	11	0.887	9.757					
CM2	16	10	1.577	15.77	18.186	3.3	60.014	48	2880.672
	14	2	1.208	2.416					
CM3	16	5	1.577	7.885	16.341	3.3	53.925	32	1725.6
	14	7	1.208	8.456					
CU1	12	8	0.887	7.096	7.096	3.3	23.417	16	374.672
CU2	14	3	1.208	3.624	8.059	3.3	26.595	48	1276.56
	12	5	0.887	4.435					
CU3	16	2	1.577	3.154	10.402	3.3	34.327	32	1098.464
	14	6	1.208	7.248					
Sum									**16879.008**

(c) FC70

FC90	Longitudinal steel provided (12-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	14	6	1.208	7.248	12.57	3.3	41.481	16	663.696
	12	6	0.887	5.322					
CL2	16	10	1.577	15.77	18.186	3.3	60.014	48	2880.672
	14	2	1.208	2.416					
CL3	18	2	1.996	3.992	19.762	3.3	65.215	32	2086.88
	16	10	1.577	15.77					
CM1	12	8	0.887	7.096	9.561	3.3	31.551	16	504.816
	10	4	0.6162	2.465					
CM2	16	6	1.577	9.462	16.71	3.3	55.143	48	2646.864
	14	6	1.208	7.248					
CM3	16	2	1.577	3.154	15.234	3.3	50.272	32	1608.704
	14	10	1.208	12.08					
CU1	12	5	0.887	4.435	6.284	3.3	20.737	16	331.792
	10	3	0.6162	1.849					
CU2	14	1	1.208	1.208	7.417	3.3	24.476	48	1174.848
	12	7	0.887	6.209					
CU3	14	7	1.208	8.456	9.343	3.3	30.832	32	986.624
	12	1	0.887	0.887					
Sum									**12884.896**

(d) FC90

Table A 3-6: Longitudinal reinforcement quantity for 12-storey models

FC30	Longitudinal steel provided (18-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	22	6	2.982	17.892	32.682	3.3	107.851	24	2588.424
	20	6	2.465	14.79					
CL2	26	8	4.165	33.32	47.516	3.3	156.803	72	11289.816
	24	4	3.549	14.196					
CL3	28	2	4.831	9.662	51.312	3.3	169.33	48	8127.84
	26	10	4.165	41.65					
CM1	18	4	1.996	7.984	20.6	3.3	67.98	24	1631.52
	16	8	1.577	12.616					
CM2	20	10	2.465	24.65	28.642	3.3	94.519	72	6805.368
	18	2	1.996	3.992					
CM3	22	8	2.982	23.856	31.421	3.3	103.689	48	4977.072
	18	3	1.996	5.988					
	16	1	1.577	1.577					
CU1	16	4	1.577	6.308	11.14	3.3	36.762	24	882.288
	14	4	1.208	4.832					
CU2	18	6	1.996	11.976	15.13	3.3	49.929	72	3594.888
	16	2	1.577	3.154					
CU3	20	3	2.465	7.395	16.537	3.3	54.572	48	2619.456
	18	3	1.996	5.988					
	16	2	1.577	3.154					
Sum									**42516.672**

(a) FC30

FC50	Longitudinal steel provided (18-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	20	6	2.465	14.79	25.09	3.3	82.797	24	1987.128
	18	2	1.996	3.992					
	16	4	1.577	6.308					
CL2	22	12	2.982	35.784	35.784	3.3	118.087	72	8502.264
CL3	24	8	3.549	28.392	40.32	3.3	133.056	48	6386.688
	22	4	2.982	11.928					
CM1	16	2	1.577	3.154	15.234	3.3	50.272	24	1206.528
	14	10	1.208	12.08					
CM2	18	5	1.996	9.98	21.019	3.3	69.363	72	4994.136
	16	7	1.577	11.039					
CM3	18	11	1.996	21.956	23.533	3.3	77.659	48	3727.632
	16	1	1.577	1.577					
CU1	14	6	1.208	7.248	8.48	3.3	27.984	24	671.616
	10	2	0.6162	1.232					
CU2	16	6	1.577	9.462	11.878	3.3	39.197	72	2822.184
	14	2	1.208	2.416					
CU3	16	8	1.577	12.616	12.616	3.3	41.633	48	1998.384
Sum									**32296.56**

(b) FC50

FC70	Longitudinal steel provided (18-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	18	9	1.996	17.964	22.695	3.3	74.894	24	1797.456
	16	3	1.577	4.731					
CL2	22	6	2.982	17.892	32.682	3.3	107.851	72	7765.272
	20	6	2.465	14.79					
CL3	24	2	3.549	7.098	36.918	3.3	121.829	48	5847.792
	22	10	2.982	29.82					
CM1	14	9	1.208	10.872	13.533	3.3	44.659	24	1071.816
	12	3	0.887	2.661					
CM2	16	10	1.577	15.77	18.186	3.3	60.014	72	4321.008
	14	2	1.208	2.416					
CM3	18	5	1.996	9.98	21.019	3.3	69.363	48	3329.424
	16	7	1.577	11.039					
CU1	14	6	1.208	7.248	9.022	3.3	29.773	24	714.552
	12	2	0.887	1.774					
CU2	16	3	1.577	4.731	10.771	3.3	35.544	72	2559.168
	14	5	1.208	6.04					
CU3	16	5	1.577	7.885	11.509	3.3	37.98	48	1823.04
	14	3	1.208	3.624					
Sum									**29229.528**

(c) FC70

FC90	Longitudinal steel provided (18-storey)								
column	Dia (mm)	No	Wgt.per meter (Kg/m)	Total wgt. (Kg/m)	Sum wgt. (Kg/m)	Length used (m)	Total wgt. (Kg)	No of column	Over all total wgt. (Kg)
CL1	16	12	1.577	18.924	18.924	3.3	62.449	24	1498.776
CL2	22	2	2.982	5.964	30.614	3.3	101.026	72	7273.872
	20	10	2.465	24.65					
CL3	22	8	2.982	23.856	33.716	3.3	111.263	48	5340.624
	20	4	2.465	9.86					
CM1	14	4	1.208	4.832	11.928	3.3	39.362	24	944.688
	12	8	0.887	7.096					
CM2	16	6	1.577	9.462	16.71	3.3	55.143	72	3970.296
	14	6	1.208	7.248					
CM3	16	12	1.577	18.924	18.924	3.3	62.449	48	2997.552
CU1	12	8	0.887	7.096	7.096	3.3	23.417	24	562.008
CU2	14	8	1.208	9.664	9.664	3.3	31.891	72	2296.152
CU3	16	3	1.577	4.731	10.771	3.3	35.544	48	1706.112
	14	5	1.208	6.04					
Sum									**26590.08**

(d) FC90

Table A 3-7: Longitudinal reinforcement quantity for 18-storey models

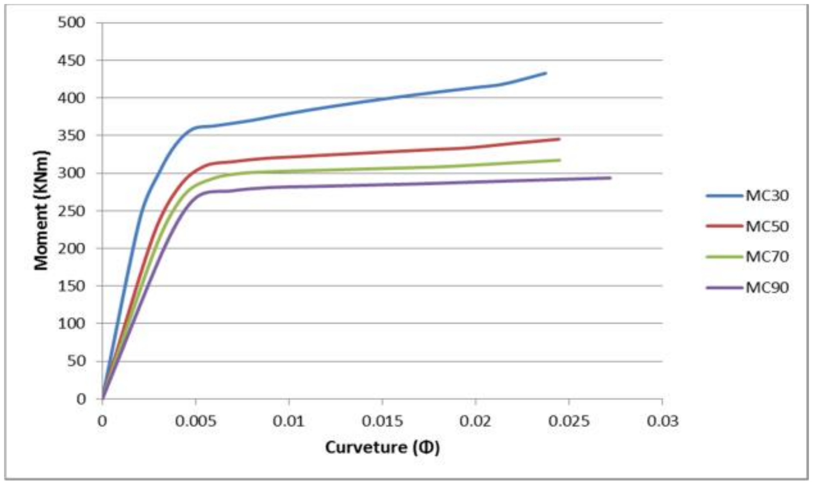
Frame model	Total space covered by Columns and total form work area required (18-story)									
FC30	Columns	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m×m)	0.5×0.5	0.6×0.6	0.63×0.63	0.4×0.4	0.465×0.465	0.49×0.49	0.29×0.29	0.34×0.34	0.355×0.355
	Gross concrete area (m^2)	0.25	0.36	0.3969	0.16	0.216225	0.2401	0.0841	0.1156	0.126025
	Number of columns	24	72	48	24	72	48	24	72	48
	Total gross concrete area (m^2)	6	25.92	19.05	3.84	15.57	11.52	2.02	8.32	6.05
	Sum of total gross area (m^2)	**98.29**								
	Form work area (m^2)	5.2	6.24	6.55	4.16	4.84	5.096	3.02	3.54	3.69
	Total form work area (m^2)	124.8	449.28	314.50	99.84	348.19	244.61	72.38	254.59	177.22
	Sum of form work area (m^2)	**2085.41**								
FC50	X-sections (b×h) (m×m)	0.44×0.44	0.525×0.525	0.555×0.555	0.345×0.345	0.4×0.4	0.425×0.425	0.255×0.255	0.3×0.3	0.31×0.31
	Gross concrete area (m^2)	0.1936	0.275625	0.308025	0.119025	0.16	0.180625	0.065025	0.09	0.0961
	Number of columns	24	72	48	24	72	48	24	72	48
	Total gross concrete area (m^2)	4.65	19.85	14.78	2.86	11.52	8.67	1.56	6.48	4.61
	Sum of total gross area (m^2)	**74.98**								
	Form work area (m^2)	4.58	5.46	5.77	3.59	4.16	4.42	2.65	3.12	3.22
	Total form work area (m^2)	109.82	393.12	277.06	86.11	299.52	212.16	63.65	224.64	154.75
	Sum of form work area (m^2)	**1820.83**								
FC70	X-sections (b×h) (m×m)	0.42×0.42	0.502×0.502	0.53×0.53	0.323×0.323	0.375×0.375	0.4×0.4	0.245×0.245	0.285×0.285	0.295×0.295
	Gross concrete area (m^2)	0.1764	0.252004	0.2809	0.104329	0.140625	0.16	0.060025	0.081225	0.087025
	Number of columns	24	72	48	24	72	48	24	72	48
	Total gross concrete area (m^2)	4.23	18.14	13.48	2.5	10.13	7.68	1.44	5.85	4.18
	Sum of total gross area (m^2)	**67.63**								
	Form work area (m^2)	4.37	5.22	5.51	3.36	3.9	4.16	2.55	2.96	3.07
	Total form work area (m^2)	104.83	375.9	264.58	80.62	280.8	199.68	61.15	213.41	147.26
	Sum of form work area (m^2)	**1728.23**								
FC90	X-sections (b×h) (m×m)	0.385×0.385	0.48×0.48	0.507×0.507	0.305×0.305	0.358×0.358	0.382×0.382	0.235×0.235	0.273×0.273	0.285×0.285
	Gross concrete area (m^2)	0.148225	0.2304	0.257049	0.093025	0.128164	0.145924	0.055225	0.074529	0.081225
	Number of columns	24	72	48	24	72	48	24	72	48
	Total gross concrete area (m^2)	3.56	16.59	12.34	2.23	9.23	7.00	1.33	5.37	3.90
	Sum of total gross area (m^2)	**61.55**								
	Form work area (m^2)	4.00	4.99	5.27	3.17	3.72	3.97	2.44	2.84	2.96
	Total form work area (m^2)	96.10	359.42	253.09	76.13	268.07	190.69	58.66	204.42	142.27
	Sum of form work area (m^2)	**1648.85**								

Table A 3-8 (a) Total space covered by Columns and total form work area required for 18-storey models

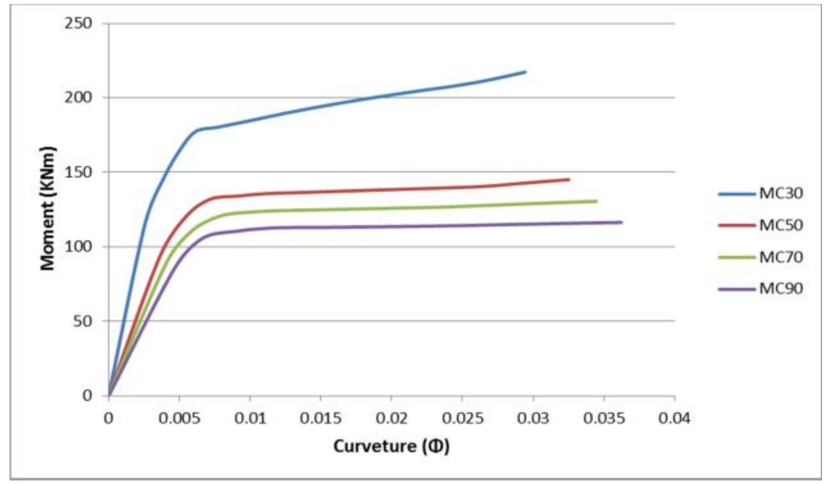
Frame model	Total space covered by Columns and total form work area required (12-story)									
FC30	Columns	CL1	CL2	CL3	CM1	CM2	CM3	CU1	CU2	CU3
	X-sections (b×h) (m×m)	0.44×0.44	0.515×0.515	0.55×0.55	0.35×0.35	0.4×0.4	0.43×0.43	0.27×0.27	0.295×0.295	0.325×0.325
	Gross concrete area (m^2)	0.1936	0.265225	0.3025	0.1225	0.16	0.1849	0.0729	0.087025	0.105625
	Number of columns	16	48	32	16	48	32	16	48	32
	Total gross concrete area (m^2)	3.10	12.73	9.68	1.96	7.68	5.92	1.17	4.18	3.38
	Sum of total gross area (m^2)	**49.8**								
	Form work area (m^2)	4.58	5.36	5.72	3.64	4.16	4.47	2.81	3.07	3.38
	Total form work area (m^2)	73.22	257.09	183.04	58.24	199.68	143.10	44.93	147.264	108.16
	Sum of form work area (m^2)	**1214.724**								
FC50	X-sections (b×h) (m×m)	0.39×0.39	0.45×0.45	0.485×0.485	0.31×0.31	0.35×0.35	0.375×0.375	0.245×0.245	0.265×0.265	0.285×0.285
	Gross concrete area (m^2)	0.1521	0.2025	0.235225	0.0961	0.1225	0.140625	0.060025	0.070225	0.081225
	Number of columns	16	48	32	16	48	32	16	48	32
	Total gross concrete area (m^2)	2.43	9.72	7.53	1.54	5.88	4.5	0.9604	3.37	2.60
	Sum of total gross area (m^2)	**38.53**								
	Form work area (m^2)	4.06	4.68	5.04	3.22	3.64	3.9	2.55	2.76	2.96
	Total form work area (m^2)	64.90	224.64	161.41	51.58	174.72	124.8	40.77	132.29	94.85
	Sum of form work area (m^2)	**1069.96**								
FC70	X-sections (b×h) (m×m)	0.37×0.37	0.43×0.43	0.455×0.455	0.29×0.29	0.33×0.33	0.355×0.355	0.235×0.235	0.25×0.25	0.285×0.285
	Gross concrete area (m^2)	0.1369	0.1849	0.207025	0.0841	0.1089	0.126025	0.055225	0.0625	0.081225
	Number of columns	16	48	32	16	48	32	16	48	32
	Total gross concrete area (m^2)	2.19	8.87	6.62	1.35	5.23	4.03	0.88	3.00	2.60
	Sum of total gross area (m^2)	**34.77**								
	Form work area (m^2)	3.85	4.47	4.73	3.02	3.43	3.69	2.44	2.6	2.96
	Total form work area (m^2)	61.57	214.66	151.42	48.26	164.74	118.44	39.10	124.8	94.85
	Sum of form work area (m^2)	**1017.84**								
FC90	X-sections (b×h) (m×m)	0.31×0.31	0.375×0.375	0.39×0.39	0.27×0.27	0.308×0.308	0.34×0.34	0.22×0.22	0.24×0.24	0.27×0.27
	Gross concrete area (m^2)	0.0961	0.140625	0.1521	0.0729	0.094864	0.1156	0.0484	0.0576	0.0729
	Number of columns	16	48	32	16	48	32	16	48	32
	Total gross concrete area (m^2)	1.54	6.75	4.87	1.17	4.55	3.70	0.77	2.76	2.33
	Sum of total gross area (m^2)	**28.44**								
	Form work area (m^2)	3.22	3.9	4.06	2.81	3.2	3.54	2.29	2.50	2.81
	Total form work area (m^2)	51.58	187.2	129.79	44.93	153.76	113.15	36.61	119.81	89.86
	Sum of form work area (m^2)	**926.69**								

Table A 3-8 (b) Total space covered by Columns and total form work area required for 12-storey models

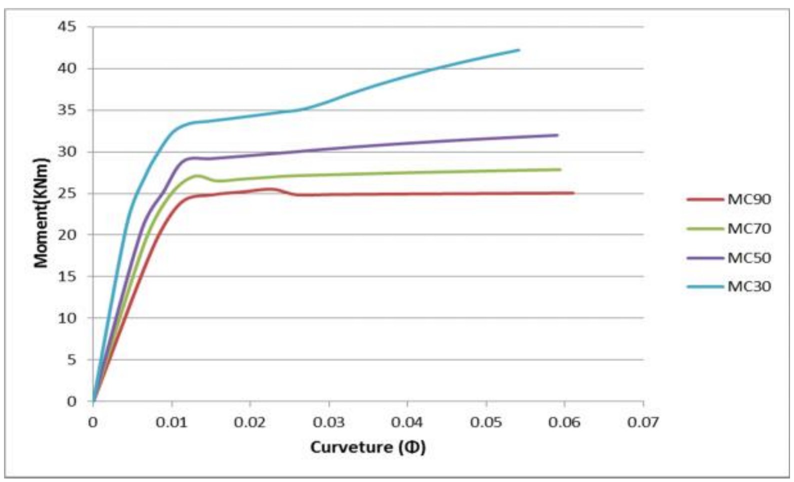
APPENDIX B



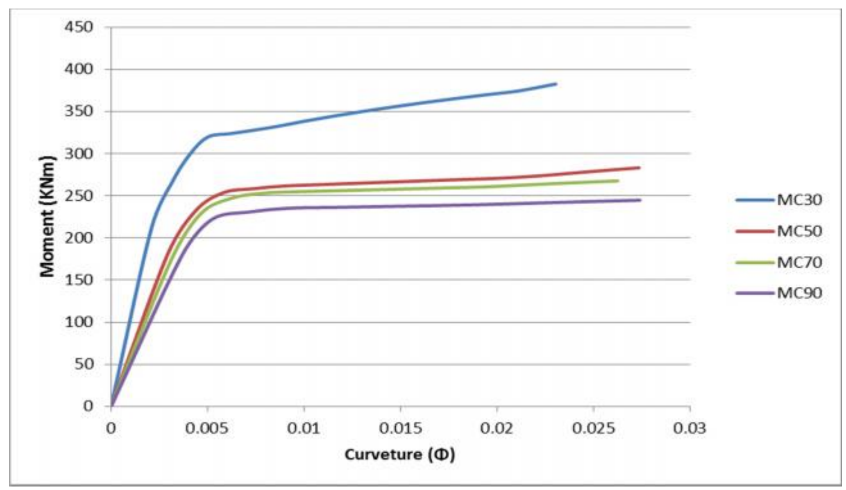
a) Column CL3



b) Column CM3

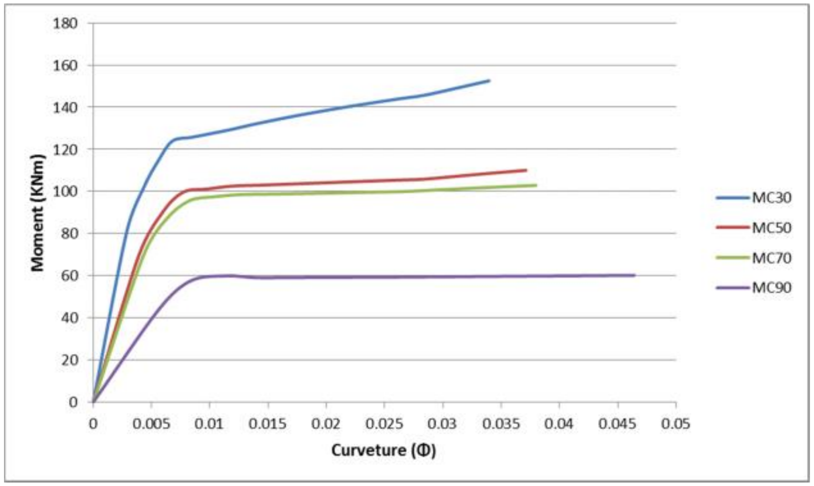


c) Column CU1

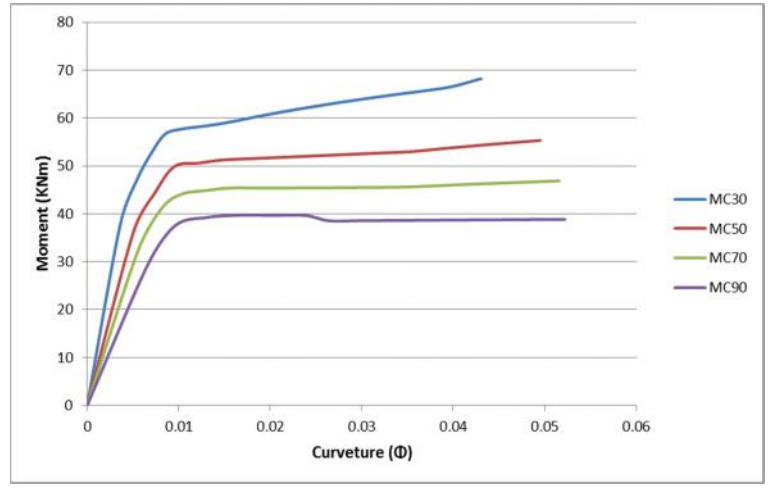


d) Column CL2

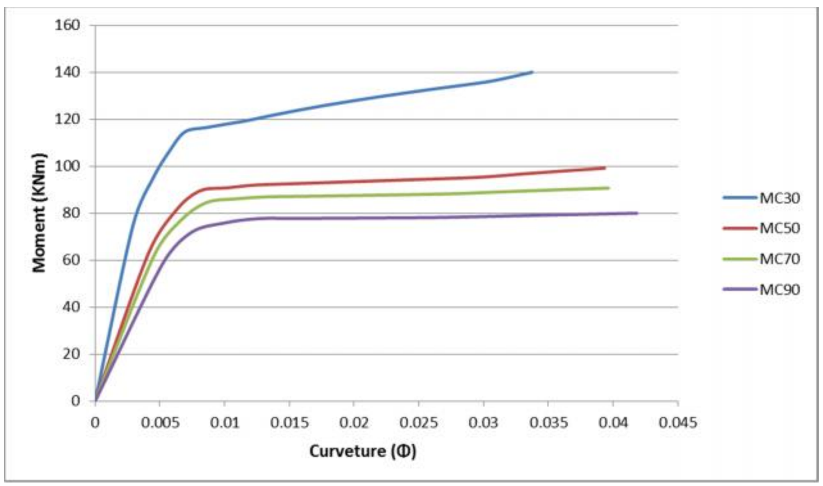
Figure B 3.1 Graphs of Moment-Curvature curves for columns of 18-storey



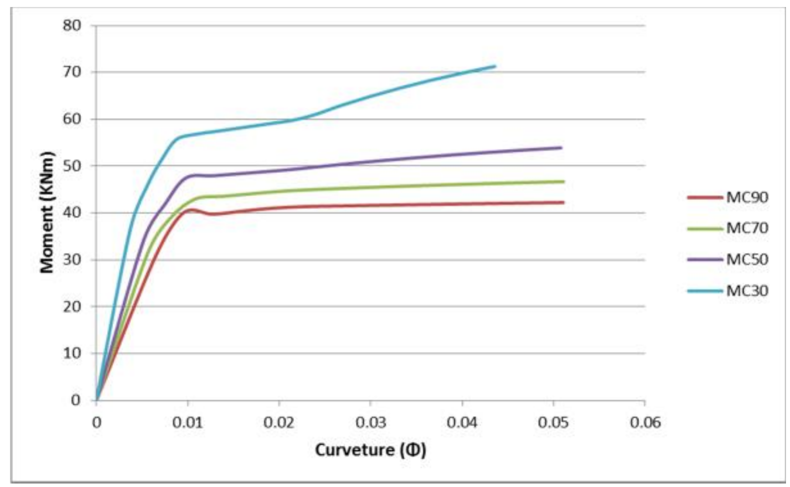
a) Column CL1



c) Column CM1



b) Column CM3



d) Column CU2

Figure B 3.2 Graphs of Moment-Curvature curves for columns of 12-storey