Critical Review of Reinforced Concrete Design Codes and Their Relevance to the United Arab Emirates

استعراض دقيق لمعايير تصميم الخرسانة المعززة وملاءمتها لدولة الإمارات العربية المتحدة

by

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ABSTRACT

The United Arab Emirates, is a home to structural engineers from different nations who hold expertise in their regional building concrete design codes. Since the 1980's and 1990's The British Standard (BS 8110) and the American Standards (ACI 318) have been of significance and widely used within the Municipalities in UAE. To maintain a uniformity and consistency in design and analysis of structures, it is required to critically review design codes to closely examine the similarities and differences among code provisions. This could help structural engineers to switch between codes. Hence, an attempt is made to carry out a general and parametric comparison of some international design codes followed by a three-tier critical review of ACI 318-14 and BS 8110-97 considering the fact that these codes are widely used within UAE. The three-tier comparison involves examining the results of literature review, theoretical investigation and practical design of frame elements of a G+40 story building using ETABS software. The results were compared in terms of dead and live loads and their combination, flexural and shear capacity of beams, columns and slabs, deflection and minimum and maximum amounts of longitudinal and transverse reinforcement, to arrive at a more economical solution without compromising strength and stability requirements. The results of the three-tier critical review showed that designs conforming to British Standards are preferred over the ACI Standards owing to their adaptability to the construction industry and environment in UAE which contributes the best possible solution.

ملخص

تعد دولة الإمارات العربية المتحدة موطن لعدد كبيرمن المهندسين الإنشائيين من بلاد مختلفة وهم يحملون معهم خبرة بلادهم في معايير التصـميم الخرسـانية المعززة. ومنذ الثمانينات والتسـعينات من القرن الماضـي كانت بلديات دولة الإمارات تعطي اهتمام كبيروتسـتخدم على نطاق واسـع المعيار البريطاني (BS 8110) والعايير الأمريكية (ACI 318) وذلك لتحافظ على نظام موحد للتصاميم وعلى ثبات لنتائج تحليل الإنشاءات.
من المهم أن نسـتعرض بدقة أحكام هذه المعايير وأن نسـتعرض أوجه الشـبه بينها، وهذا من شـأنه أن يسـاعد المهندسين الإنشائيين ليختاروا بينها.

وعليه، فقد تم عمل مقارنة عامة بين معايير التصـميم الدولية متبوعا باسـتعراض دقيق علي ثلاثة مراحل ACI 318 و BS 8110-97 مع مراعاة أن هذه المعايير مستخدمة علي نطاق واسع بدولة الإمارات.

تشمل المقارنة التي ستتم على ثلاث مراحل: فحص نتائج استعراض نظري، فحص افتراضي، وتصميم عملي للعناصر G+40 لبناية متعددة الطوابق مع استخدام البرنامج ETABS.

تمت مقارنة النتائج في حالة وجود أو عدم وجود حمولة دائمة او متغيره أو مزيج من الإثنان معاً، ألتحمل ألانحنائي والقصي للجسوروالاعمدة والعقدات ، درجة الإنحراف الأدني والأعلى مع التقوية العرضية، حتى تم التوصل لنتائج ذات حلول اقتصادية أكثر بدون التأثير على قوة وثبات البناء .

اثبتت نتائج الإستعراض الدقيق للثلاث مراحل تفضيل استخدام المعايير البريطانية على معايير الـــ ACI وذلك لتكيف المعايير البريطانية مع بيئة الصناعة الإنشائية في دولة الإمارات ومساهمتها بالحلول الأفضل.

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SYMBOLS

BS 8110-97

A_{sv}	total cross-sectional area of shear reinforcement
A_s	area of tension reinforcement
A_s'	area of compression reinforcement
A_c	net cross-sectional area of concrete in a column
A_{sc}	area of longitudinal reinforcement
b	width of section
b_{col}	width of column
b_o	clear height between end restraints
b_e	effective height
b_{ex}	effective height in respect of x-x axis
b_{ey}	effective height in respect of y-y axis
c	nominal cover to reinforcement
d	effective depth of the tension reinforcement
d’	depth to the compression reinforcement
F_k	characteristic load
f_k	characteristic strength
f_{cu}	characteristic compressive cube strength of concrete
f_y	characteristic tensile strength of reinforcement
f_{yv}	characteristic strength of links
g_k, G_k	characteristic dead load
h_{col}	depth of column
h	overall depth of section
K	coefficient given by $M/f_{cu}bd^2$
K′	coefficient given by $Mu/f_{cu}bd^2$ = 0.156 when redistribution does not exceed 10 per cent
l	effective span

M	design ultimate moment
M_u	design ultimate moment of resistance
N	design ultimate axial load
q_k, Q_k	characteristic imposed load
s_v	spacing of links along the member
V	design shear force due to ultimate loads
v	design shear stress
v_c	design concrete shear stress
w_k, W_k	characteristic wind load
x	depth to neutral axis
z	lever arm
γ_f	partial safety factor for load
γ_m	partial safety factor for material strengths
Φ	diameter of main steel
Φ'	diameter of links

ACI 318-14

a	depth of equivalent rectangular stress block
a_v	shear span
A_s	area of non-prestressed longitudinal reinforcement
A_v	area of shear reinforcement with spacing s
$A_{v,min}$	minimum area of shear reinforcement within spacing s
b_w	width, web width or diameter of circular section
c	distance from extreme compression fibre to neutral axis
d	distance from extreme compression fibre to centroid of longitudinal tension reinforcement
d’	distance from extreme compression fibre to centroid of longitudinal compression reinforcement
d_{bar}	diameter of longitudinal reinforcement bar

d_{bend}	diameter of bend in anchorage bars
$d_{stirrup}$	diameter of transverse reinforcement or stirrups
f'_c	specified compression strength of concrete
f_y	specified yield strength of non-prestressed reinforcement
l	span length of beam or one-way slab
M_n	Nominal flexural strength at section
$M_{n,req}$	Required nominal flexural strength at section
M_u	Factored moment at section
n	number of longitudinal or transverse reinforcement bars
V_C	Nominal shear strength provided by concrete
V_N	Nominal shear strength
$V_{n,\ req}$	Required nominal shear force at the section
$V_{n,req,max}$	Maximum required nominal shear force at the section
$V_{n,\ req,cr}$	Critical required nominal shear force at the section
V_S	Nominal shear reinforcement provided by shear reinforcement
$V_{s,\ req}$	Required shear strength provided by shear reinforcement
V_u	Factored shear force at the section
$V_{u,\ cr}$	Critical factored shear force at the section
$V_{u,max}$	Maximum factored shear force
S_b	center-to-center spacing of reinforcement bars
Φ	Strength reduction factor
β_1	factor relating to depth of equivalent rectangular compression stress block to depth of neutral axis
ε_s	Tensile strain in reinforcement

CHAPTER 1

INTRODUCTION

“Design is the methodical investigation of the stability, strength and rigidity of the structure” (FAO 2011)(Gowrishankar *et al.*, 2018)

1.1 Background

Structural engineering includes methodical investigations, through analysis to ensure stability and robustness of the structure and design it for maximum safety. Structural design engineering is all about analyzing a structure for a given set of loads, at its ultimate scenarios to study the behavior of a structure. Such analysis efficiently explains how the structure responds to worst case scenarios, which may or may not be met during design life of the structure. This behavioral response gives engineers an idea about the extent of design and detailing required by the structure.

Design preceded by a vigorous analysis of the structure under different combination of loads is the methodical procedure prerequisite to ensure buildability. However, it is mandatory to also ensure that every structure is built to standards appropriate for the respective region. And for this it is required that regional legislatures develop and enforce building standards and practices conforming to the topography, climate and geology of the region. These documents can also be called as codes/standards.

A structural engineer plays a vital role in designing and giving the required stability and strength to a structure. But structural engineers always require design aids such as standards to which designs should conform. Usually when a structural engineer is assigned with a project, there are a couple of questions for which answers are sought before commencing analysis and design. These questions are; Does the country you want to build the structure, use any construction regulations or standards to follow? If yes, how strong and stringent are they? How often are they updated and revised? The UAE is a home to structural engineers from different nations, and hence they use multiple national design standards for building design. These design standards are often the regional standards in which structural engineers have expertise. The municipalities in UAE, have set design standards which are stringently conformed to during the design stage and any design deviations from these standards will be penalized. These standards vary from emirate to emirate. The UAE widely uses the ACI and BS design codes but, the Eurocode2 has its prominence as well in this part of the world, where it is also considered as one of the major codes used in UAE. However, UAE does not have a truly national building design code as a result of which, they encourage the designs using mainly two international design codes – the ACI 318 and BS 8110 which will be of prime focus at the later stages of this research.

1.2 Research Significance

The significance of this research lies in the increased importance given to design codes around the world attributing to the loss of life and property due to flaws in design. Predominantly, in UAE, design conforming to the American and British standards, ACI 318 and BS 8110 respectively, have been followed and approved by municipalities. Approvals of designs from local authorities are mandatory before commencing construction. This implies a need to conform to design standards. As mentioned above, UAE is a home to structural engineers from different nations and each of them have an expertise on their regional codes. There is a need to maintain a uniformity and consistency among designs to enable easy understanding and buildability. However, switching between the codes is a cumbersome process. There is a need today, to understand the disparities among codal provisions especially when your region of work changes, as many codes, although they follow similar design philosophy, follow various approaches with regards to design methodology.

1.3 Research Objective

The primary objective of this research work is to critically review the concrete design code provisions of British and American standards parametrically and compare the findings of previous literature with the theoretical study and practical design – the three-tier critical analysis approach. However, this work also tries to compare some of the major provisions of the concrete building design codes of some international codes, namely, ACI 318, BS 8110 and Eurocode 2. It is expected that this comparative study of the three codes in general and the two codes, ACI and BS in particular will help understand the similarities and disparities between codes, which will inturn improve consistency and uniformity in designs while switching between codes. It concludes from the following study as to which code – ACI or BS – is suitable and more favorable to UAE building conditions. Such a study will be useful to structural designers who are based in a country where more than one code is approved for structural design, just like the UAE. It has also been suggested under available literature that mixing the use of different design codes could lead to conservative or unconservative results for the required dimensions, reinforcements and section modulus (Bakhoum, Mourad & Hassan, 2016). Hence, this practice is not appreciated in the construction industry.

1.4 Research Methodology

The proposed comparative study of building design codes is carried out step-wise. Firstly, the general comparison of the three international codes namely, ACI 318, BS 8110 and Eurocode 2 are carried out, which includes reviewing the major limit states of flexure, shear, axial compression under ultimate limit states and deflection, durability and cracking under serviceability limit states. This general and parametric comparison enables in reviewing the differences and similarities between limit states of designs, and a detailed parametric study of empirical equations to study the implications on the output as a result of change in parameters. This wide reviewing then narrows to the two main codes used in the UAE the ACI 318 and BS 8110. The later chapters primarily focus on the disparities and similarities between provisions of ACI 318 and BS 8110. This is a twofold approach, firstly comparing the main provisions of flexure, shear, deflection etc., followed by the modelling and analyzing of a G+40 story mixed use building using both codes with the help of a commercial software like ETABS. This step-wise approach aims at concluding the best possible solution – safe and economical- adapting to the UAE construction environment.

1.5 Research Challenges

Different national codes and standards have a very wide set of documentation regarding guidelines and how they were developed. To review and study each of these design standards is a task which consumes time. Also, understanding the philosophy behind each codal provision and how they have been developed. A wide range of literature was necessary to study and review, in order to complete the three-tier comparative study of the standards. Above all, these conditions had to be compared with the building environmental conditions in UAE to arrive at the best possible solution which will be explained in detail in the chapters considered hitherto.

1.6 Dissertation Outline

This piece of work collates the general similarities and differences of some major codes while it specifically focuses on American and British building standards.

Chapter 2 includes the review of literature on some international design codes, focusing on the major design provisions of both ultimate limit states and serviceability limit states.

Chapter 3 deals with the general comparison of major international building codes. This includes an overview of some of the provisions of different limit states like flexure, shear, deflection, cracking etc. as recommended by different codes.

Chapter 4 deals with the parametric study of provisions in three different codes – ACI 318-14, BS 8110-97 and Eurocode2. This study compares the different parameters used in the empirical equations in different codes and how they impact on the size and detailing of the required section.

Chapter 5 focuses on the comparison of ACI and BS code provisions specifically in terms of flexural design, shear design, deflection and minimum/maximum reinforcement areas stipulated by the two codes. This helps us compare both codes in terms of strength, stability and the more economical design.

Chapter 6 elaborates on the critical comparison of design provisions recommended by ACI and BS codes. This is achieved by modelling and designing a G+40 multi-story mix use building using a commercial software like ETABS Ultimate.

Chapter 7 illustrates on the discussions drawn from the above general and critical review of code provisions along with recommendations.

CHAPTER 2

REVIEW ON GENERAL COMPARISON OF INTERNATIONAL CONCRETE DESIGN CODES

2.1 General

A code is basically a standard which provides a common language and platform to structural engineers about the requirements for design, construction and operation of a structure. In short, they are some of the first tools used by structural designers to guide them through the different stages of designing structural elements, members and frames. Codal provisions, if enforced by the legislature of a region, should be mandatorily implemented because design and work cannot commence and neither progress, unless, it is ensured that all norms, recommendations and standards to protect public health, safety and welfare, are satisfied by the proposed design. Codes do not just supplement the construction and designing stages of a structure but they also govern repair, maintenance and rehabilitation of a structure through its service life so that the structure suffers minimal chances of failure and deterioration. These standards and regulations needed to be enforced and enacted by the regional legislature in order to bring uniformity and consistency during structural designing.

2.2 Significance of Building Design Standards

These building standards were developed and written by the collaborative approach of a large panel or committee of expert professionals and academicians, after critically reviewing the drawbacks in design from previous experiences and recent researches carried then. The topography, geology and adverse climatic changes possible in the region, also greatly impacted the recommendations suggested by the proposed codes. Specific building codes relate to the type of exposure the proposed structure would be exposed to. Most countries in the world have a National Building Code which has been written specifically for the building to be designed in that nation's buildable condition and environment. This implies to design provisions for high temperatures, humidity, designs resistant to earthquakes, hurricanes, cyclones typhoons, soil settlement and liquefaction etc. This explains why a structural design needs to conform and follow the national building code requirements of a country/state/town. Hence, these standards have become an integral part of the construction industry. It is the national or regional building departments responsible to make sure that every structure constructed within its jurisdiction conforms to the safety regulations and design provisions enforced and enacted by the legislature. Legal sanctity of a code is inevitable if safe, sound and robust structures are desired. Loss of life and property as a result of defective design, under design and improper construction practices led to the evolution of building design codes.

This shows that design codes came up in most of the countries with a purpose of following consistency. However, there are nations which do not have any structural design norms for themselves and approve the use of design codes of advanced countries. Sometimes the use of multiple codes is permitted as mentioned in the previous chapter. This practice demands an insight into the most effective and economical design possible using multiple codes.

2.3 Review on General Comparison of Design Code Provisions

The wide range of literature available with comparison of international codes adds on to this piece of research work. However, the objective is to focus on evaluating the differences in actions, strength, design criteria, minimum requirements etc. and conclude as to which code recommends economical guidelines in practice. Jawad (Jawad, 2006) in his paper compared the safety provisions, flexural, shear and column design of ACI 318M-02, BS 8110-1985 and Eurocode2-1992. He inferred from his studies that the EC2 partial safety factors were more liberal than ACI code. Mourad et al. (Bakhoum, Mourad & Hassan, 2016) in their paper reviewed the load intensities, both permanent and variable loads, of ACI 318, EC2, ECP 201-2011, IS 875 and BS 8110. They concluded that, the difference in intensities of variable loads was more at areas of stairs, balconies of BS and ACI. Lee & Scanlon (Lee & Scanlon, 2010) reviewed the minimum thickness provisions for both one way and two way slabs in different codes namely, ACI 318, BS 8110, EC2 and AS3600. Hegger et al. (Siburg, Ricker & Hegger, 2014) carried out a critical review of the punching shear design of footings according to various codes. Tabish & Reena (Izhar & Dagar 2018) performed an analytical study of EC2,IS 456-2000, ACI 318, BS 8110 and CSA. Their study dealt with the comparison of commonalities and differences in beam, column and slab design parameters. Hany et al. (Hany et al, 2018) worked with four different codes to conclude the variations in actions, safety factors and resistance of RC sections to flexure and axial loads. On the other hand, Alnuaimi and Patel (Alnuaimi & Patel, 2013) studied the variations in bar anchorage lengths, lap lengths and limit states of different codes. Nandi and Guha (Nandi & Guha, 2014) compared the units, design equations and criteria of BS 8110, EC2 and IS 456. Kim and Park (Kim and Park, 1996) suggested that compressive strength, tensile reinforcement ratio and shear span-depth ratio were the important variables affecting shear strength of concrete members. Slowik (Slowik, 2014) stated in her paper that the size effect was considered in the equations for the shear capacity of members without transverse reinforcement as in Eurocode2 whereas, some codes like the ACI 318-2002 do not consider the size effect. However, Karim (Karim, 1999) suggested in his paper that, the ACI provisions for shear strength has imperfections.

Bacinskas et al (Structures, 2008) statistically investigated the accuracy of the long-term deflection predictions made by the various design codes including Eurocode 2, ACI 318, ACI 435, SP 52-101 and the flexural layered deformation model proposed by Kaklauskas. They found that the Eurocode 2 overestimates the long-term deflection while ACI 318 and ACI 435 underestimated it. Lee and Scanlon (Lee & Scanlon, 2010) conducted parametric study on the control of deflection of reinforced concrete slabs, and compared the various design provisions in the ACI 318:08, BS 8110:97, Euro-code 2 and AS 3600:01. They concluded that although the minimum thickness values are easy to apply, limitations need to be placed on the applicability of current ACI 318:08 values due to the assumption that the slab thickness is independent of applied dead and live loads and no limits are specified on the applicable range of span lengths. The deflection control of slabs and beams was mainly dealt using the span-depth ratio provision of slabs to determine the minimum thickness of the slab and beams. However, a detailed study of deflection control required calculating the short-and long-term deflections. Presently, there is no simple procedure given in available literature to calculate the maximum deflection of two way slabs, however, (Nayak, S.K, 2004) suggest that this could be estimated by modifying the Branson's empirical formula for beams and one-way slabs. Santhi et al. (Santhi, Prasad & Ahuja, 2007) made an attempt to estimate the significance of parameters like total thickness, concrete compressive strength, concrete cover etc. on the total deflection of flat plates and concluded that, a careful application of above parameters could reflect in reduction of deflection. It was also revealed that available provisions of determining deflections varied widely among codes. N. Subramanian (Subramanian, 2010) in his paper compared the minimum reinforcement requirement of tension reinforcement and shear reinforcement among different codes of which, he made a conclusion that some of the provisions in the Indian code needed a revamping when compared to their counterparts in ACI 318. N. Subramanian (Subramanian, 2005) also made a comparison of the crack width provisions of some major codes and came to a conclusion that, except ACI 318, most codes followed relatively the same procedure to determine the design surface crack width. On the contrary, the ACI 318, calculated the maximum permissible spacing between reinforcements. The durability provisions as per (M.B. Anoop, K. Balaji Rao, T.V.S.R. Appa Rao, 2001) it was stated that the durability provisions greatly varied among codes primarily because the type and exposure condition of environments varied. But the quality control of the concrete along with appropriate execution of placing and compaction reflected on the durability of the structure. Alnuaimi et al. (Alnuaimi, Patel & Al-mohsin, 2013) carried out a comparative study on the anchorage length of bars which proved that ACI provisions were on the less economical side.

Hence, research has been carried out since many years regarding the critical review of design codes and their commonalities and differences.

CHAPTER 3

GENERAL COMPARISON OF INTERNATIONAL CONCRETE DESIGN CODES – ACI 318-14, BS 8110-97 AND EUROCODE2 (BS EN 1992-1-1)

This chapter focusses on a general comparison of major international code provisions with respect to several parameters such as units, actions and strengths, safety factors, Ultimate Limit States of flexure, shear and axial design, minimum reinforcement requirements and Serviceability Limit States of cracking, deflection and durability provisions of some international concrete design codes.

3.1 System of Units

Most international codes follow the Metric International System of units – SI unit systems or the MKS (Meter-Kilo-Second) system while, only the ACI 318 predominantly uses the U.S. Customary unit - FPS (Foot-Pound-Second) system. The units can be converted for ease of practical application but research says that it is always good to stay with the units the code guidelines have been designed for; unless the codes themselves recommend a suitable switch between units. However, the ACI 318M-11 recommends the equivalence between SI metric units and U.S. customary units in Appendix F of the code. Hence, a detailed comparison of codes in the upcoming chapter will be executed using the MKS system.

3.2 Actions (Loads)

Actions (loads) and resistances (strength) of sections and their safety factors vary widely among most international codes. These inconsistencies in provisions of different codes reflect on the calculated design moments and required reinforcements as well. Most codes specify the unit weights of different materials to enable calculation of permanent loads – self weight of the structure. These are mostly based on the standard unit weights and hence do not vary much among codes. The following table, Table 1 helps us to make a comparison between the imposed load intensities for different types of building occupancy.

USE	Code	Floors (kN/m2)	Corridors (kN/m2)	Stairs (kN/m2)	Balconies (kN/m2)
Residential	ASCE 7-10	1.92	4.79	4.79	2.88
	ACI 318-14	1.9	4.8	4.8	4.8
	EC2	2	2	3	4
	BS 8110-97	1.5	2	3	3
USE	**Code**	**Floors**	**Corridors**	**Stairs**	**Balconies**
Office	ASCE 7-10	2.4	3.83	4.79	3.6
	ACI 318-14	2.4	4.8	4.8	4.8
	EC2	3	3	3	3
	BS 8110-97	2.5	4	4	4
USE	**Code**	**Floors**	**Corridors**	**Stairs**	**Balconies**
Shops	ASCE 7-10	6	6	4.79	-
	ACI 318-14	6	6	4.8	-
	EC2	5	5	5	-
	BS 8110-97	4	5	4	-

Table 1 – Values of imposed loads in different building codes

The above table gives the values of imposed load intensities for different types of building occupancies recommended by the standards for design loads, ACI 318, ASCE 7-10, EC2 and BS 6399-96. It can be seen from the tabulated data that large differences can be seen in the load intensities of corridors and balconies of residential buildings. Similarly, stair load intensities laid down by codes are largely different for shops. Mourad et al, 2016 also carried out a comparison of the variable actions (live loads) in ACI 318-14, ASCE 7-10, EC2 and ECP 210 for different types of buildings. When load intensities of variable loads suggested by different codes were compared with respect to those of EC2, they inferred that variable loads specified by ACI and ECP would give a less economical design as compared to EC2. While the BS standards are more economical than EC2.

As inferred from our studies, it can be seen that variable load intensities recommended by ACI 318 and ASCE 7-10 are marginally higher than other codes. However, BS 6399 code gives the least load intensities.EC2 is quite reasonable and conservative in load intensities. The ACI considers a greater load intensity value for buildings of certain importance factor just to consider a safer approach but this could overestimate the required design.

3.2.1 Load Safety Factors (γ_f)

Almost all codes mandatorily impose partial safety factors to account for inaccuracies during designing or a possible situation of unusual loading of the member (which is often termed as Ultimate Design Loads). Design load is obtained by multiplying the characteristic load with a partial safety factor (γ_f), stipulated by the respective design codes (Scott, Kim & Salgado, 2003). It reflects on the accuracy of various loads being predicted (Jawad,2006).

*Design Load = Characteristic Load * Partial safety factor* (γ_f)

The following table, Table 2, shows a comparison of the partial safety factors for permanent loads (DL) and variable loads (LL) at Ultimate Limit State, according to different codes.

Code	Dead Load (DL)	Live Load (LL)
ASCE 7-10	1.2	1.6
ACI 318M-02	1.2	1.6
EC2	1.35	1.5
BS 8110-97	1.4	1.6

Table 2: Partial safety factors for loads at Ultimate Limit State

It can be deduced from the above table that almost every code, except the EC2:1992, impose a relatively higher live load factor of 1.6 to variable loads. But, as a contrary, the dead load factors vary widely in comparison. ACI 318 and ASCE 7-10 recommend the least dead load factors (Bakhoum, Mourad and Hassan, 1996). Researches show that larger the partial safety load factors, larger the design moments and shear forces estimated which inturn requires a larger section for the same. This may at times be practically uneconomical. As dead loads are static, they require only a small factor of safety while considering ultimate design load. But variable actions tend to require a greater factor of safety considering the most critical load case of an unprecedented increase in variable actions during service life. The ACI 318, EC2 consider a very low factor of safety for dead loads because they are assigned as a static load case. The highest dead load factor of given codes, is considered by BS 8110 and it simply overestimates the static load case. But the scenario of live load is different. All codes except the EC2 estimate a higher factor of safety for live loads than dead loads. A hand calculation was done to compare the ultimate design loads. For this study a dead load of 2.5kN/m^2 and a live load of 5kN/m^2 was assumed for all codes. APPENDIX A gives an insight into the disparity in Ultimate Design loads calculated using different codal provisions. These loads were then factored with the respective safety factors for ultimate design loads and the following inferences were made:

- The BS 8110 provisions gave the highest ultimate factored load of 11.5kN/m^2
- The least factored ultimate load was given by Eurocode2 provisions with a value of 10.875kN/m^2, while the ASCE 7-10 and ACI 318 values marginally increased to 11kN/m^2.

3.2.2 Wind Loads

Most codes and standards base their wind loading on the random vibration-based gust loading factor approach to assess the effects of along winds and crosswind effects on tall structure (Davenport, 1967) (Tamura et al., 2005). However, most codes recommend using the wind tunnel test procedure to closely examine the response of the structure to wind loading. This is specifically more effective in the case of tall irregularly shaped buildings. But, even before conducting a wind tunnel test codes recommend collecting basic pre-requisites inputs such as basic wind speed, profile of expected wind loading, terrain condition etc. Just like seismic codes, wind loading codes also calculate a return period to account for the recurrence interval of hurricanes under hurricane zones (Kwon & Kareem, 2013). Wind pressures acting on a building surface with the tributary areas give the loads and moments acting at each level(Kwon & Kareem, 2013). However, wind load calculation carried out using software analysis in the following section will be program determined depending on the code chosen.

3.2.3 Material Safety Factors (γ_m)

Generally, codes use a material safety factor to account for inaccuracies and understrength of members. The characteristic strength is divided by the material safety factor to achieve the design strength of the material. The ACI 318 and the Eurocode2 uses the characteristic cylindrical compressive strength unlike other codes which use the characteristic cube compressive strength of concrete.

Design Strength = Characteristic Strength / Material partial safety factor (γ_m)

The following table, Table 3, shows the material safety factors recommended by the considered codes.

Code	Concrete	Steel
ASCE 7-10	-	-
ACI 318M-02	-	-
EC2	1.5	1.15
BS 8110-97	1.5	1.05

Table 3 – Material safety factors (concrete in flexure) for different codes.

We can infer from the table that all codes impose the same factor of safety for concrete unlike steel. However, it can be seen that the factor of safety for concrete is higher than that of steel considering the reduction in strength of concrete which may arise due to improper placing, compaction, and curing procedures that largely impact the strength of final specimen (Jawad, 2006). It can be deduced from the above table, that, material factor values against ACI 318 and ASCE 7-10 are not specified. This is because, unlike other codes, the American standards use a strength reduction factor ϕ. The ACI code specifies different ϕ values depending on the nature of stress the structural member is subjected to. Hawileh (Hawileh et al., 2009) stated that major difference between the safety factors in different codes for materials is that these factors are applied to both yield strength of reinforcing steel and compressive strength of concrete. On the other hand the strength reduction factor ϕ , in ACI 318, acts as an overall factor and is applied directly to the nominal moment strength of the cross section.

$$\boxed{M_{des} = \phi M_n}$$

The following table, Table 4, shows the strength reduction factors of ACI 318.

Stress Condition	ϕ Factors
Flexure	0.9
Axial tension	0.9
Shear and Torsion	0.75
Compression members spirally reinforced	0.7
Compression members tied reinforced	0.65
Bearing	0.65

Table 4: Strength reduction factor ϕ

According to Jawad (Jawad, 2006), the ϕ value and material safety factors reflect the probable quality control achievable and reliability of workmanship and inspection. This means φ value indirectly implies the targeted quality of the work as an outcome of workmanship.

3.2.4 Load Combinations

Load combinations imposed by codes widely vary from one national building standard to another. This partly depends on the weather conditions and geology of the region, the material properties etc. The following table, Table 5, summarizes the load combinations of the considered codes.

Code	Load Combinations
ACI 318M-02	1.4D
	1.2D+1.6L+0.5Lr
	1.2D+1.6Lr+(L or 0.8W)
	1.2D+1.6W+1.0L+0.5Lr
	0.9D+1.6W
EC2	1.35D+1.5L
	1.0D+1.5W
	1.35D+1.5L+0.9W
BS 8110-97	1.4D+1.6L
	1.4D+1.4W
	1.2D+1.2L+1.2W
	1.0D+1.4W

Table 5: Comparison of Load Combinations

For a typical member with DL=2LL, maximum uniformly distributed ultimate design load in EC2 would be 7.1% lower than ACI and 4.8% lower than that of BS 8110. (Jawad, 2006). A detailed comparison of the ultimate loads according to different design codes has been solved manually for comparative study in APPENDIX A. All codes consider effects of wind loads as a default. But ASCE 7-10 stands out as the live load is bifurcated as live and roof live loads unlike other codes. This implies the significance ASCE 7-10 gives to roof live loads in particular.

3.3 Behaviour of Reinforced Concrete Section in Flexure

All design codes follow a similar design philosophy to calculate the ultimate moment of resistance for sections governed by flexure. Of all the three strain zones (compression controlled, tension controlled and transition), a concrete beam cross section is characterized as tension controlled. This design philosophy of tension-controlled section is based on the strain limit rather than the stress limit. The nominal flexural strength is approached when the strain in extreme compression fiber and the strain in tension reinforcement reaches guidelines specified by respective codes (Hawileh et al., 2009). The relationship between concrete compressive stress distribution and concrete strain is depicted well by a rectangular, parabolic or trapezoidal block, depending on each code provisions (Hawileh et al., 2009). These concrete compressive stress- strain blocks for various design codes are shown below.

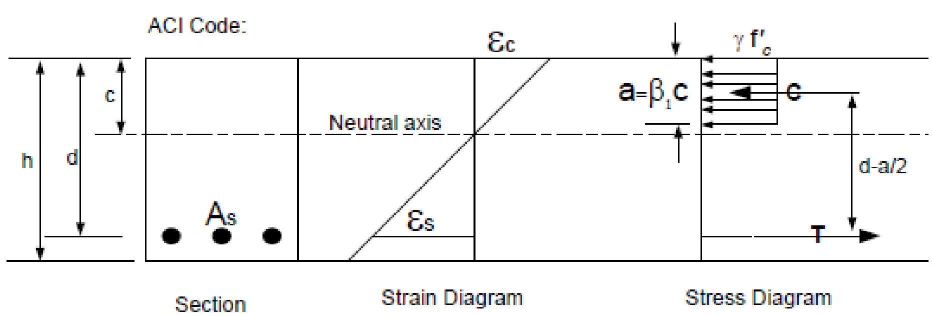


Figure 1: Concrete stress-strain block in ACI 318-14 (Jawad,2006)

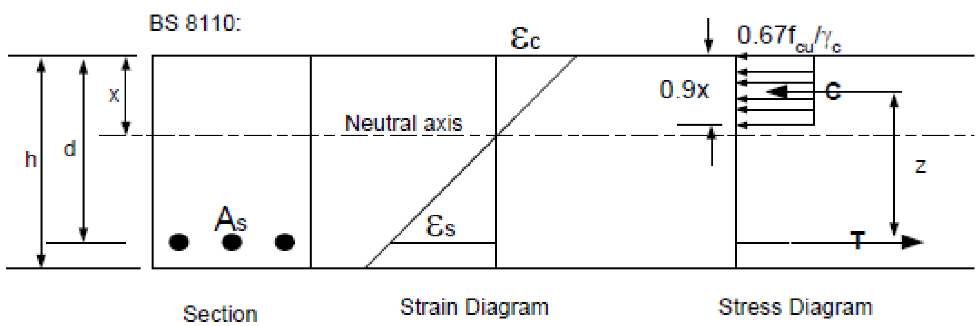


Figure 2: Concrete stress-strain block in BS 8110-97 (Jawad,2006)

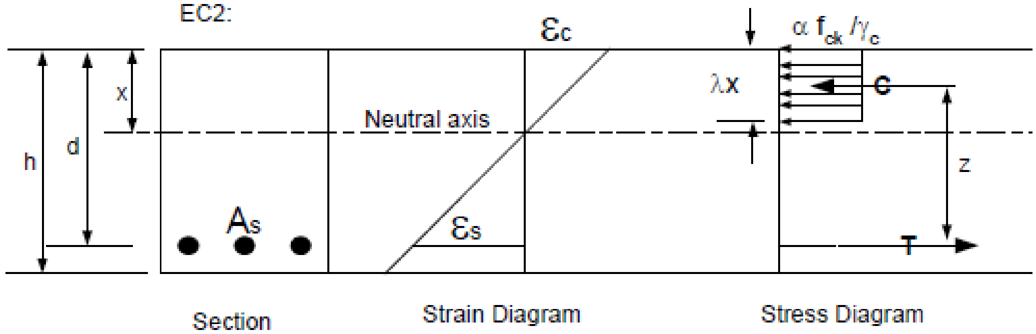


Figure 3: Concrete stress-strain block in EC2 (Jawad,2006)

Comparison of Equivalent Stress-strain blocks in different codes

A review of the concrete stress-strain block shows that the maximum usable ε_u strain at extreme compression fiber according to ACI 318-05 is assumed as 0.003 unlike the BS 8110 and EC2 which assume $\varepsilon_u = 0.0035$.

The concrete compressive stress-strain block is very significant in determining the ultimate moment of resistance M_u of a concrete section under flexure. Bakhoum et al. (Bakhoum, Mourad & Hassan, 2016) performed a study to analyze the ultimate moment of resistance of a singly reinforced section according to provisions of different codes but with the same material strengths – concrete and steel – for a better comparison. This was done by calculating a ratio of, ultimate moment of resistance obtained from ACI 318 , BS 8110 and ECP 203-2007 , relative to the M_u value obtained using EC2 (Bakhoum, Mourad & Hassan, 2016).

It was stated that if the relative ratio with respect to EC2 was greater than 1 then the considered code was conservative in flexural design ie, it was less economical. The following table has been adopted from (Bakhoum, Mourad & Hassan, 2016)to illustrate this comparison of Mu with respect to value of Mu in EC2 for a better understanding.

f_{ck} N/mm^2	f_{yk} N/mm^2	ρ %	Values Relative to EC2 Code: ACI/EC2	EC2/EC2	BS 8110/EC2	ECOP 89/EC2	EC2 Value bd^2 N/mm^2
25	360	0.5	1.05	1.0	1.00	1.00	1.48
25	360	1.0	1.06	1.0	1.00	1.00	2.78
25	360	1.5	1.08	1.0	1.00	1.00	3.92
25	500	0.5	1.05	1.0	1.00	1.00	2.01
25	500	1.0	1.08	1.0	1.00	1.00	3.68
25	500	2.0	1.14	1.0	1.00	1.00	6.03
40	360	0.5	1.04	1.0	1.00	1.00	1.51
40	360	1.0	1.05	1.0	1.00	1.00	2.91
40	360	2.0	1.07	1.0	1.00	1.00	5.40
40	500	0.5	1.05	1.0	1.00	1.00	2.07
40	500	1.0	1.06	1.0	1.00	1.00	3.93
40	500	2.0	1.09	1.0	1.00	1.00	7.03

Notes: 1 - The values shown in the last column should be multiplied by bd^2 in (mm) to obtain the ultimate moment of resistance of the sec. in (N.mm).
2 - Columns 4,5,6,7 give relative values with respect to EC2.
3 - The above values are derived for under reinforced sections ($\rho < \rho_{balanced}$).

Figure 4: Comparison of Ultimate moment of resistance of singly reinforced concrete section in four codes (Bakhoum, Mourad & Hassan, 2016)

As mentioned above through this study (Bakhoum, Mourad & Hassan, 2016) deduced that, the ratio of $M_{u\ ACI\ 318}$ to $M_{u\ EC2}$ is greater than 1 for all reinforcement ratios and this difference increases with an increase in the reinforcement ratio (ρ). This difference is about 4% - 14%. This implies that ACI 318 offers a greater ultimate moment of resistance than EC2. On the other hand, the ratios of BS 8110, and ECP 203-2007 equals to 1. This means they all give the same ultimate moment of resistance for a given concrete strength, reinforcement strength and reinforcement ratio. This consistency in results can be seen for EC2, BS 8110 and ECP 203 because all the three codes follow the same equivalent concrete stress-strain block and same material partial safety factors (Bakhoum, Mourad & Hassan, 2016). It can be summarized from this study is that since the $M_{u\ ACI\ 318}$ is higher than other codes, ACI 318-14 requires smaller sections than other considered codes for the same load and material properties. It is more economic as compared to other codes.

The steel reinforcement ratio significantly affects the moment capacity of a section under flexure. Jawad (Jawad, 2006) stated in his paper that EC2 and BS 8110 show a similar linear relationship between moment capacity and reinforcement ratio ρ, attributing to similarity of modelling and safety factors. ACI formula is observed to give a higher moment capacity for lower steel ratios ($\rho \leq 0.03$) unlike BS 8110 and EC2.

3.4 Behaviour of Reinforced Concrete Section in Shear

In spite of continued efforts in the field of research, since the 20th century and before, shear failure modes and its mechanism continues to be one of the least understood approaches in ultimate limit state design. Shear failure comes with a heavy price and could be dangerous. Firstly, because shear failure is brittle and comes without a warning. Secondly, because a beam's flexural strength is greatly reduced due to shear, when compared with the reduction of flexural strength due to pure flexure (Gaetano Russo et al., 2005). However, this complexity of a section in shear exists primarily because several theories have been developed over time. But experimental results govern the design procedures for concrete members as the theories mentioned above have been developed by different researchers and they contradict available previous literatures.

The process of shear is assumed to be initiated by cracks which form at a particular direction which causes the transverse reinforcement along that direction to yield. As this continues, formation of new cracks precedes the yielding of longitudinal reinforcement. The more acute crack is developed than previous ones which leads to collapse of panel due to slipping and crushing of local concrete. This mode of failure is brittle and catastrophic. With regard to the available literature on shear, its failure and mechanism; shear strength of concrete involved multiple variables attributing to its complexity. Variables that affect shear strength include concrete compressive strength, shear span-depth ratio, tensile reinforcement ratio, maximum aggregate size, spacing of flexural cracks and diameter of bars (Kim & Park, 1996) (Rebeiz et al, 2001). It has been widely accepted that three significant variables which affect shear strength are, concrete compressive strength, shear span-depth ratio and tensile reinforcement ratio (Kim & Park, 1996) (Rebeiz et al, 2001). Works by other academicians stressed on the fact that, the effective length-depth ratio of a beam could significantly affect the failure mode in concrete members (Slowik, 2014). In short, what is termed as the 'size effect' which included all dimensions – depth, effective length and width directly impacted the shear resistance in an element (Slowik, 2014). The professional literature does not provide any distinction between the two major components of concrete shear strength, namely, ultimate shear strength and cracking shear strength. (Rebeiz, 1999) stated that the ultimate shear strength and not the cracking shear strength was more reliable in terms of derivation of design equation. The concrete shear strength in a section is governed by the compressive strength of concrete, the ratio of longitudinal reinforcement, depth and width of the section. Many concrete sections are capable of resisting the applied shear stress without any shear reinforcement. In such sections the applied shear stress is resisted by a concrete shear strength which is contributed by three major components – the uncracked concrete in the compression zone, the dowelling action of the longitudinal reinforcement and the aggregate interlock across flexural crack (Jawad, 2006).

To understand the shear failure mechanism, a variety of different approaches were attempted. Provisions of flexural behavior are very similar in most codes. This is because these guidelines are based on some fundamental theories and assumptions, like plane sections remain plane, and the concrete stress-strain diagrams, which do not vary greatly among different international codes. However, this is not the case with shear predictions and provisions, especially for sections without shear reinforcement. The approaches for shear failure mechanism were theoretical investigations developed by researchers. Some of these theories included; shear friction theory (Vecchio & Collins 1986), strut-and-tie modelling (Zhang et al 2009) and the plastic theory (Zhang 1997) (Lucas ,Oehlers and Mohamed Ali, 2011). An attempt is made below to briefly explain what each theory meant.

The shear-friction theory is based on the concept that the shear along the interface planes will be resisted by the friction between the faces. This is done by resistance to shearing off protruding portions of the concrete and by the dowelling action of reinforcement crossing the interface (Shyh-Jiann Hwang, 2000).

The strut-tie modelling of assumed a mechanism of load transfer is – truss analogy. According to this analogy, once inclined cracking occurs, with the compressive zone and the flexural reinforcement forming the longitudinal strut and ties and stirrups forming the transverse ties (Kotsovos, 2007).

The plastic theory proposed by Zhang (Zhang, 1997) was developed using a model to predict the load carrying capacity of RC beams without shear reinforcement (Lucas, Oehlers and Mohamed Ali, 2011). Zhang's model in plastic theory hypothesized two need for two requirements in order for the beam to adopt a plastic theory. First, the applied load must be sufficient to cause the formation of a critical diagonal crack, and secondly, the applied shear load must be enough to cause the sliding and crack (Lucas, Oehlers and Mohamed Ali, 2011).

This is one reason why concrete structures continue to suffer brittle failure in spite of advances in shear design methods (Kotsovos, 2017). Below is a brief narration about how the three significant variables impact the shear strength of concrete.

- Shear span-to-depth ratio

  (Slowik, 2014) in her paper stated that although members may be characterized by the same shear span-to-depth ratio, they still can suffer shear failure attributing to the difference in length to depth ratio. The study carried out by (Slowik, 2014) emphasized the importance of including effective length as variable while determining the shear strength. To be more precise, a/d is a ratio which takes into account the distance between the applied force and the support, and d is the effective depth of the section. While l/d refers to , l_{eff} being the effective length of the beam and d the effective depth of the section. The studies of Taub and Neville (Taub and Neville, 1960) and Kani (Kani, 1966) greatly highlighted the impact of a/d ratio (Karim S. Rebeiz et al., 2001). Olonisakin and Alexander (Alexander and Olonisakin, 1999) made an attempt to study the mechanics of internal shear transfer using reinforced concrete beam specimens with no transverse reinforcement and deduced the co-existence of beam and arching action under shear transfer mechanism. This finding was supported by (Rebeiz, 1999) where it was stressed on including both arch action of short beams and beam action of slender beams for a precise shear mechanism.

- Size effect

  The size of a beam, especially in terms of its depth and slenderness could actually explain the severity at which it can be subjected to shear loads. It is widely observed that deep beams are extremely vulnerable to the shear failure. (Slowik, 2014) carried out experiments to assess the shear capacity of sections, and concluded that, with an increase in the absolute depth of the section there was a decrease in the shear capacity of the section. This meant that increasing the depth of the section to meet design demands, due importance must be given to the shear design of the section.

- Compressive Strength

  Since the time, shear design has gained importance, researchers and academicians always thought that the shear strength of a section was controlled by the compressive strength of the member. This misconception was valid until 1909 when Talbot (Talbot, 1909) provided evidence of other factors influencing shear mechanism. Clark (Clark, 1951) justified in his paper that shear capacity of concrete increased with the compressive strength of concrete, which was contradicted by Kani (Kani, 1967).

(Rebeiz et al., 2001) carried out an in-depth study on the effect of √f'c on the shear strength of concrete. He compared the effect of √f'c on ultimate shear strength and cracking shear strength for both normal strength concrete and high strength concrete. His studies deduced that, compressive strength had no correlation with ultimate strength of concrete in both normal strength beams and high strength beams. There was no correlation with the cracking strength of concrete in high strength beams but there was a weak correlation with the cracking strength of concrete in normal strength beams. This proved that compressive strength alone was not adequate to predict the shear strength of beams

- Tensile reinforcement ratio

Kani et al. (Kani & Elzanaty, et al 1986) concluded in their paper on the significance of tensile reinforcement ratio as a parameter for shear strength. But this was contradicted by (Taub and Neville 1960) who voted that, tensile reinforcement ratio had least impact on the cracking and ultimate shear strength of concrete in the case of short beams when compared with long beams.

Out of all codes, the BS 8110 predicted the best shear strength results. EC2 results were more conservative (less economical) than those given by BS 8110 (Jawad, 2006). The maximum applied shear force is limited, in EC2 and BS 8110, to certain values which are calculated using empirical formulas given in the design codes. These stress values are limited to sections close to the support. Unlike the BS 8110 and the EC2 codes, the ACI 318 limits the amount of shear reinforcement by ensuring that it is not too high (Jawad, 2006). The combined studies of (Kani, 1967), (Zsutty, 1968), (Mphonde and Frantz, 1984), (Ahmad et al., 1986), (Elzanaty et al., 1986), (Sarsam and Al-Musawi, 1992) have shown that the available equation for shear strength prediction do not consider the wholesome contribution of several variables with application possible only with the normal strength concrete and not for high strength concrete.

3.5 Behaviour of Reinforced Concrete Section under Axial Compression

Of all the available literature on comparative study of behavior of a concrete structural element towards pure axial compression was presented well by an analytical study. This analytical study compared the ultimate axial strength of columns, P_u, predicted using provisions of ACI 318, EC2 and BS 8110, for short columns in which the effect of buckling was neglected. This study was done for different concrete compressive strengths and reinforcement ratios. Results showed that EC2 offers the highest axial strength compared to AC1 318 and BS 8110 (as all ratios relative to EC2 are less than 1.0 , it can be interpreted that they offer comparatively less axial strength) (Bakhoum, Mourad & Hassan, 2016). It can be inferred, in simple language, that for the same section dimensions, EC2 provisions yields the highest axial strength compared to ACI 318-14 and ECP 203-2007 (Bakhoum, Mourad & Hassan, 2016). The following figure, Figure 5 was adopted from (Bakhoum, Mourad & Hassan, 2016) to compare the ultimate strength of axially loaded short columns with respect to values from EC2.

f_{ck} N/mm^2	f_{yk} N/mm^2	ρ %	Values Relative to EC2 Code				EC2 Value
			$\frac{ACI}{EC2}$	$\frac{EC2}{EC2}$	$\frac{BS\ 8110}{EC2}$	$\frac{ECOP\ 89}{EC2}$	bd N/mm^2
25	500	1.0	0.78	1.0	0.87	0.77	18.60
25	500	3.0	0.73	1.0	0.87	0.77	27.30
40	500	1.0	0.80	1.0	0.87	0.77	27.15
40	500	3.0	0.75	1.0	0.87	0.77	35.85

Notes: 1- The values shown in the last column should be multiplied by the cross section dim. in (mm) to obtain the ultimate strength of column (Newton).
2- Columns 4,5,6,7 give relative values with respect to EC2.

Figure 5: Comparison of Ultimate Strength of axially loaded short columns (Bakhoum, Mourad & Hassan, 2016)

3.6 Behaviour of Reinforced Concrete Section in combined Flexure and Axial Compression

Most of the international design codes do not give separate provisions for columns under axial compression and columns under a bending moment. Instead most codes adopt interaction diagrams to predict the behavior of such columns. These interaction diagrams are a plot of moment (M) and axial compression (P) along the x and y axis respectively. It is developed for rectangular sections of a specific concrete compressive strength and yield strength of concrete. It is the M-P interaction diagram which actually governs if the section will fail under tension or compression (Jawad, 2006).

We adopt studies that compare the behavior of three columns designed using ACI 318-14, BS 8110 and EC2 provisions.

It is mandatory that every code allows provisions for accidental eccentricities to accommodate for a load that falls at a specified eccentricity unexpectedly. This is considered in most codes by imposing a limitation on column strength, which is generally 0.80 times the calculated strength of tied column in ACI 318 and 0.87 times the calculated strength of tied column in BS 8110 (Jawad, 2006). Following figure shows the interaction diagrams according to ACI 318-14, EC2 and BS 8110.

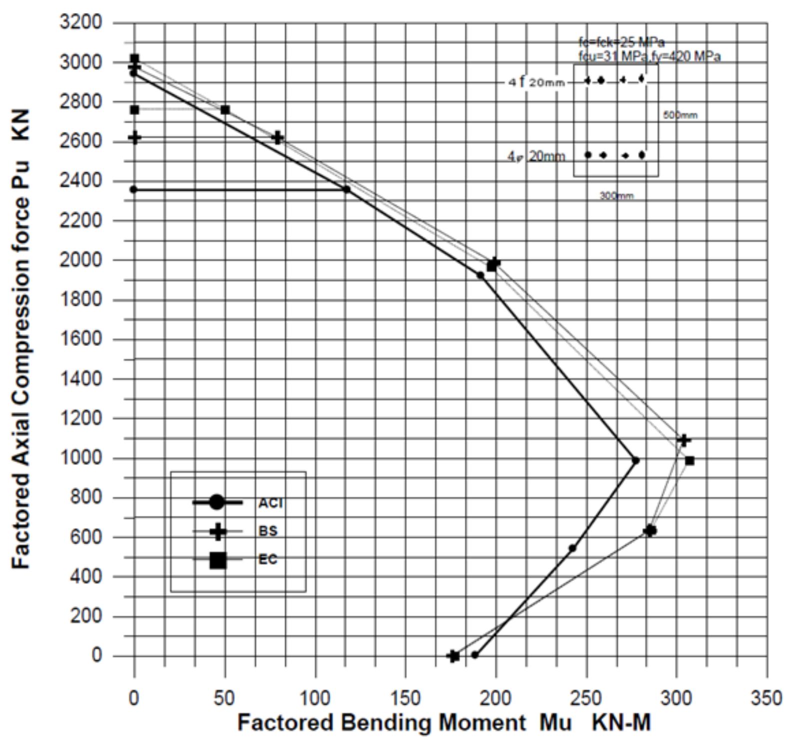


Figure 6: Comparison of M-P interaction diagrams for short columns (Jawad,2006)

The graph above shows that EC2 and BS 8110 follow a similar trend unlike ACI 318-14. This is because of the similarity in distribution of stress blocks, diagrams and material partial safety factors. It is very evident from the figure that ACI code design criteria is less economical and more conservative than other codes (Jawad, 2006).

3.7 Behaviour of Reinforced Concrete Section under Deflection

Generally, span-depth ratios are put forward by every code to control deflection. To closely examine the deflections of members and how they behave needs to determine the short and long-term effects. Deflection of concrete elements such as beams and one way slabs are calculated in terms of short term and long term deflection provisions given in codes. Short term deflections are immediate deflections due to permanent loads or live loads in service condition. Long term deflections are deflections due to creep and shrinkage and creep effects. These provisions are available in codes for beams and one-way slabs.

Deflection control is one of the limit states of designs to be considered during the design of flexural members such as beams and slabs namely. The limit state of deflection, as it is termed in all design codes, is one of the limit states of serviceability. Excessive deflections that arise in slabs and beams is of great concern from the serviceability point of view rather than the safety. This is because, excessive deflections could cause a psychological discomfort among its occupants and tamper the aesthetic appearance of the structure. However, deflections in slabs and beams can also progressively affect all elements it supports, such as blockworks, partitions etc. As a result, codes specify the minimum span/depth ratios for beams and slabs such that they maintain safe deflection limits at an initial stage and in the long run. These ratios cannot be depended explicitly as, they may not give guarantee if the span of the member is too large or if the member is subjected to unusual loads (Menon D., & Pillai U. S., 2005).

Deflections are inversely proportional to flexural rigidity and also depend on long term effects like creep and shrinkage. Most codes specify two limits of deflection. One is the final deflection limit, which includes all loads, effects of temperature, creep, shrinkage etc. While initial deflection is considered after the application of partition and finishes.

Deflections calculation of two-way slabs is a complex task and is not within the scope of this research work. Codes lay down span-depth ratios for two-way slabs although they have large disparities and inconsistencies. No codes provide provisions to calculate the long term deflection of two way slabs whereby recent research has been carried out to calculate effects due to shrinkage and creep using Equivalent Frame Method. (Nayak S.K, 2004) concluded that, the long-term deflection calculation due to creep and shrinkage can be accounted separately using a multiplier on the short-term deflection.

3.8 Behaviour of Reinforced Concrete Section under Cracking

Ever since the advent of codal provisions, structural design to ultimate limit state was significantly considered while the serviceability limit state was often ignored. It has been studied that some modern structures experience premature failures. This is because, modern structures are safe with respect to ultimate limit states. But most structural failures are reported in terms of serviceability. Cracking is undesirable in structures not just because it affects the aesthetic appearance of the structure but also it adversely affects the durability and long-term performance of the structure. Cracking occurs in structures mainly when the tensile strength of concrete is exceeded due to flexural tensile stress, tension due to shear or lateral tensile stress. The degree of cracking is accounted in terms of width and spacing of cracks. Hence, codes stipulate minimum concrete covers for structural elements and crack width limits to limit the cracking and enhance the durability of a structure. Most codes set a maximum crack width limit while, some codes specify a maximum spacing of the tensile reinforcement.

Subramanian (Subramanian, 2005) concluded in his paper that research proves there is no correlation between the crack width and corrosion. He also concluded that BS 8110 and IS 456-2000, use exactly the same equation to calculate the design surface crack widths. However, the ACI 318 until its version in 1995, adopted a similar equation (which included depth of cover, steel strain, area of concrete in tension etc.) to calculate the design crack width which was succeeded by an equation to calculate the maximum spacing of reinforcement closest to the surface.

This research will confine its scope to cracks caused by flexural tensile stresses and their regulations in respective codes. The following chapter discusses the parametric comparison of crack width calculation in different codes.

CHAPTER 4

PARAMETRIC REVIEW OF SOME INTERNATIONAL CONCRETE DESIGN CODE PROVISIONS

This chapter of the research includes a review of the different design provisions and guidelines of some major design codes and primarily focuses on the comparison of different parameters upon which these guidelines are based. Code guideline inputs vary from one to another and so do the output results also. This is because although most design codes use similar design philosophies, they differ in the way these guidelines are developed and which parameters or factors these guidelines depend on. Our objective in this chapter is to compare the provisions of individual structural elements, such as slabs, beams, column, footings etc., laid out by some international design codes namely the ACI 318-14, BS 8110-97 and Eurocode2 as per code provisions. Manual calculations are also carried out for certain parameters to support the findings given in literature.

4.1 Parametric study of slab design provisions

This sub chapter restricts comparison to design provision of one way and two-way slabs according to the ACI 318-14, BS 8110-97 and Eurocode2.

4.1.1 Conditions for one-way and two-way slab

All codes follow the same guidelines to differentiate between a one-way and two-way slab. A ratio of the longer span to the shorter span helps determine if a given slab is one-way or two-way.

One-way slab: $\frac{longer\ span}{shorter\ span} > or = 2$

Two-way slab: $\frac{longer\ span}{shorter\ span} < \ 2$

4.1.2 Minimum thickness provisions for slabs

The critical limit state of design for slabs is the serviceability limit state of deflection. Slab design is governed by deflection than by ultimate limit states of bending, shear and compression. Table 6 presents the empirical formula and the parameters that build the formula for minimum thickness provisions of slabs according to various codes.

Code							
ACI 318-14	Minimum thickness provisions for one way slabs						
	Simply supported	One end continous		Both ends continous		Cantilever	where 'h' = overall slab thickness for normal weight concrete and fy = 60,000psi
	l/20	l/24		l/28		l/10	
	Minimum thickness provisions of two way slabs						
	Without drop panels			With drop panels			fy, psi
	Exterior panels		Interior panels	Exterior panels		Interior panels	
	Without edge beams	With edge beam		Without edge beams	With edge beams		
	$l_n/33$	$l_n/36$	$l_n/36$	$l_n/36$	$l_n/40$	$l_n/40$	40,000
	$l_n/30$	$l_n/33$	$l_n/33$	$l_n/33$	$l_n/36$	$l_n/36$	60,000
	$l_n/28$	$l_n/31$	$l_n/31$	$l_n/31$	$l_n/34$	$l_n/34$	75,000
	where l_n is the clear span in the long direction						

Code			
BS 8110-97	Minimum thickness of both one and two way slabs		
	Support conditions	Rectangular section	* For two way slabs the ratio is based on shorter span ; appropriate modification factor applied for both tension and compression reinforcement (Cl 3.4.6.5 & Cl 3.4.6.6 BS 8110-97)
	Cantilever	7	
	Simply supported	20	
	Continuous	26	

Code					
Eurocode 2	Minimum thickness provisions for slabs				
	Structural system	K	ρ = 1.5%	ρ = 0.5%	If $\rho \leq \rho_o$ $\frac{\ell}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 32\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right]$ If $\rho > \rho_o$ $\frac{\ell}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right]$
	Simply supported slabs	1.0	14	20	
	End span of slabs	1.3	18	26	
	Interior spans of slabs	1.5	20	30	
	Cantilever slabs	0.4	6	8	

Table 6: Minimum thickness provisions of slabs

Most codes specify minimum thickness provisions as a method to control deflection. From the above table, it can be inferred in general that most code provisions the support condition of the slab is an important parameter for determining the minimum thickness of the slab. These support conditions include simply supported, continuous and cantilever slabs. However, some code provisions give a detailed classification on thickness provisions depending on the position of the panel along the slab system considered as in the case of ACI 318-14.

- ACI 318-14 minimum thickness values are independent of applied loading effects and no limits are specified on the range of span to which these guidelines can be applied. The ACI provisions are a function of span length for both one and two-way slabs for different yield strengths of steel.

- BS 8110-97 provisions are common for both one and two-way slabs. These provisions are a function of support condition and applied to a span range of up to 10m.
- The Eurocode2 provisions vary significantly from all other code provisions as it depends on parameters like reinforcement ratio and support conditions of the slab. This is dependent on whether the actual reinforcement ratio ρ is larger than a given reference ratio ρ_o as per guidelines.

Lee and Scanlon (Lee & Scanlon, 2010) performed a parametric analysis of the thickness provisions of ACI 318-08, AS 3600-2001, BS 8110 and Eurocode2. Lee and Scanlon made an attempt to formulate an empirical relation for the minimum thickness of slabs and compared how effective the other code guidelines were. Their proposal was dependent on applied loads, long term multipliers, effects of cracking and target deflection to span limitations and allowable deflections of l/240 and l/480. (Lee & Scanlon, 2010). They varied live loads, deflection limits and span lengths to critically review one-way slabs and edge supported two-way slabs. According to their studies, all provisions except ACI 318 considered variations of live load in establishing minimum thickness. All provisions except ACI 318 showed a general trend of decreasing span- depth ratio with increasing span length. These studies deduced that the ACI provisions were satisfactory in most cases for l/480 deflection limit except for simply supported case of one-way slabs under a live load of 70psf and span of 20feet

(Lee & Scanlon, 2010). Our parametric study of slab provisions shows that only few codes consider the effect of loads on the thickness provisions. Nayak (Nayak S.K, 2004) suggested that, no codes provide provisions to calculate the long-term deflection of two-way slabs whereby recent research has been carried out to calculate effects due to shrinkage and creep using Equivalent Frame Method.

The thickness provisions of flat slabs are relatively similar to those of solid slabs except for variations in the ratio of span/depth.

4.1.3 Minimum reinforcement provisions for slab

This sub chapter would discuss the minimum reinforcement provisions enforced by different codes for slab design, one-way slab design in particular. This refers to the minimum percentage of steel required for one-way slabs, as a percentage of area of concrete section. Table 7, presents the minimum percentage of reinforcements for one-way slabs according to different international codes.

<table>
<tr><th colspan="6">Minimum reinforcement of slab (One way slab)</th></tr>
<tr><th>Standard</th><td colspan="5">Minimum percentage of reinforcement</td></tr>
<tr><th rowspan="4">ACI 318-14</th><td>Reinforcement type</td><td>fy, psi</td><td colspan="3">As,min</td></tr>
<tr><td>Deformed bars</td><td>< 60,000</td><td colspan="3">0.0020Ag</td></tr>
<tr><td rowspan="2">Deformed bars or welded wire reinforcement</td><td rowspan="2">$\geq$ 60,000</td><td rowspan="2">Greater of:</td><td colspan="2">$\frac{0.0018 \times 60,000}{fy}$ Ag</td></tr>
<tr><td colspan="2">0.0014Ag</td></tr>
<tr><th rowspan="3">BS 8110-97</th><td>Reinforcement type</td><td>Minimum %</td><td colspan="3" rowspan="3">where % refers to 100As/Ac</td></tr>
<tr><td>fy = 250 N/mm^2</td><td>0.24</td></tr>
<tr><td>fy = 460 N/mm^2</td><td>0.13</td></tr>
<tr><th>Eurocode2</th><td colspan="2">$A_{s,min} = 0,26 \frac{f_{ctm}}{f_{yk}} b_t d$ but not less than $0,0013 b_t d$</td><td colspan="3">where b_t denotes mean width of tension zone</td></tr>
</table>

Table 7: Comparison of Minimum reinforcements for one-way slab

The ACI minimum reinforcement requirements depend on the yield strength of reinforcement steel or the grade of steel. The British, Indian and Egyptian codes are also based on the same guidelines as the ACI in terms of different reinforcement percentages for different grades of steel. All these reinforcement percentages are with respect to the area of gross concrete section.

4.1.4 Punching shear provisions for flat slab

Critical control perimeter for punching

The critical control perimeter is that perimeter within which the flat slab is susceptible to punching shear from concentrated axial loads on columns. Hence, the punching shear strength of a flat slab is checked within this perimeter. The critical perimeter is calculated at a distance ‘d’ from the column face. This distance varies from one half to double effective depth of the slab. Following table, Table 8, shows the distance ‘d’ for various codes.

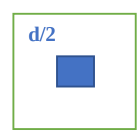
Critical control perimeter for flat slabs		
Standard	**Distance of critical control perimeter from the face of the column**	
ACI 318-14	d/2 from the face of the column	
BS 8110-97	1.5d from the face of the column	
Eurocode2	2d from the face of the column	
where, d is the effective depth of the slab		

Table 8: Comparison of critical control perimeter in different codes

- It can be seen that all the three codes follow different spacing for critical control perimeter.
- However, BS 8110 recommends a higher distance from face of column and Eurocode2 recommends the highest distance of two times the distance from

the face of the column. ACI 318 offers the least distance from face of column.

- Bartolac et al (Bartolac et al., 2015) suggested that lower control perimeter values resulted in lower slab punching force.

4.2 Parametric study of beam design provisions

For the feasibility of comparison, this comparative study is restricted to the design guidelines of singly reinforced rectangular beam according to the ACI 318-14, BS 8110-97 and Eurocode2.

4.2.1 Span-depth ratio provisions of beams

Following Table 9, shows the span/depth ratio comparison of different codes and their parameters.

Span/Depth ratio of Beams (Singly reinforced beams)					
Standard	Minimum depth of non-prestressed beams				
ACI 318-14	Support condition	Minimum h			* applicable for normal concrete weight and fy = 60,000 psi
	Simply supported	l/16			
	One end continuous	l/18.5			
	Both ends continuous	l/21			
	Cantilever	l/8			
BS 8110-97	Basic span/effective depth ratio for rectangular beams				
	Support condition	s/d ratio			* For two way slabs the ratio is based on shorter span ; appropriate modification factor applied for both tension and compression reinforcement (Cl 3.4.6.5 & Cl 3.4.6.6 BS 8110-97)
	Cantilever	7			
	Simply supported	20			
	Continuous	26			
Eurocode2	Minimum thickness provisions for rectangular beams				
	Structural system	K	ρ = 1.5%	ρ = 0.5%	If $\rho \leq \rho_o$ $\frac{\ell}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 32\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right]$ If $\rho > \rho_o$ $\frac{\ell}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right]$
	Simply supported slabs	1.0	14	20	
	End span of slabs	1.3	18	26	
	Interior spans of slabs	1.5	20	30	
	Cantilever slabs	0.4	6	8	
	*The ratios are obtained from l/d ratios for different ρ				

Table 9: Comparison of span/depth ratio of beams

The span/depth provisions of all three codes are different.

- Guidelines presented by BS 8110-97 depends merely on the support condition of the beam and the span range it is applicable to. Modification factors may be applied for tension and compression reinforcement.
- ACI 318 provisions just like those of the slab minimum thickness provisions, depends on the span length of the beam and support condition.
- The Eurocode2 provisions for span/depth ratio of the beam are similar to that of minimum thickness provisions for slab. Eurocode2 provision depends on parameters like reinforcement ratio and support conditions of the slab. This is dependent on whether the actual reinforcement ratio ρ is larger than a given reference ratio ρ_o as per guidelines.

4.2.2 Ultimate design moment of resistance

The ultimate moment of resistance (M_u) for a section in pure flexure is compared here. The parameters that govern the ultimate moment of resistance vary from code to code. The following table, Table 10 presents the empirical formulas for ultimate moment of resistance formulas recommended by different codes. This tabulation helps us relate the parameters that govern M_u according to different codes.

Ultimate Moment of Resistance of a rectangular beam in flexure (Singly reinforced beam)		
Standard	Empirical formula	
ACI 318-14	$M_u = \phi$ Asfy (d - a/2)	where, ϕ = Strength reduction factor ; As = area of longitudinal tension reinforcement in^2 ; fy = specified yield strength of reinforcment in psi; d= distance from extreme compression fibre to centroid of longitudinal reinforcement in. ; a = depth of equivalent rectangular stress block in.
BS 8110-97	M_u = 0.156fcubd^2	where, fcu = characteristic strength of concrete ; b = the width of the rectangular beam section ; d = effective depth of the section
Eurocode2	M_{Rd} = 0.167fcubd^2	where, fcu = characteristic strength of concrete ; b = the width of the rectangular beam section ; d = effective depth of the section
* The cylindrical strength is assumed as 0.8 times the cube strength		

Table 10: Comparison of Ultimate Moment of Resistance (M_u)

- ACI 318-14 provisions are distinctly different from the other two code provisions. In ACI 318, the equation is multiplied by a strength reduction factor to get M_u.
- BS 8110 and Eurocode2 provisions are both, functions of the characteristic strength of concrete, width of section and effective depth of section.

A hand calculation was carried out for a beam of cross-section 15” X 30” and a span of 30’, to determine the Ultimate Moment of Resistance of a section at a given loading for the above section using three different code provisions as given in Table 10. It was concluded that,

- BS provision gave maximum value for ultimate moment of resistance followed by ACI. Both code provisions vary marginally.

- Eurocode2 provisions gave a value that varied widely from BS and ACI values.
- The detailed calculation has been attached in the APPENDIX B.

4.2.3 Area of tension reinforcement in singly reinforced beams

The empirical calculation of required area of reinforcement in singly reinforced beams vary significantly.

Area of tension reinforcement (Singly reinforced beams)		
Standard	Empirical formula	
ACI 318-14 *	As = $\frac{Mu}{\varphi fy\,(d - \frac{a}{2})}$	where Mu = design moment; ϕ = strength reduction factor; fy = yield strength of steel; d = depth of section
BS 8110-97	As = $\frac{Mu}{0.87fy\,z}$	where Mu = ultimate design moment; fy = yield strength of steel reinforcement; z = lever arm distance = 0.95d
Eurocode2	A_{s1} = M / 0.87fyz	where M = ultimate design moment; fy = yield strength of steel reinforcement; z = lever arm distance = 0.95d
* The cylindrical strength is assumed as 0.8 times the cube strength		

Table 11: Comparison of formula for the area of tension reinforcement

- ACI 318 provision is a function of just two parameters: the compressive strength of concrete and the yield strength of steel.
- In the BS and Eurocode2, A_{st} guidelines are a function of design moment, yield strength of steel and lever arm distance.

A hand calculation was carried out for a beam of cross-section 15” X 30” and a span of 30’, to determine the required area of tension reinforcement of a section at a given loading for the above section using three different code provisions as given in Table 11. It was concluded that,

- All the three codes gave very closely agreeing results with minute variation.
- Maximum area of tension reinforcement was given by ACI provisions and least by BS and EC2 provisions.

- Required reinforcement areas given by BS and ACI codes are closely similar attributing to the similar empirical formula to determine area of tension steel. The detailed calculation has been attached in the APPENDIX C.

4.2.4 Minimum and maximum reinforcement provisions for beams

Sometimes a section may be larger than required for flexural strength requirements to satisfy architectural needs. Such members are susceptible to brittle facture if they have too less steel. This is one reason why codes prescribe a minimum limit of tension steel. Similarly, codes also prescribe a maximum limit of tension reinforcement to avoid compression failure of concrete before the tension failure of steel. The minimum and maximum reinforcement provisions are a function of the gross area of the concrete section.

Minimum and Maximum reinforcement provisions for beams			
Standard	Minimum tensile steel for flexure As / $b_w d \geq$		Maximum tensile steel for flexure $\leq$
ACI 318-14 *	$\frac{0.224\sqrt{fck}}{fy} \geq \frac{1.4}{fy}$		Net tensile strain in extreme tensile steel $\geq$ 0.005
BS 8110-97	Steel Reinforcement	$\frac{100As}{Ac} \geq$	0.04bD
	fy = 250 N/mm^2	0.0024	
	fy = 460 N/mm^2	0.0013	
Eurocode 2	$\frac{0.26fctm}{fy} \geq$ 0.0013		0.04bD
* The cylindrical strength is assumed as 0.8 times the cube strength			

Table 12: Minimum and maximum reinforcement provision for beams

The minimum tensile steel for flexure according to different codes is presented by different formulas for different codes. However, the maximum steel requirement is presented as a function of area of gross concrete section. The tabulation in Table 12, shows that the longitudinal steel provisions of the Indian code depends only on the yield strength of steel and is independent of concrete strength unlike other codes, in which the minimum steel requirement is a function of both yield strength of steel and concrete strength.

Subramanian (Subramanian N, 2010) compared the Indian code provisions of limiting reinforcement ratios with those of ACI 318 and suggested that, minimum reinforcement ratios which depend only on the yield strength of steel cannot efficiently determine the minimum steel required in a section specially when it comes to high strength concrete members. Hence, Subramanian suggested to revise the minimum steel provisions of IS 456. 4 percentage of maximum tension reinforcement in flexural members according to BS 8110 and EC2 are also on the higher side. However, the ACI 318 limits the maximum tension steel in a flexural member such that the member remains ‘tension controlled’ and it does not fail under concrete compression at ultimate loads.

4.2.5 Shear resistance in beams without shear reinforcement

Minimum shear reinforcement

Transverse reinforcement helps restrain the development and propagation of shear cracks and increase ductility. The minimum shear reinforcement permitted by different codes vary significantly unlike the tension reinforcement requirements. This anomaly can be attributed to the inconsistency between the shear design guidelines of each code and the fundamental theories of concrete which has been dealt with in the previous section. The following table, Table 13, shows the minimum shear reinforcement requirements as per different codes.

Minimum shear reinforcement		
Standard	Minimum shear reinforcement As / $b_w s_v \geq$	
ACI 318-14*	$\frac{0.9\sqrt{fck}}{16fy} \geq \frac{0.33}{fy}$	when applied shear is greater than 0.5 times concrete strength
BS 8110-97	$\frac{0.4}{0.87fyv}$	when only minimum links are required
Eurocode2	$\frac{0.08\sqrt{fck}}{fy}$	when applied shear is less than shear strength of concrete
* The cylindrical strength is assumed as 0.8 times the cube strength		
* * Area of tension reinforcement is calculated in ECP203-2007 using R-ω curves and charts		

Table 13: Comparison of minimum shear reinforcement requirement

It can be seen that the minimum shear reinforcement requirements vary widely among different codes. Some code empirical formulas include the dimensions of the section while, some include only the yield strength of steel and the compressive strength of concrete. Jawad (Jawad, 2006) recommends in his paper that the ACI code stands unique as it limits the minimum applied shear stress at a section by limiting the shear strength provided by shear reinforcement so that the amount of shear is not too high. The minimum shear reinforcement provisions of British and Indian standards are independent of the compressive strength of concrete, unlike other codes.

4.3 Parametric study of column design provisions

This sub chapter of the parametric study deals with the comparison of design provisions of columns confirming to standards of the ACI 318-14, BS 8110-97 and Eurocode2. In this study we adopt the design provisions for a short-braced column.

4.3.1 Comparison of slenderness ratio

Slenderness ratio is a significant parameter in the design of columns. This is one parameter which determines if the column under consideration is predominantly governed by buckling. It hence, differentiates columns as slender and short. The following table, Table 14, shows the slenderness ratio according to three different codes and the parameters that influence the calculation of slenderness ratio.

Slenderness ratio		
ACI 318-14	$\frac{klu}{r}$	where, k = effective length factor; lu = unsupported length of column; r = radius of gyration
BS 8110-97	$\frac{lex}{h}$ or $\frac{ley}{b}$	where, lex and ley = effective length along the x and y spans; b = width of section; h = depth of section
Eurocode2	$\lambda = l_o / i$	l_o = effective length of column; i= radius of gyration of uncracked section

Table 14: Comparison of slenderness ratio of columns

It can be inferred from the above table that most codes adopt a ratio which is a function of, the effective length of the column along either x or y direction upon the depth or width of the section. Some codes like the ACI 318 and the Eurocode2 consider the effect of radius of gyration.

4.3.2 Condition for short column

It can be deduced from the slenderness ratio discussed above if a particular column is a short column or not. This is practically possible by setting a limit to the slenderness ratio. If the ratio is exceeded beyond the stipulated ratio the column is termed as a short column. The table, Table 15, below collates the short column conditions for the three considered codes.

Condition for short column		
ACI 318-14	$\frac{klu}{r}$ < 34 braced $\frac{klu}{r}$ < 22 unbraced	where, k = effective length factor; lu = unsupported length of column; r = radius of gyration
BS 8110-97	$\frac{lex}{h} < 15\ or$ $\frac{ley}{b} < 15$ (Braced) $\frac{lex}{h} < 10\ or$ $\frac{ley}{b} < 10$ (Unbraced)	where, lex and ley = effective length along the x and y spans; b = width of section; h = depth of section
Eurocode2	$\lambda < \lambda_{lim}$	l_o = effective length of column; i= radius of gyration of uncracked section

Table 15: Comparison of condition for short column effects

This table shows that except the Eurocode which sets a limit λ_{lim}, all other codes provide a ratio for both braced and unbraced columns to be differentiated as short columns. This means that it is required that the ratio of slenderness falls below the given value. This λ_{lim} value is given by,

$$\lambda_{lim} = 20*A*B*C / \sqrt{n}$$

where, the λ_{lim} value depends on the effective creep ratio, load sharing capacity between concrete and reinforcement and end moment ratio (Ekneligoda et al, 2007). This shows that the calculation of λ_{lim} is a very detailed process unlike any codes and considers several parameters into consideration. While other codes simply include a ratio of the effective height of the column and one of their lateral dimensions, the Eurocode2 provisions seem more sophisticated in this approach.

4.3.3 Design Ultimate Axial load

The design ultimate axial load is the ultimate factored load that acts on the column for which the column is designed to calculate the resultant moments. The following table sums the equations for ultimate axial load according to the three different codes.

It can be noted that all of the codes, give an ultimate axial load value that is the sum of a factor times concrete and steel counterparts. Except that the code multiplication factor changes from code to code, the formulation of the empirical formula remains the same.

A hand calculation was carried out for a column section 200 X 500 with an A_{st} value equal to 1610 mm^2, at a given loading for the above section using three different code provisions as given in Table 16. It was concluded that,

- Maximum axial strength value was given by Eurocode2 provisions and varied distinctly from other code provision values.
- BS code values varied slightly with respect to ACI 318 values. ACI values was greater than BS code value.

  The detailed calculation has been attached in the APPENDIX D.

Design ultimate axial load	
ACI 318-14	Pumax = 0.8 [0.85f'c (Ag - Ast) + fyAst] where f'c is the concrete compressive strength; Ag = area of gross concrete section; Ast = area of tension reinforcement; fy = yield strength of reinforcement.
BS 8110-97	N = 0.35fcuAc + 0.7Ascfy where fcu is the concrete compressive strength; Ac = area of gross concrete section; Asc = area of tension reinforcement; fy = yield strength of reinforcement.
Eurocode2	Pu = 0.57fckAc + 0.87fykAs where fck is the concrete compressive strength; Ac = area of gross concrete section; As = area of tension reinforcement; fyk = yield strength of reinforcement.
* The cylindrical strength is assumed as 0.8 times the cube strength	

Table 16: Comparison of design ultimate axial load in different codes

At times, when the column is subjected to both an axial load and a bending moment ie, flexure, then the M-N diagram or interaction charts are preferred to calculate the reinforcement area in column design. The BS 8110 and Eurocode2 provisions are in close agreement with each other. However, (Ekneligoda et al, 2007) in their paper studied a comparison of the column design guidelines of both BS 8110 and Eurocode2 and concluded that according to a comparison of the M-N interaction charts at the balance point , a marginal decrease of normal load was observed in Eurocode2 and a 15% increase of moment was observed , indicating that Eurocode2 would overestimate the flexural capacity. Jawad (Jawad, 2006) studied the M-P interaction curves, (another notation for M-N interaction charts) and concluded that the ACI code design criteria is less economical and more conservative.

4.3.4 Minimum and maximum longitudinal reinforcement ratio of columns

The minimum and maximum longitudinal reinforcements are calculated as a percentage of the gross cross-sectional area of concrete section. It can be noted from the table below that ACI recommends both minimum and maximum reinforcements higher than the other codes. The least ratio is provided by the Eurocode2 provisions. However, it is mandatory that the limits set by the codes should not be violated.

Most codes require a minimum of four bars in a rectangular column and a minimum of six bars in a circular column (Jawad, 2006).

Minimum and maximum longitudinal reinforcement ratio in columns		
	Min reinforcement ratio	**Max reinforcement ratio**
ACI 318-14	0.01	0.08
BS 8110-97	0.004	0.06
Eurocode2	0.002	0.04

Table 17: Comparison of minimum and maximum longitudinal reinforcement

4.3.5 Minimum eccentricity requirements for column design

There are namely two types of failures that occur due to eccentricity. These are tensile failure, as a result of large effective eccentricity; and compression failure which is more likely under a small eccentricity. But it is required by most codes to allow for accidental eccentricities of loading (Jawad, 2006). Jawad (Jawad, 2006) in his paper discussed that codes generally impose an upper limit on the limit of pure axial column capacity which is less than the calculated ultimate strength, which is 0.8 times the calculated strength of tied column according to ACI and 0.87 as in BS 8110.

Minimum eccentricity e_{min}	
ACI 318-14	$e_{min} = h/30 \geq 20mm$
BS 8110-97	e_{min} = 0.05 * overall dimension of column in the plane of bending $\leq$ 20mm
Eurocode2	$e_o = h/30 \geq 20mm$

Table 18: Comparison of minimum eccentricity in different codes

All codes mention minimum eccentricity value with respect to the overall dimension of the column along the plane of bending.

4.4 Parametric study of crack width provisions

Cracking as mentioned in the previous chapter is significant not just because it affects the aestheticity of the building but also because it affects durability and performance of building in the long run.

This sub chapter would focus on flexural cracking and the variables affecting crack width such as steel reinforcement, concrete cover, flexural reinforcement ratio and arrangement of rebar (Allam et al., 2012). There are several parameters that affect the cracking of the cover of members, namely cover depth, concrete quality and crack width. The following table, Table 19, collates the equations used by different building codes and their approaches to calculate the crack width of a section.

Crackwidth provisions in different codes	
Standard	**Formula**
ACI 318-08/ACI318-05	$s = 380 (280 / f_s) - 2.5c \leq 300 (280 / f_s)$
	where, s = maximum spacing of reinforcement closest to tension face,mm
	c = least distance from surface of reinforcement to tension face, mm
BS 8110-2-85	$W_d = (3 * a_{cr} * \varepsilon_m) / \{1 + 2 [(a_{cr} - c_{min}) / (h - x)]\}$
	where, W_d = Design surface crack width
	a_{cr} = distance from the point considered to the surface of nearest longitudinal bar
	ε_m = average strain at the level of cracking
	c_{min} = minimum cover to tension steel
	h = overall depth of member
	x = depth of neutral axis
Eurocode2	$W_k = S_{rmax} (\varepsilon_{sm} - \varepsilon_{cm})$
	where, W_k = design crack width, mm
	$\varepsilon_{sm} - \varepsilon_{cm}$ = mean steel strain
	S_{rmax} = maximum crack spacing in mm

Table 19: Comparison of crack width provision in different codes

It can be deduced from the above table that most codes derive their crack width equations by multiplying the maximum crack width with the mean strain in flexural steel reinforcement. Hence, crack width significantly depends on the arrangement of bars crossing the cracks and the bond between the concrete and steel.

4.5 Parametric study of deflection provisions in beams and slabs

Deflection as mentioned in previous chapter is a significant limit state in terms of serviceability. Deflections to structural members can cause psychological discomfort among its occupants and tampers the aesthetic appearance. Deflections can also cause damages to nonstructural elements connected or supported by the deflected member. Hence codes specify span/depth ratios for structural members like beams and slabs as discussed in the previous sub chapters.

But, a maximum permissible deflection limit is essential for members subjected to flexure whereby, deflections could adversely affect the strength or serviceability of the structure. These limitations should include immediate and time-dependent deflections considering all loads – live loads and effects of temperature, creep and shrinkage. The following table, collates the permissible maximum deflection limits recommended by three different codes.

It can be inferred from the table that most codes recommend different deflection limits for members that support non-structural members and for those that do not support nonstructural elements as well. All codes specify two deflection limits namely, immediate deflection and time-dependent deflection. All codes show a relatively similar trend in deflection limits. Codes also recommend computing total deflection of a member as a sum of short-term and long-term deflections using empirical formulas. This detailed computation will be discussed in the following chapter.

<table>
<tr><th>Standard</th><th colspan="5">Maximum permissible deflections</th></tr>
<tr><td rowspan="6">ACI 318-14</td><td>Member</td><td colspan="2">Condition</td><td>Deflection</td><td>Limit</td></tr>
<tr><td>Flat roofs</td><td colspan="2" rowspan="2">Not attached to or donot support any non-structural elements which are likely to be damaged under large deflections</td><td>Immediate deflection due to max of Lr, S and R</td><td>$\ell/180$</td></tr>
<tr><td>Floors</td><td>Immediate deflection due to L</td><td>$\ell/360$</td></tr>
<tr><td rowspan="2">Roofs or floors</td><td rowspan="2">Supports or attached to non structural elements</td><td>Likely to be damaged by large deflections</td><td rowspan="2">Total deflection occuring after attachment of all non-structural elements (sum of time dependant deflection, due to sustained loads and immediate deflection, due to any additional live loads)</td><td>$\ell/480$</td></tr>
<tr><td>Not likely to be damaged by large deflections</td><td>$\ell/240$</td></tr>
<tr><td colspan="5"></td></tr>
<tr><td rowspan="3">BS 8110-85</td><td>Member</td><td colspan="2">Deflection</td><td colspan="2">Limit</td></tr>
<tr><td rowspan="2">Beams or slabs</td><td colspan="2">Deflection occuring after the construction of finishes and partitions</td><td colspan="2">lesser of $\begin{cases} \frac{span}{500} \quad or \\ 20mm \end{cases}$</td></tr>
<tr><td colspan="2">Total deflection</td><td colspan="2">span / 250</td></tr>
<tr><td rowspan="3">Eurocode2</td><td colspan="3">Condition</td><td>Member</td><td>Limit</td></tr>
<tr><td colspan="3">Initial deflection (under quasi-permanent loads)</td><td rowspan="2">Beams, slabs, cantilevers</td><td>span/500</td></tr>
<tr><td colspan="3">Total deflection (under quasi-permanent loads)</td><td>span/250</td></tr>
</table>

Table 20: Maximum permissible deflection limit provision of different codes

4.6 Parametric study of durability provisions

This sub chapter discusses the durability provisions for RC structures in three different codes ACI 318, BS 8110 and Eurocode2. These provisions are based on a minimum grade of concrete, maximum water-cement ratio and minimum concrete cover – which are deemed to satisfy (Anoop et al., 2001).

To enhance the durability of a structure at the design stage, it is required by the ACI code to provide the appropriate w/cm (water-cementitious ratio) and composition of the material. Most codes specify on the w/cm ratio to achieve low permeability. This is possible only if a consistency is maintained between the selected value of f'c and the maximum w/cm ratio to achieve the targeted durability. The objective is to ensure that the maximum w/cm ratio is not exceeded. These specifications help maintain a good quality of concrete.

4.6.1 ACI 318 -14

The following figure, clearly defines the different exposure categories as in ACI 318 (ACI Committee, 2005)

Category	Class	Condition	
Freezing and thawing (F)	F0	Concrete not exposed to freezing-and-thawing cycles	
	F1	Concrete exposed to freezing-and-thawing cycles with limited exposure to water	
	F2	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water	
	F3	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water and exposure to deicing chemicals	
Sulfate (S)		Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass[1]	Dissolved sulfate (SO_4^{2-}) in water, ppm[2]
	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \leq SO_4^{2-} \leq 2.00$	$1500 \leq SO_4^{2-} \leq 10,000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10,000$
In contact with water (W)	W0	Concrete dry in service Concrete in contact with water and low permeability is not required	
	W1	Concrete in contact with water and low permeability is required	
Corrosion protection of reinforcement (C)	C0	Concrete dry or protected from moisture	
	C1	Concrete exposed to moisture but not to an external source of chlorides	
	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources	

Exposure class	Examples
F0	• Members in climates where freezing temperatures will not be encountered • Members that are inside structures and will not be exposed to freezing • Foundations not exposed to freezing • Members that are buried in soil below the frost line
F1	• Members that will not be subject to snow and ice accumulation, such as exterior walls, beams, girders, and slabs not in direct contact with soil • Foundation walls may be in this class depending upon their likelihood of being saturated
F2	• Members that will be subject to snow and ice accumulation, such as exterior elevated slabs • Foundation or basement walls extending above grade that have snow and ice buildup against them • Horizontal and vertical members in contact with soil
F3	• Members exposed to deicing chemicals, such as horizontal members in parking structures • Foundation or basement walls extending above grade that can experience accumulation of snow and ice with deicing chemicals

Figure 7: Exposure categories and classes in ACI 318-14 (ACI 318-14)

Above is the Table 19.3.1 of Section 19 in ACI 318-14 (Committee, 2005). The exposure categories are subdivided to exposure classes depending on the severity of exposure. For example, in the term F0, F refers to the exposure category and 0 refers to the exposure classes.

Exposure class	Maximum w/cm[1]	Minimum f_c', psi	Additional requirements			Limits on cementitious materials
			Air content			
F0	N/A	2500	N/A			N/A
F1	0.55	3500	Table 19.3.3.1			N/A
F2	0.45	4500	Table 19.3.3.1			N/A
F3	0.40[2]	5000[2]	Table 19.3.3.1			26.4.2.2(b)
			Cementitious materials[3] — Types			Calcium chloride admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No type restriction	No type restriction	No type restriction	No restriction
S1	0.50	4000	II[4][5]	Types IP, IS, or IT with (MS) designation	MS	No restriction
S2	0.45	4500	V[5]	Types IP, IS, or IT with (HS) designation	HS	Not permitted
S3	0.45	4500	V plus pozzolan or slag cement[6]	Types IP, IS, or IT with (HS) designation plus pozzolan or slag cement[6]	HS plus pozzolan or slag cement[6]	Not permitted
W0	N/A	2500	None			
W1	0.50	4000	None			
			Maximum water-soluble chloride ion (Cl^-) content in concrete, percent by weight of cement[7]		Additional provisions	
			Nonprestressed concrete	Prestressed concrete		
C0	N/A	2500	1.00	0.06	None	
C1	N/A	2500	0.30	0.06		
C2	0.40	5000	0.15	0.06	Concrete cover[8]	

Figure 8: Maximum w/cm, minimum f'c, air content and maximum water-soluble chloride ion for different exposure conditions in ACI 318-14 (ACI 318-14)

Above is an image of the Table 19.3.2.1 in Section 19 of ACI 318-14 (Committee, 2005). It specifies the maximum water cement ratio for the corresponding f'c, the air content requirement, specification of cementitious material, maximum water-soluble chloride ion and cover requirement. It also provides standards for the maximum w/cm ratio corresponding to the respective exposure conditions.

The figure below specifies on the concrete cover required for corrosion protection.

Type of member	*Cover thickness, in (mm)*
Reinforced concrete	
Walls and slabs	2 (51)
Other members	2.5 (64)
Precast concrete under plant control	
Walls and slabs	1.5 (38)
Other members	2 (51)

Figure 9: Minimum cover requirements for concrete members in ACI 318-14 (ACI 318-14)

4.6.2 BS 8110-97

According to the British standards, there are five categories of exposure and these categories define the environment the structure is exposed to. Depending on the exposure categories, the nominal cover for each exposure condition has been specified. Following figures, elaborate on the exposure condition and the nominal covers.

Environment	**Exposure conditions**
Mild	Concrete surfaces protected against weather or aggressive conditions
Moderate	Exposed concrete surfaces but sheltered from severe rain or freezing whilst wet Concrete surfaces continuously under non-aggressive water Concrete in contact with non-aggressive soil (see sulfate class 1 of Table 7a in BS 5328-1:1997) Concrete subject to condensation
Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing or severe condensation
Very severe	Concrete surfaces occasionally exposed to sea water spray or de-icing salts (directly or indirectly) Concrete surfaces exposed to corrosive fumes or severe freezing conditions whilst wet
Most severe	Concrete surfaces frequently exposed to sea water spray or de-icing salts (directly or indirectly) Concrete in sea water tidal zone down to 1 m below lowest low water
Abrasive[a]	Concrete surfaces exposed to abrasive action, e.g. machinery, metal tyred vehicles or water carrying solids
NOTE 1 For aggressive soil and water conditions see 5.3.4 of BS 5328-1:1997. NOTE 2 For marine conditions see also BS 6349. [a] For flooring see BS 8204.	

Figure 10: Exposure classes and description in BS 8110-97 (BS 8110-97)

Conditions of exposure (see 3.3.4)	Nominal cover Dimensions in millimetres				
Mild	25	20	20[a]	20[a]	20[a]
Moderate	—	35	30	25	20
Severe	—	—	40	30	25
Very severe	—	—	50[b]	40[b]	30
Most severe	—	—	—	—	50
Abrasive	—	—	—	See NOTE 3	See NOTE 3
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content (kg/m^3)	275	300	325	350	400
Lowest grade of concrete	C30	C35	C40	C45	C50

NOTE 1 This table relates to normal-weight aggregate of 20 mm nominal size. Adjustments to minimum cement contents for aggregates other than 20 mm nominal maximum size are detailed in Table 8 of BS 5328-1:1997.

NOTE 2 Use of sulfate resisting cement conforming to BS 4027. These cements have lower resistance to chloride ion migration. If they are used in reinforced concrete in very severe or most severe exposure conditions, the covers in Table 3.3 should be increased by 10 mm.

NOTE 3 Cover should be not less than the nominal value corresponding to the relevant environmental category plus any allowance for loss of cover due to abrasion.

[a] These covers may be reduced to 15 mm provided that the nominal maximum size of aggregate does not exceed 15 mm.

[b] Where concrete is subject to freezing whilst wet, air-entrainment should be used (see 5.3.3 of BS 5328-1:1997) and the strength grade may be reduced by 5.

Figure 11: Minimum cover requirements for concrete sections in BS 8110-97 (BS 8110-97)

4.6.3 Eurocode2 (BS EN 1992-1-1)

The Eurocode2 prescribes a very detailed classification of exposure conditions as in the following figures. The later elaborates on the limits and minimum requirements of the various aspects of durability.

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

Figure 12: Different exposure conditions in Eurocode2 (BS EN 1992-1-1)

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Figure 13: Minimum cover requirements for different exposure classes and structural classes in Eurocode2 (BS EN 1992-1-1)

4.7 Discussions and recommendations of the parametric study

The parametric study's objective was primarily the comparison of design provisions, equations and design methodologies adopted by the three concrete design codes. The parametric study covered the primary design provisions of slabs, beams and column design which include flexural design, flexural reinforcement, shear design, shear reinforcement and minimum thickness guidelines. It also brushes through some critical design provisions of punching shear in flat slabs. The parametric study also compares the deflection and crack width guidelines recommended by different codes. Following are some major inferences drawn from the parametric study of design provision and comparison of results from ACI 318-14, BS 8110-97 and Eurocode2 (BS EN 1992-1-1).

4.7.1 Load, partial safety factors and load combinations

- The variable actions of ACI 318 and ASCE 7-10 are more conservative than other codes for corridors and stairs of residential and office buildings and floors, corridors and stairs of shops.
- Most codes follow a live load safety factor of 1.6 except the Eurocode2 which recommends 1.5.
- The dead load factors are least in ACI 318 and ASCE 7-10 followed by Eurocode2 and BS 8110-97. The BS 8110-97 gives the highest dead load safety factor.

- The maximum ultimate design load is calculated using provisions of the British code.
- Most codes follow the same material safety factor provisions for concrete and steel, 1.5 (flexural) and 1.25 respectively. However, the ACI 318 does not recommend a material safety factor for specific materials but, a total strength reduction factor φ applied to the nominal moment. The φ factor depends on stress condition.

4.7.2 Slab design provisions

- The minimum thickness provisions of slabs depend on support conditions and span length. The Eurocode2 provisions are distinct as they include the reinforcement ratios for minimum thickness provisions along with support conditions and span length.
- In the comparison of one-way slab provisions, the ACI 318 show a general trend of decreasing span/depth ratio with increasing span length.
- The ACI 318 provisions are conservative for l/240 deflection limit while they are unconservative for l/480 deflection limits.

4.7.3 Flexural and shear design provisions in beams

- In computing the flexural design strength of beams, the EC2 provisions are less economical than those of ACI 318.
- The EC2 and BS 8110 provisions show close agreement in flexure and axial compression results.

- ACI provisions of flexure with axial compression are more conservative.
- The shear strength provisions are diverging distinctly for different codes attributing to the incompatibility between shear mechanisms and fundamental theories of concrete.

4.7.4 Column design provisions

- Column design provisions in all design codes follow similar philosophy in terms of slenderness ratio, short column condition and design ultimate axial load.
- The maximum and minimum longitudinal reinforcements provisions in columns are agreeing among different codes. This is exceptional in the case of ACI 318-14, in which the minimum and maximum reinforcement ratio is 1% and 8% respectively. These values are the highest when compared with other codes.

4.7.5 Sectional design under constant parameters

- For a given constant live load, dead load and wind load, when a section was designed as per different design codes, following observations were made for a given concrete section:
    - ACI 318 gives a greater ultimate moment of resistance as compared to BS 8110 and EC2. Hence, ACI requires smaller sections – more economic.

- The longitudinal reinforcement in columns was least when designed as per EC2
- For a given section dimension EC2 yields highest axial strength.
- Longitudinal and transverse reinforcement is least in slabs designed in accordance with EC2 provisions.

4.7.6 Crack-width provision evaluation of different codes

It can be inferred that of all the design code provisions, only the ACI provision stands out. The ACI adopts a maximum reinforcement spacing coefficient to determine crack width rather than a coefficient to determine the crack width itself of the section under consideration. Allam et al (Allam et al., 2012) also inferred the same and concluded that this deviation in the trend can be attributed to research carried out off late, which suggest that crack width was influenced by bar spacing more, than the reinforcement corrosion as it was believed initially. Studies also showed that,

- The Eurocode 2 provisions gave more realistic values.
- Values of crack width given by British standards were very similar attributing to the similarity in provisions and parameters used in the empirical formula.
- Until 1995, ACI 318 adopted a similar equation to calculate the design surface crack width just like other codes.

$$w_{cr} = [(11 \times 10^{-6})\, 3\sqrt{dc}\left(\frac{Ae}{n}\right)\beta]\ f_{st}$$

The above equation greatly overestimated the crack width values except at high steel stress and high values of reinforcement. The calculated crack widths according to the formulae did not correlate with the crack width in members tested under controlled laboratory conditions which, led the ACI to dispense the then crack width calculations and adopt a simplified approach to calculate maximum bar spacing (Subramanian, 2005).

Most codes enforce a maximum permissible crack width limit for structural members exposed to various exposure conditions. Most of the permissible max crack width limits lie under 0.4mm for most codes. The ACI 318 is exceptional as, it enforces a maximum permissible spacing for different types of reinforcement types in Table 24.3.2 of ACI 318-14. ACI 318-14 states that, for beams with grade 60 reinforcement and 2in clear cover to primary reinforcement with fs = 40,000psi maximum bar spacing is 10 in.

4.7.7 Deflection provision evaluation of different codes

Following inferences can be drawn from the parametric study of the empirical formulas.

- The maximum permissible deflection of members, in most codes, are specified as supporting/not supporting and attached/not attached to non-structural elements which are likely to be damaged by large deflections.
- The maximum deflection limits of most codes closely agree with others. Although they marginally vary among their provisions.

- Provisions of all most codes considered except ACI 318, show a general trend of decreasing span-depth ratio with an increase in span (Scanlon & Lee,2010).
- The American code gives less economical results for span lengths of up to 40ft while considering deflection limits of l/240 and l/480.
- In the case of flat slabs without drops, the ACI code values are more practical for the given span ranges, for deflection limits of l/240. However, it gave unconservative results for larger spans and deflection limits of l/480.
- The above table and the span-depth minimum thickness provisions considered in the previous sub chapters primarily consider reinforced concrete beams and one-way slabs. Two-way slab deflection control and limits are out of scope of this research as they are more complex to estimate. This is because there is no simplified procedure available in any codes or literature to estimate maximum deflections of two-way slabs.

4.7.8 Durability provision evaluation of different codes

The following are the inferences drawn from the comparative study of durability aspects in different codes. Our comparison is limited to only few durability aspects such as, exposure condition, minimum grade of concrete, maximum w/cm ratio and minimum concrete cover.

a) Exposure condition:

    - The ACI has only five categories and four classes for each category unlike the Eurocode which provides five classes and nine subclasses. Eurocode2 provides a more extensive classification.

- Also, codes like the ACI 318-14 define exposure condition as the environment surrounding the concrete while the BS 8110-97 classify exposure condition based on the environment to which the whole structure is exposed.

b) Minimum grade of concrete:

- The minimum grade of concrete required by BS 8110 for a given class of exposure is marginally higher than provisions of other codes.

c) Maximum w/cm ratio:

- The Eurocode2 values for sea-water exposure are high as compared to other codes.
- The w/cm ratio value of 0.4 in ACI 318 for marine environment is much less than that prescribed by other codes for the same class of exposure.
- The w/cm ratio of most codes range between values 0.4 to 0.6, except in Eurocode2 where in some class of exposure the w/cm values are as high as 0.7.

d) Concrete covers:

- Most codes specify a concrete cover required by individual structural members at the design stage.

However, some codes also specify the concrete cover values to be considered for the respective exposure condition.

4.8 Inferences from the parametric study – two best possible solutions for critical reviewing

It can be seen that each code stands out in different aspects. It was intended to make a general comparison of the design provisions in different codes and make the process of switching between codes easier for structural engineers. However, the customary units that each code follows, the cube or cylinder compressive strength of concrete, difference in steel grades etc. need to be given attention, while switching between codes, apart from the design methodologies and empirical relations.

In the UAE, the ACI 318-14 and the BS 8110 have been predominantly the major design codes used in building design and regional municipalities. Structures designed conforming to the two codes have been approved by the local authorities. Although the British standards are phased out in the UK and EC2 is now extensively used for design all over Europe. But this is in contrast to the scenario in UAE, where still a large number of designers and consultants depend and use the British Standards. Therefore, of all the three design codes compared here, the ACI 318-14, BS 8110-97 and Eurocode2, we limit the following section and its objectives to the two extensively used codes in the UAE, the ACI 318 and the BS 8110.

The following section will collate detailed study and comparison of the two codes and their relevance in the UAE.

CHAPTER 5

CRITICAL REVIEW OF ACI 318-14 AND BS 8110-97 CODE PROVISIONS

5.1 General

The innumerous literature available to us, explicitly discusses about the comparisons of structural specifications, provisions, design methodologies using ACI 318 and BS 8110. As discussed in the previous section, this section would focus primarily on two codes – the ACI 318-14 and BS 8110-97, the main features of, and the differences between them.

As mentioned earlier, the main motive of carrying out a detailed comparison of ACI 318 and BS 8110 is, to critically review the major standards that are widely used in the UAE. This will help us to obtain an insight into the extent to which these codes differ and agree with each other. This insight is not restricted to just engineers within the UAE but also to engineers in any part of the world, where a national design code does not exist and structural design using either ACI 318 or BS 8110 is approved by local authorities. In short, the aim of this critical review is to enable a structural engineer to switch between codes. Our aim is to find the cheapest solution conforming to appropriate safety, serviceability and aesthetic consideration.

5.2 Review on Parametric study of ACI 318 and BS 8110 code provisions

The parametric study carried out in this chapter explains the variation in design outputs owing to different codes due to the difference in parameters adopted by different codes in estimating the required strength, reinforcement etc. We will closely look at the main features in common and the differences between ACI 318 and BS 8110 codes. (Alnuaimi et al., 2013). Alnuaimi et al. conducted a comparative study of the amount of required reinforcement using the ACI and BS building codes and found that, the BS code requires less reinforcement than ACI for the same value of design loads (Alnuaimi et al., 2013). However, when the load safety factors were included to calculate the design loads, the ACI provisions require less reinforcement than the BS (Alnuaimi et al., 2013). . Tabsh (Tabsh, 2013) found that the BS load combinations gave a larger factored load than the ACI load combinations. The ACI 318 design strength is A_sf_y while the design strength according to BS 8110 is $0.95A_sf_y$. Tabsh (Tabsh, 2013) also found that the flexural capacity of singly reinforced sections was closely predicted by both codes. However, the ACI 318 provisions predicted a lower shear capacity than corresponding equations in BS code Tabsh (Tabsh, 2013). Alnuaimi et al. concluded in his paper that, the minimum flexural reinforcement required by ACI code is larger than BS code for RC rectangular beams (Alnuaimi et al., 2013). In contrast the minimum reinforcement required by ACI code is smaller than BS code requirements for RC rectangular beams (Alnuaimi et al., 2013).

Following sections will deal with a detailed comparison of flexural design of beams and slabs, shear design of beams and slabs, deflection provisions, and minimum/maximum reinforcement requirements in ACI 318-14 and BS 8110-97.

5.3 Design in flexure

5.3.1 ACI 318-14

The design of reinforced concrete elements includes the design of the section and the detailing. The ACI 318 has namely two approaches to design – the Allowable Stress Design method (Working Stress Design) and the Strength Design method (Ultimate Strength Design).

In ACI 318-14, members in flexure are designed on the basis of Strength Design Method. Whereby, since the 1970's the ACI code designs members for 'Strength' which otherwise means 'Ultimate'. This method is also termed as the Ultimate Design Method, which is based on principles of strain compatibility and static equilibrium along the depth of the section. The strength design approach considers the hypothetical cases of overloads in structures and the inelastic behavior of steel and concrete. Following are the assumptions of Strength Design Method (Saatcioglu, n.d.):

- Strain in reinforcement and concrete are directly proportional to the distance from neutral axis.
- The maximum compressive strain in the extreme compression fiber is 0.003, when the flexural member is said to have reached its flexural capacity.
- Stress in reinforcement varies linearly with the strain until up to specified yield strength after which the strains increase but stress remains constant.
- Tensile strength of concrete is neglected

- The stress distribution of concrete is represented by the corresponding stress-strain relation.

According to the provisions in ACI 318, a member in flexure could fail in either of the three modes of failure depending on the strain in the tension reinforcement, when the strain at the extreme compression fiber is 0.003 (Saatcioglu, n.d.):

- Tension controlled: A tension controlled section has strains in the extreme tension reinforcement $\geq$ 0.005. Safe designs of most sections are tension controlled as they display efficient ductile behavior which allows the redistribution of stresses and provides warning before failure.
- Compression controlled: The value of strain in the extreme tension reinforcement is equal to or less than the yield strain 0.002; where 0.002 is the yield strain of Grade 60 reinforcement. This type of section is least desirable as failure is sudden brittle fracture without warning.
- Transition region: Sections which lie between tension-controlled and compression-controlled.

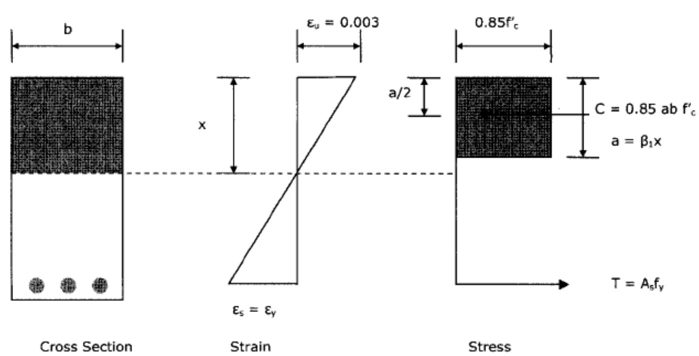


Figure 14: Concrete stress-strain block in ACI 318 (Hawileh et al., 2009)

The strength reduction factor φ , (as discussed in the previous section) for tension controlled section is 0.9. ACI 318 also requires that the φ factors be used with the corresponding load factors to achieve the compatibility between ACI 318 φ factors and ASCE 7-10 load combinations and factors. ACI 318 recommends the use of Design Aids – Flexure 1 to Flexure 9 for the design of reinforced concrete sections in flexure (Saatcioglu, n.d.).

5.3.2 BS 8110-97

The BS 8110 adopts the Ultimate Limit State approach of flexural design. The strain in extreme compression fiber is equal to 0.0035. This approach is based on following assumptions (docslide.us,2008):

- Plane sections remain plane for strain distribution in concrete and reinforcement.
- Stress Stain relation in concrete is given by the following stress block and strain diagram.
- Tensile strength of concrete is neglected
- When a reinforced concrete section is in only flexure, the lever arm should not be greater than 0.95 times the effective depth of the section. This applies to all beams and slabs.

The design strength in concrete is given by $\frac{0.67fcu}{\gamma m}$. Where $0.67f_{cu}$ is the maximum compressive stress in concrete at failure ($0.67f_{cu}$ is obtained by applying an additional safety factor to the value of maximum compressive stress in concrete – $0.8f_{cu}$)

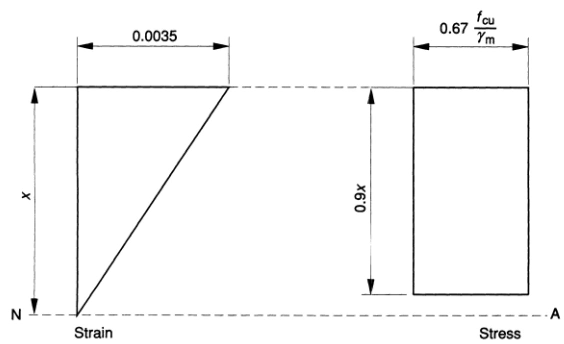


Figure 15: Concrete stress-strain block in BS 8110 (BS 8110-97)

Using the BS 8110, sections can be designed as under-reinforced, balanced and over-reinforced sections. Of these, under-reinforced sections are recommended to avoid a compression failure of concrete, which is sudden and brittle. Under-reinforced sections fail when the reinforcement has reached its yield strength and begins to deform while it provides ample warning against failure.

5.4 Design in shear

ACI 318 assumes that the shear strength of concrete is directly proportional to the square root of concrete cylinder compressive strength whereas in the BS code it is proportional to the cubic root of cube concrete compressive strength (Alnuaimi et al.,2013). As shear design philosophy varies among codes it is attributed as the main reason for inconsistencies in shear design. Following are the code provisions and empirical formulas for shear design in ACI 318-14 and BS 8110-97.

5.4.1 ACI 318-14

The shear provisions for ACI 318-95 were initially based on the shear friction concept – considering the shear at the interface of cracking but studies that followed provided evidence that shear friction method could not appreciably correlate with the observed shear failure (Hwang et al., 2000). As per the present code provisions, the nominal required shear strength is φ times the sum of concrete shear strength V_C and steel shear strength V_s , where φ is the strength reduction factor for concrete in shear and has a value of 0.75.

5.4.1.1 Concrete shear strength V_C *(Section 22.5.5.1 ACI 318-14)*(*ACI Committee, 2005)*

The shear strength in concrete is given by two sets of equation. One, is a simplified equation which is generally preferred for a practical approach and the other is a set of detailed equations for a longer but detailed process.

$$Vc = 2\sqrt{f'c}\,bd \quad \textit{Simplified equation}$$

$$Vc = \text{lesser of} \begin{cases} \left(1.9\sqrt{f'c} + 2500\rho w\,\frac{Vu\,d}{Mu}\right) bw\,d \\ \left(1.9\sqrt{f'c} + 2500\rho w\right) bw\,d \\ 3.5\,bw\,d\,\sqrt{f'c} \end{cases} \quad \textit{Detailed equations}$$

5.4.1.2 Steel shear strength Vs *(Section 22.5.10.5.3 ACI 318-14)(ACI Committee, 2005)*

The shear strength offered by the transverse reinforcement can be given using the following equation,

$$\boxed{Vs = \frac{Av\,fy\,d}{s}}$$

Where, fy = stress in reinforcement

d/s = number of stirrups given by diameter of bar over bar spacing

Finally,

Nominal shear strength of concrete section, $\boxed{Vn = Vc + Vs}$

And it is required that, $\boxed{\varphi Vn = \varphi\,(Vc + Vs) \geq Vu}$

Where, φ = strength reduction factor of concrete in shear = 0.75

Vn = Nominal shear strength of concrete section

Vc = Shear strength in concrete

V_s = Shear strength in steel

V_u = Design shear force on the concrete section

The design model that has been elaborated for comparison uses a practical design approach. The example solved in APPENDIX G1 & G2 elaborates the shear design to ACI 318.

Karim (Rebeiz, 1999) in his paper stated that the cracking shear strength provisions in ACI were relatively conservative when compared to the proposals made by him specially for high reinforcement ratios.

5.4.2 BS 8110-97

The British code follows an entirely different approach to shear design. This approach includes calculating the design shear stress and the concrete shear stress; comparing between the values and providing appropriate transverse reinforcement as per codal provisions.

5.4.2.1 Design shear stress υ

The design shear stress at any cross-section is calculated from,

$$\boxed{\upsilon = \frac{V}{b\,d}}$$

And in no case should υ exceed: $0.8\sqrt{fcu}$ or 5 N/mm2

5.4.2.2 Concrete shear stresse υ_c

Values for design concrete shear stress υ_c are given in Table 3.8 of BS 8110-97 (BS 8110-1:1997, 1999), and is a function of effective depth and $\frac{100As}{bd}$.

The values of υ is compared with υ_c to determine the form and area of reinforcement in beams and slabs. This is done using the guidelines given in Table 3.7 of BS 8110-97 (BS 8110-1:1997, 1999).

The same principle is followed for shear design of concrete slabs as well.

5.5 Deflection

BS 8110-97 and ACI 318-14 provide codal provisions to calculate the short-term deflection, deflection due to creep-shrinkage (long term deflection) and deflection effects for beams and one-way slabs. This is because one-way slabs behave pretty much similar to beams and are considered as wide beams. The deflection limits are in terms of span-depth ratios for beams and slabs in most codes and these deflection limits should satisfy deflection control in reinforced concrete sections, unless a detailed deflection is needed. The deflection limits for two-way slabs in most codes is determined using span-depth ratios and these provisions vary significantly from code to code as there is no detailed deflection control guidelines specified for two-way slabs in any code. Shrinkage deflection is a result of drying of concrete. The shrinkage deflection is calculated either using empirical formulas or the curvatures of the section under drying of concrete. While, creep deflection is a result of deformation due to sustained load and is calculated by considering empirical formulas that use creep-coefficient or the curvature of the section under sustained load.

The calculation of deflection of two-way slabs involves a complex analysis and hence is out of scope of this research. Deflection control is mainly considered at service load levels. Following is a review of deflection control provisions in ACI 318-14 and BS 8110-97.

5.5.1 ACI 318-14

ACI 318 recommends a two-tier approach to limit and control deflection in Chapter 24 of ACI 318-14. These two approaches are a direct approach using minimum thickness provisions and an indirect approach using empirical formulas of elastic deflection.

a. Minimum thickness provisions: In structural members such as beams, one-way slabs and two-way slabs, deflection is controlled by limiting the minimum overall thickness requirements of the concrete sections as prescribed in Sections 9.3.1, 7.3.1 and 8.3.1 of ACI 318-14 (ACI Committee, 2005) respectively. These requirements for concrete sections are for those members which do not support or which are not attached to non-structural elements which are likely to be damaged by large deflections. Satisfying these provisions of the code could control deflections in structural elements like beams and slabs to a great extent.
b. Empirical approach using methods of formulas for elastic deflection: This approach is applicable to members that do not meet minimum requirements stated above and for those members which support or are attached to non-structural elements which are likely to be damaged by large deflections as in Section 24.2.3 through 24.2.5 of ACI 318-14 (ACI Committee, 2005). All

calculated deflections should be limited to those in Table 24.2.2 of ACI 318-14 (ACI Committee, 2005).

The above deflection provisions in Section 24.2 are applicable to only those elements which are attached/not attached and support/do not support non-structural elements specifically. However, deflection control of members which could adversely affect any structural member attached to them is explicitly dealt in ACI 209 which does not cover the scope of this research work.

5.5.1.1 Minimum thickness provisions

Below are the minimum thickness provisions of span-depth ratio for one-way slabs and two-way slabs. Following span-depth ratio should be considered while assuming the minimum thickness of slabs before detailed design.

ACI 318-14	**Minimum thickness provisions for one way slabs**						
	Simply supported	One end continous		Both ends continous		Cantilever	where 'h' = overall slab thickness for normal weight concrete and fy = 60,000psi
	l/20	l/24		l/28		l/10	
	Minimum thickness provisions of two way slabs						
	Without drop panels			With drop panels			fy, psi
	Exterior panels		Interior panels	Exterior panels		Interior panels	
	Without edge beams	With edge beam		Without edge beams	With edge beams		
	$l_n/33$	$l_n/36$	$l_n/36$	$l_n/36$	$l_n/40$	$l_n/40$	40,000
	$l_n/30$	$l_n/33$	$l_n/33$	$l_n/33$	$l_n/36$	$l_n/36$	60,000
	$l_n/28$	$l_n/31$	$l_n/31$	$l_n/31$	$l_n/34$	$l_n/34$	75,000
	where l_n is the clear span in the long direction						

Table 21: Minimum thickness provision for slabs (ACI 318-14)

The minimum thickness provisions of slabs with and without drop panels are used for simplified calculation of minimum thickness of flat slabs.

Below are the minimum thickness provisions of span-depth ratio for non-prestressed beams. Following span-depth ratio should be considered while assuming the minimum thickness of slabs before detailed design.

Span/Depth ratio of Beams (Singly reinforced beams)			
Standard	Minimum depth of non-prestressed beams		
ACI 318-14	Support condition	Minimum h	* applicable for normal concrete weight and fy = 60,000 psi
	Simply supported	l/16	
	One end continuous	l/18.5	
	Both ends continuous	l/21	
	Cantilever	l/8	

Table 22: Minimum thickness provisions for beams (ACI 318-14)

5.5.1.2 Empirical approach using methods of formulas for elastic deflection

The empirical approach rather a detailed approach using elastic deflection, includes an elaborate calculation of Immediate deflection and time-dependent deflection

- Immediate Deflection Δi

  Immediate deflection of uncracked prismatic members is calculated using methods or formulas for elastic deflection.

$$\boxed{\Delta i \quad = \quad (kwl^4) / 384\, E_c\, I_g}$$

The above formula considers effects of cracking and reinforcement on the member stiffness. Note E_c I_g is a constant value.

The immediate deflection of a two way slab is calculated by plugging in E_c and I_e instead of the constant E_cI_g . This is because I_e is effective moment of inertia. Once concrete cracks, the concrete in tension zone does not efficiently contribute in resisting forces and moments. This requires an I value that considers effects of cracking also. ACI code adopts the effective moment of inertia where I_e ,

$$Ie = (\frac{Mcr}{Ma})^3\, Ig + (\,1 - \,(\frac{Mcr}{Ma})^3\,))\, I_{cr} \qquad < \qquad I_g$$

$where,\ Mcr = \frac{fr\,Ig}{yt}$ and, $f_r = 7.5\,\sqrt{f'c}$

Code provisions in Section 24.2.3.7 of ACI 318-14 (ACI Committee, 2005) also states that, Ie for prismatic one-way slab and beams shall be considered using the same equation for moment at the midspan in simple and continuous members and at the support, for cantilevers.

- Time-dependent deflection

Time dependent deflection is a result of sustained load with effects of creep and shrinkage of flexural members. The ACI code states that total deflection is calculated as a product of Immediate deflection caused by sustained load and factor λ_Δ , where λ_Δ is given by

$$\lambda_\Delta = \frac{\xi}{1+50\rho'}$$

where, ξ is the time dependent factor for sustained loads and ρ' is taken at the midspan for simple and continuous spans and at the support for cantilever.

Hence, the total deflection is given by the equation,

$$\Delta_{total} = \delta_{immediate} + \lambda\delta_{immediate}$$

Maximum permissible deflection is as per the values given in the table below,

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	**ℓ/180***
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	**ℓ/360**
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	**ℓ/480‡**
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		**ℓ/240§**

*Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.
†Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.
‡Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.
§Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

Figure 16: Maximum permissible deflection (ACI 318-14)

5.5.2 BS 8110-2-85

The deflection control in BS 8110 is given by equations for limiting the span-depth of the section. These values are dependent on the boundary conditions and span range. Following span-depth ratio should be considered while assuming the minimum thickness of slabs before detailed design.

	Basic span/effective depth ratio for rectangular beams		
BS 8110-97	Support condition	s/d ratio	* For two way slabs the ratio is based on shorter span ; appropriate modification factor applied for both tension and compression reinforcement (Cl 3.4.6.5 & Cl 3.4.6.6 BS 8110-97)
	Cantilever	7	
	Simply supported	20	
	Continuous	26	

	Minimum thickness of both one and two way slabs		
BS 8110-97	Support conditions	Rectangular section	* For two way slabs the ratio is based on shorter span ; appropriate modification factor applied for both tension and compression reinforcement (Cl 3.4.6.5 & Cl 3.4.6.6 BS 8110-97)
	Cantilever	7	
	Simply supported	20	
	Continuous	26	

Table 23: Minimum thickness provisions of beams and slabs (BS 8110-97)

For flat slabs, if the width of the drop is greater than one-third of the span of the slab, the basic span-depth ratio can be applied and otherwise a modification factor of 0.9 is used. In BS 8110, the detailed deflection analysis of a reinforced concrete element is considered as its curvature under sustained permanent loads/ live loads, creep and shrinkage. The sum of curvatures gives us the total deflection of the member. The

Code specifies different approach to calculate the curvatures of cracked and uncracked section. Section 3.6 of BS 8110-2-85 (BS 8110-2:1985, 1985) gives following set of assumptions for,

a. Cracked sections:
    1. Plane sections remain plane
    2. Reinforcement in tension or compression is assumed to be elastic and its modulus of elasticity = 200kN/mm^2
    3. Concrete in compression is also assumed to be elastic. Modulus of elasticity of concrete is obtained from Table 7.3 of BS 8110-2-85 (BS 8110-2:1985, 1985).
    4. Stress in concrete in tension is calculated by assuming the stress distribution to be triangular. Stress at the neutral axis is equal to zero and a value of 1N/mm^2 at the centroid of tension steel which reduces to 0.55N.mm^2.
b. Uncracked section:
    1. Concrete and steel are both assumed to be fully elastic in tension and in compression
    2. Modulus of elasticity of steel = 200kN/mm^2
    3. Modulus of elasticity of concrete is obtained from Table 7.3 of BS 8110-2-85 (BS 8110-2:1985, 1985).

The British code procedure of calculating the Immediate deflection of reinforced concrete sections in uncracked stage is entirely considering the curvature of the section at different conditions. The empirical formulas below are used to determine the curvatures.

5.5.2.1 For cracked sections

For cracked sections, the curvature $1/r_b$ is given by the equation

$$\boxed{\frac{1}{rb} = \frac{fc}{XEc} = \frac{fs}{(d-x)Es}}$$

where,

$\frac{1}{rb}$	=	curvature at midspan of simple & continuous members, and at the support for cantilevers
fc	=	design service stress in concrete
Ec	=	short term elastic modulus of concrete
fs	=	design service stress in tension
d	=	effective depth of section
x	=	depth of neutral axis
Es	=	modulus of elasticity of reinforcement

5.5.2.2 For uncracked section

For uncracked section, the curvature is given by,

$$\boxed{\frac{1}{rb} = \frac{M}{IEc}}$$

where,

In the above equations Ec – value of moment of elasticity is obtained depending on if loading is short-term or long-term loading. In the code, under section 3.6, under long term loading the effective modulus of elasticity is taken as, $\frac{1}{1+\varphi}$ times the short-term modulus of elasticity where, φ is the appropriate creep coefficient given in Section 7.3 of BS 8110-85 (BS 8110-2:1985, 1985) (Crcrecruits.files.wordpress.com, 2014).

5.5.2.3 Shrinkage curvature

Shrinkage curvature is calculated by considering the shrinkage strains. Shrinkage curvature is given by,

$$\boxed{\frac{1}{rcs} = \frac{\varepsilon cs * \ \alpha e * Ss}{I}}$$

where,

$\frac{1}{rcs}$	is the shrinkage calculator
αe	is the modular ratio = $\frac{Es}{Eeff}$
εcs	is the free shrinkage strain
E_{eff}	is the effective modulus of elasticity of the concrete which can be taken as Ec/(1 + φ)
Ec	is the short-term modulus of the concrete
Es	is the modulus of elasticity of reinforcement

Φ is the creep-coefficient

I is the second moment of area of either the cracked or the gross section

Ss is the first moment of area of the reinforcement about the centroid of the cracked section or the gross section

The creep curvature is calculated by considering creep coefficient.

Total long-term curvature of the section is calculated as follows:

$$\boxed{\text{Total long-term curvature} \quad = \quad (C) + \{(B) - (A)\} + (D)}$$

where,

(A) is the Instantaneous curvature under permanent load

(B) is the instantaneous curvature under total load

(C) is the long-term curvature under permanent load and,

(D) is the shrinkage curvature

This long-term curvature is further used to calculate the total deflection as,

$$\boxed{\text{Total deflection} \quad = \quad \text{Constant} * \text{Total curvature} * I^2}$$

The total deflection of a concrete section can also be derived as a function of the total curvature.

Deflection from curvature = $a = (K L^2) / r_b$

However, Santhi et al. (Santhi, Prasad & Ahuja, 2007) suggested that instantaneous deflection or short-term deflection is given by

$$\boxed{\delta = (wl^4) / (384\ E\ I)} \quad \text{for uncracked section}$$

It has been recommended by the British code, that deflection of slabs is best given by the respective span-depth ratio and it should satisfy the deflection control provisions.

However, if a further detailed study of deflection is required, the code suggests considering a strip of the slab spanning across the shorter edge of the slab connecting centers of longer side (Crcrecruits.files.wordpress.com, 2014). The bending moments of the strip is obtained using elastic analysis and deflection of this strip is then calculated as though it were a beam (Crcrecruits.files.wordpress.com, 2014).

5.5.3 Maximum permissible deflections

The following ratios give the excessive deflection limits due to vertical loads. A sag in a member, produces a noticeable deflection if deflection exceeds l/250, where l is span of simple or continuous member or length of cantilever (Crcrecruits.files.wordpress.com, 2014).

For members which support or are attached to non-structural elements likely to be damaged by large deflections:

- For brittle material min of $\begin{cases} \frac{L}{500} \ or \\ 20mm \end{cases}$

- For non-brittle partitions and finishes min of $\begin{cases} \frac{L}{350} & or \\ 20mm \end{cases}$

In simple language the table below gives the maximum permissible deflection limits comparison of ACI and BS codes.

Condition	Deflection limits	
	ACI	BS
Members supporting non-structural elements that are not likely to be damaged by large deflections	span/240	span/250
Members supporting non-structural elements likely to be damaged by large deflections	span/480	span/500 < 20mm

Table 24: Comparison of maximum permissible deflection limits in ACI and BS

5.6 Minimum and Maximum reinforcement area

Since the comparative study is limited to flexural design and shear design. The discussions have been derived from the observations made from the code provisions for minimum maximum reinforcements in both codes together with the elaborated design examples solved in the APPENDIX FI, F2, G1, G2, H1 and H2.

5.7 Discussions of the critical review of ACI 318-14 and BS 8110-97

This sub-chapter elaborates on the discussions drawn from the critical review of two major codes of relevance in the UAE, ACI 318-14 and BS 8110-97. These discussions focus primarily on the review of flexural design provisions, shear design provisions, deflection provisions and percentages of minimum maximum reinforcements prescribed under both codes.

5.7.1 Flexural design provisions

- The ACI code provisions are based on concrete cylinder strengths, f'_c, and BS 8110 provisions are based on concrete cube strength f_{cu}.
- The assumptions of both codes closely agree with each other.
- The ACI 318 and BS 8110 both are based on the simplified rectangular stress block (Alnuaimi et al., 2013). The ACI 318 assumes a maximum compressive strain of concrete equal to 0.003, while, the BS 8110 assumes a maximum compressive strain value equal to 0.0035.
- The simplified rectangular stress block gives a maximum compressive stress value of $0.85f'_c$ in ACI 318, and $\frac{0.67fcu}{1.5}$ in BS 8110.
- The strength reduction factor for ACI 318, for a section in flexure is given by $\varphi = 0.9$ (tension-controlled section). While the BS 8110 uses a partial material safety factor of 1.5 for concrete and 1.15 for steel, applied as divisors to the concrete cube strength f_{cu} and the yield strength of steel f_y.
- Area of required tension reinforcement in ACI 318-14 is,

$$\text{As} = \frac{Mu}{\varphi fy\,(d - \frac{a}{2})}$$

where, a = $d - \sqrt{(\,d^2 - \frac{2Mu}{0.85f'c\varphi b})}$

While, the area of required tension reinforcement in BS 8110-97 is,

$$\text{As} = \frac{Mu}{0.87fy\,z}$$

where, z = d (0.5 + $\sqrt{(0.25 - \frac{K}{0.9})}$ $\leq 0.95d$ and K = $\sqrt{\frac{Mu}{fcubd2}}$

- The minimum area of longitudinal reinforcement, A_{smin} in ACI 318 considers both material and geometry whereas, BS 8110 is based only on geometry (Alnuaimi et al., 2013).

To compare the similarities and differences in flexural design of reinforced concrete section, a reinforced concrete beam of span 30', width of 15" and total depth of 30" and a beam of span 9m, width of 375mm and total depth of 750 mm; was designed as a singly reinforced beam, using both ACI 318 and BS 8110 provisions respectively. Each of the beams were designed separately in terms of both U.S customary units and Metric unit systems respectively. The same dead and live loads were used in the design using both codes, including the compressive strength of concrete and yield strength of steel reinforcement according to their values in the respective codes.

The section has been modelled for practical design adopting simple singly reinforced beam. It has been tried to the level best to maintain consistency in the dimensions of reinforced concrete sections, compressive strength of concrete, yield strength of steel and loadings. The design is elaborated for reference in APPENDIX FI & F2. The following are the conclusions drawn from the comparative study:

a. The procedure for flexural design of members closely agree between ACI and BS codes.

b. For a section of given dimensions, when designed using both ACI 318 and BS 8110, it was noticed that when designed using BS 8110-97 provisions the size of the beam was larger than required for flexural strength and required only minimum amount of tension steel to resist the moments and loads.

c. Both codes closely predict the flexural capacity of under reinforced sections.

d. Both codes advocate the design of under reinforced sections or tension - controlled sections attributing to the ample warning given by the section before failure unlike an over reinforced section which displays a sudden, brittle failure.

e. The 'required reinforcement' by ACI is much greater than that required by BS code (Alnuaimi et al.,2013).

f. The minimum area of flexural reinforcement required by ACI code is larger than that required by BS code (Alnuaimi et al.,2013). A comparison was made for minimum area of flexural reinforcement using ACI 318:08 and BS 8110:97 by considering a beam of cross sectional dimension 350 X 700 mm with effective depth of 625 mm, considering an yield strength of reinforcement as

460MPa. It was seen that the minimum area flexural reinforcement required by ACI was much larger than that required by BS (Alnuaimi et al.,2013).

g. The following figure adapted from (Alnuaimi et al., 2013) shows that minimum area of flexural reinforcement according to BS provisions varies linearly with compressive strength of concrete unlike ACI provisions.

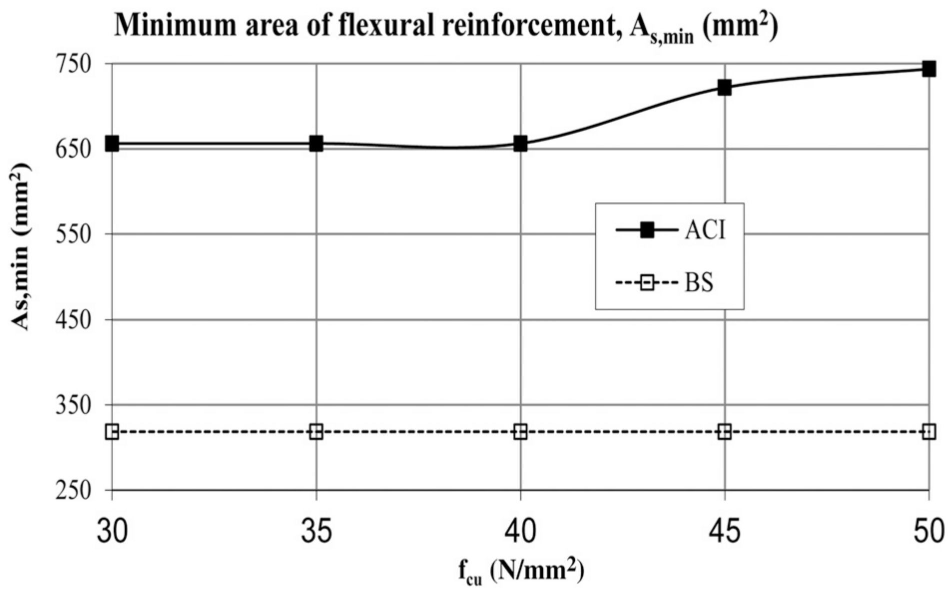


Figure 17: Plot of minimum area of flexural reinforcement with different f_{cu} (Alnuaimi et al.,2013)

h. The BS code value for maximum area of flexural reinforcement is 4%, which is very high. Although the ACI code 2002 version according to Subramanian (Subramanian, 2010), specified maximum percentage of flexural steel as 75% of balanced reinforcement ratio. However, this was changed considering complications in design of flanged sections. In the current version of ACI, ACI 318 – 14, ductility is controlled by the tensile strain in steel.

The flexural design of beams and its inferences were elaborated above. The flexural design of slabs using ACI and BS codes follow similar trends. The section has been modelled for practical design adopting simple one-way slab design. It has been tried to the level best to maintain consistency in the dimensions of reinforced concrete sections, compressive strength of concrete, yield strength of steel and loadings. The design is elaborated for reference in APPENDIX H1 & H2.

5.7.2 Shear design provisions

To compare the similarities and differences in shear design of reinforced concrete section, a reinforced concrete beam of span 15', width of 12" and total depth of 18" and a beam of span 4.5m, width of 300mm and total depth of 450mm; a singly reinforced beam was designed to resist shear, using both ACI 318 and BS 8110 provisions respectively. Each of the beams were designed separately in terms of both U.S customary units and Metric unit systems respectively. The same dead and live loads were used in the design using both codes, including the compressive strength of concrete and yield strength of steel reinforcement according to their values in the respective codes.

The section has been modelled for practical design using ACI codal provisions and an usual approach for BS 8110. It has been tried to the level best to maintain consistency in the dimensions of reinforced concrete sections, compressive strength of concrete, yield strength of steel and loadings. The design is elaborated for reference in APPENDIX G1 & G2. Owing to the large disparity in empirical equations used in BS & ACI, it was inferred that,

a. ‘Required shear reinforcement’ was more when designed using ACI provisions than BS provisions.

b. The maximum allowable spacing in ACI was much lesser than that suggested by BS 8110. BS codes offer a constant maximum permissible spacing of 0.75 times effective depth.

c. The minimum required shear reinforcement of BS code provisions is higher than the minimum shear reinforcement required by ACI provisions. Alnuaimi et al. performed an experiment on a beam of cross section 350 x 700 mm with an effective depth of 625mm and yield strength of steel as 460N/mm^2 (Alnuaimi et al.,2013). They reached a conclusion that, the minimum area of shear reinforcement required by BS code is larger than that required by ACI (Alnuaimi et al.,2013).

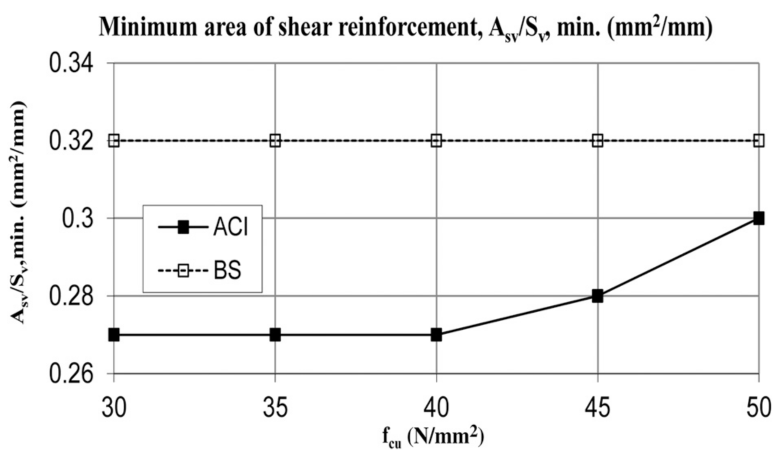


Figure 18: Plot of minimum area of shear reinforcement with different f_{cu} (Alnuaimi et al.,2013)

d. It was also found that the beam length that needs shear reinforcement required by BS code is shorter than that required for ACI code (Alnuaimi et al.,2013).

The example solved in APPENDIX G1 & G2, in the ACI design method of practical design the transverse reinforcement is designed for the entire length of a beam unlike, the British code approach in which the transverse reinforcements are closely spaced towards the supports and widely spaced at the midspans.

e. In the practical design approach, the design shear strength is considered at the critical section unlike the BS code which uses the design shear stress calculated at the supports. The critical section in ACI is at a distance 'd' from the face of the support; for which transverse reinforcement is designed.

f. The safety factors have a great impact on the required transverse reinforcement for different ultimate design loads. As a result, it can be noted that the resultant design shear force by British codes equations is greater than ACI.

g. Required shear reinforcement is greater in ACI than BS.

h. Minimum shear reinforcement is greater in BS than ACI.

5.7.3 Deflection provisions

a. The ACI provisions use an I_{eff} , effective moment of inertia to consider cracking effects of concrete while the BS code uses an effective modulus of elasticity.

b. The approach for calculating long-term deflection in both codes differ widely; as BS 8110 calculates effects of creep and shrinkage separately unlike ACI which includes creep and shrinkage effects in long term deflection.

c. The maximum permissible deflection limits of both codes are relatively similar.
d. (Alnuaimi et al., 2013) conducted a similar comparative review of codal provisions in ACI and BS. This was done by considering a standard beam of dimension 350 x 750mm and an effective depth of 625mm with concrete compressive strength of 30MPa; cylinder compressive strength of 24MPa and a steel yield strength of 460MPa. They arrived at a conclusion that short-term deflection in both codes decreased with an increase in deal live to live load ratios while contrarily, the long-term deflection increased with an increase in dead load and live load ratio.
e. The maximum permissible deflection limits are higher for ACI than BS.
f. The short-term, long-term and total deflection predicted using ACI code provisions give a higher value than BS predictions.
g. The values predicted by BS codes are within allowable limits.

For a given reinforced concrete section, for a given span considering U.S Customary Units and Metric system, the maximum permissible deflection limits recommended by both codes were calculated for comparative study. For this comparative study, a concrete section of span 4.5m and 15’ was analyzed using British and American standards respectively. This comparative study did not require consistency between different parameters but the span alone. Hence, keeping the span constant, the maximum permissible deflection control limits were applied to the spans with respect to the code provisions. The elaboration has been shown in APPENDIX E. Following are the inferences drawn from the analysis.

h. It was noted that, ACI provisions gave a greater deflection limit than BS standards for the same span of given section.
i. It was inferred that, for a given span, ACI is more liberal in terms of maximum permissible deflection as it allows for a greater deflection limit than BS.
j. On the contrary, BS provisions are seen to be more stringent by restricting their maximum permissible deflection limit to a lesser value.
k. In short, it was concluded that BS 8110 provisions have an upper hand over ACI provisions as it restricts deflection limit to a much lesser value, thus, providing an extra margin of safety when the same span gave a greater deflection limit when analyzed with ACI 318.

5.7.4 Minimum and maximum reinforcement area provisions

a. Minimum area of flexural reinforcement in beams, required by ACI 318-14 provisions is greater than that required by BS 8110-97 provisions (Alnuaimi et al., 2013).
b. Maximum area of tensile reinforcement in beams, required by BS code is comparatively a large value but is inappropriate to compare with ACI 318 provisions as, ACI 318 limits its maximum longitudinal reinforcement with respect to the tensile strain in steel.
c. Minimum area of reinforcement requirement in slabs, is greater according to BS provisions than the requirements of ACI code.

d. Maximum area of tensile reinforcement in slabs, is more in slabs designed according to BS code unlike ACI code. The ACI code limits the maximum area of reinforcement more stringently to ensure a tension-controlled section.
e. Minimum required shear reinforcement, according to BS code is greater than the ACI code requirement.
f. The required shear reinforcement for a given section is greater in ACI code than BS code (Alnuaimi et al., 2013).

5.8 Recommendations from critical review of ACI 318-14 and BS 8110-97

The following recommendations could be drawn by critically reviewing the two codes of relevance in the UAE, ACI and BS.

- The flexural design philosophies of American and British standards closely agree with each other. However, the reinforced concrete sections designed conforming to British standards required smaller sections for the same loading conditions when compared with a section designed to American standards. This concluded the fact that sections designed to ACI code provisions were more conservative.
- The shear design philosophies of the American and British standards distinctly vary. This can be viewed in the size of sections designed using either codes.
- With respect to the minimum and maximum reinforcements required for flexural and shear design. Minimum flexural reinforcement required by ACI is much greater than that required by BS. Required longitudinal reinforcement is greater in ACI than that by BS. However, this is contradictory in the case of transverse reinforcement. Required shear reinforcement is greater in ACI than in BS. Minimum shear reinforcement is greater in BS design than in ACI design.
- Deflection provision of ACI are slightly conservative when compared with BS standards.
- It is recommended from the above observations that, design conforming to BS standards could be far more economical in terms of section sizes when

compared with ACI standards. But this may not be in the case of reinforcements, as they vary for longitudinal and transverse reinforcements.

The objective of the upcoming chapter is to validate the above results with the help of the outputs from a practical model of a multistorey building, modelled, analyzed and designed using a commercial software.

CHAPTER 6

DESIGN AND ANALYSIS OF G+40 STOREY BUILDING TO ACI 318-14 & BS 8110-97 USING COMMERCIAL SOFTWARE

This chapter comprises of detail design and analysis of a multi storey building. The proposed building chosen for design and analysis is a 165 m tall G+40 storey building which will be designed conforming to both ACI 318-14 & BS 8110-97 codal provisions. The primary of objective of this chapter is to provide enough evidence through software results to support the finding and comparison made using available literature and hand calculation given in the previous chapters. The previous chapters compared fundamental theories and empirical formulas as per standards. The results from above discussions where cross checked with hand calculations. Hence, with this chapter, the three-tier comparative review is summed up by trying to compare software results.

6.1 Model

The proposed building is a G+40 storey star octagram shaped mixed used building to be designed for construction within the RAK emirate. This mixed used building accommodates commercial, office and residential spaces.

Level 1 to Level 8 are commercial spaces for shops, retails, malls etc. Level 9 to Level 21 accommodate office spaces with two offices on each floor. Level 22 to Level 41 are housing apartments with four apartments on each level. Every Level from Levels 9 to 41 have four balconies each which are proposed as open spaces. The building has a combination of four cores at each center of the building which accommodate lift shafts and stair rooms. The cores run throughout the height of the building. The proposed building is a typical building with two stages of typical floors.

A mix used building was chosen to accommodate different occupancies so that the design could reflect the variations that arise due to differences in live load considered for the respective type of occupancy. The three major categories of occupancies compared here are commercial, office and residential. The building follows a star octagram shape which makes it vertically irregular with large partial cantilever balconies supported by just a single column aligned at 45 degrees.

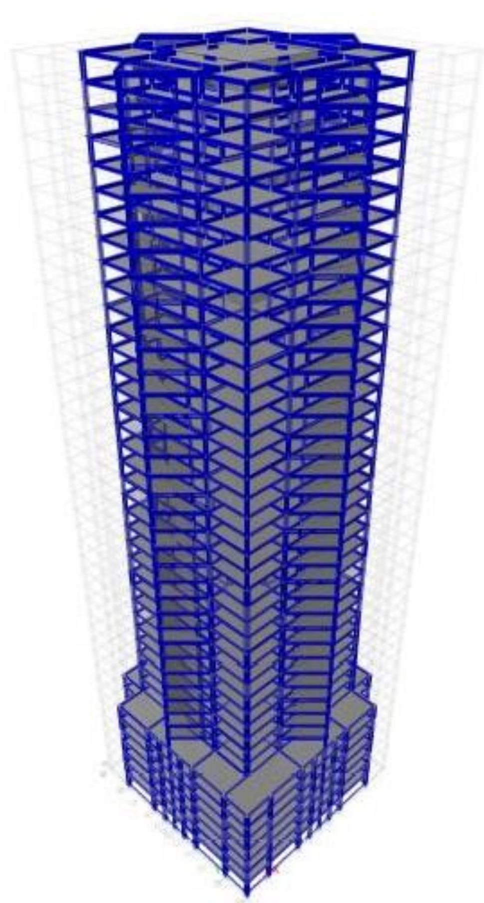


Figure 19: 3D of G+40-star octagram shaped building

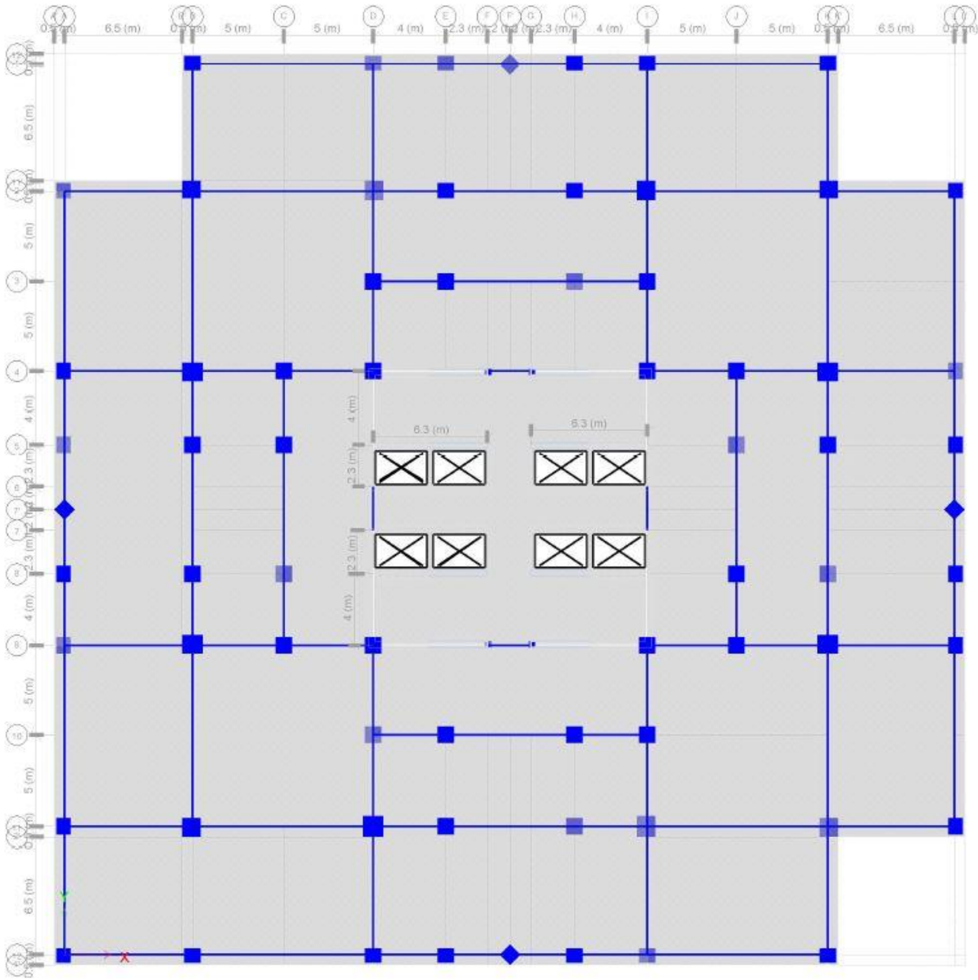


Figure 20: Commercial typical floor plan

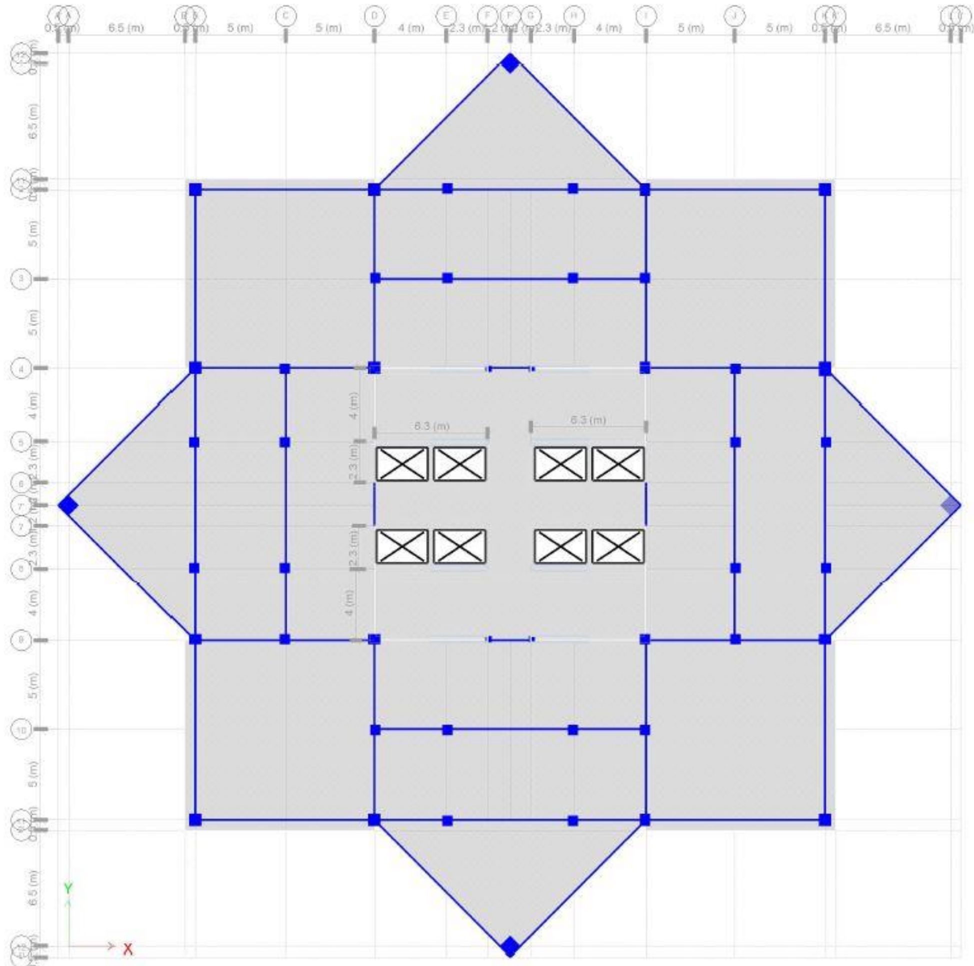


Figure 21: Typical floor plan of office and residential spaces

6.2 Design

The proposed tower was modelled in ETABS. Two separate models were designed – one as per the ACI code and the other as per BS standards. The typical floor plan of the commercial space has a dimension of 49 m x 49 m with 12 grids along both X and Y directions. This floor plan is closely similar to a square except for the re-entrant corners.

The building then follows a star octagram shape from levels 9 up to the roof. This typical floor plan with a dimension of 35 m x 35 m.

Along the plinth levels that accommodates commercial spaces, between level 1 to level 8, along X direction are grid lines A to L and along Y direction are gridlines 1 to 12, 12 grids along each direction

Sections for Preliminary Analysis And Design

- **Frame Sections**

  To begin with design, both models were assigned with initial estimated frame and slab sections. The following details give an insight on the sections used for preliminary design.

  - **Columns**

    The 40 storey building was divided into sections of 10 floors each to assign columns. The peripheral columns were assigned with larger sections as compared to internal columns.

Level 2-11	900 x 900	750 x 750
Level 12-21	900 x 900	750 x 750
Level 22-31	650 x 650	550 x 550
Level 32-41	650 x 650	550 x 550
Balcony Column	800 x 800	

- **Beams**

  It is proposed that the slab system adopted is a flat slab system. Hence, beams at peripheries of each slab is 200 x 700.

  Lift beams connecting cores are 300 x 500 and all tie beams are 200 x 600 and balcony beam is 300 x 800.

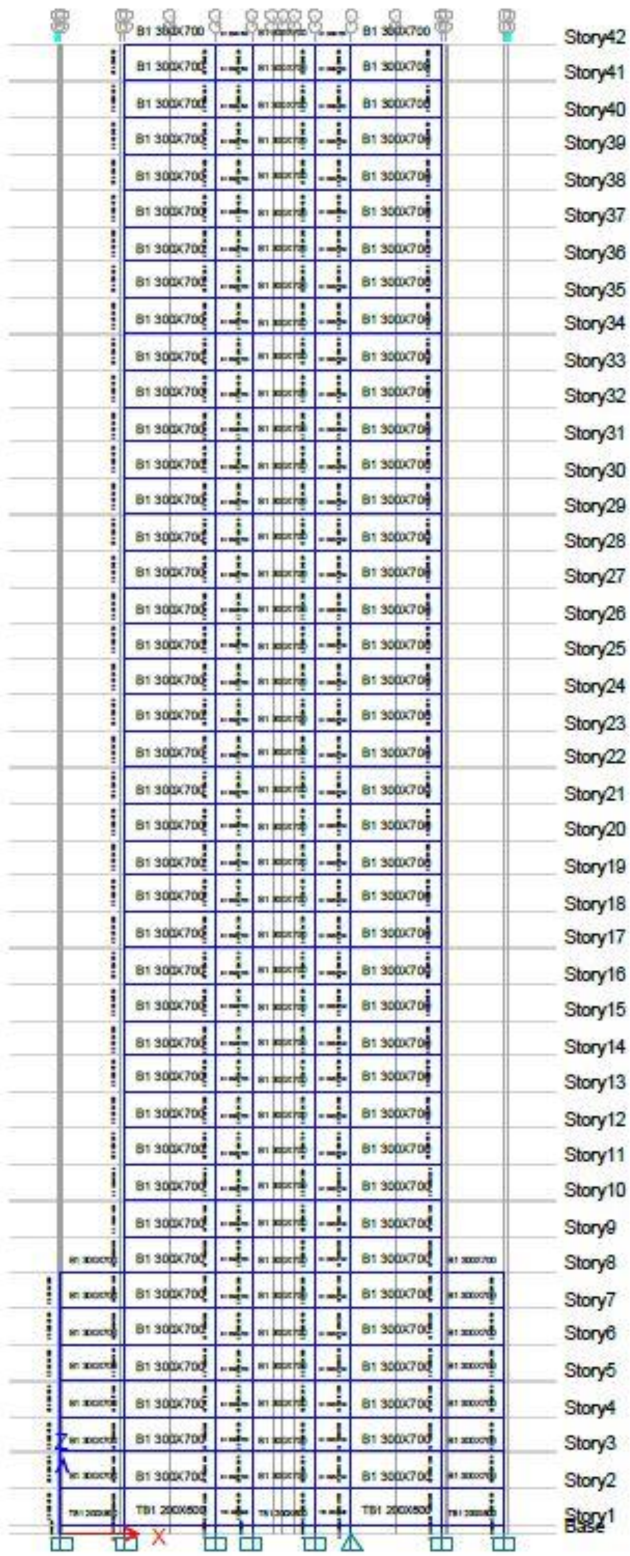


Figure 22: Beam and Column section elevation (Illustrative)

- **Floor slabs**

  The floor slabs are 200 mm thick while the partial balcony slab is 150 mm thick.

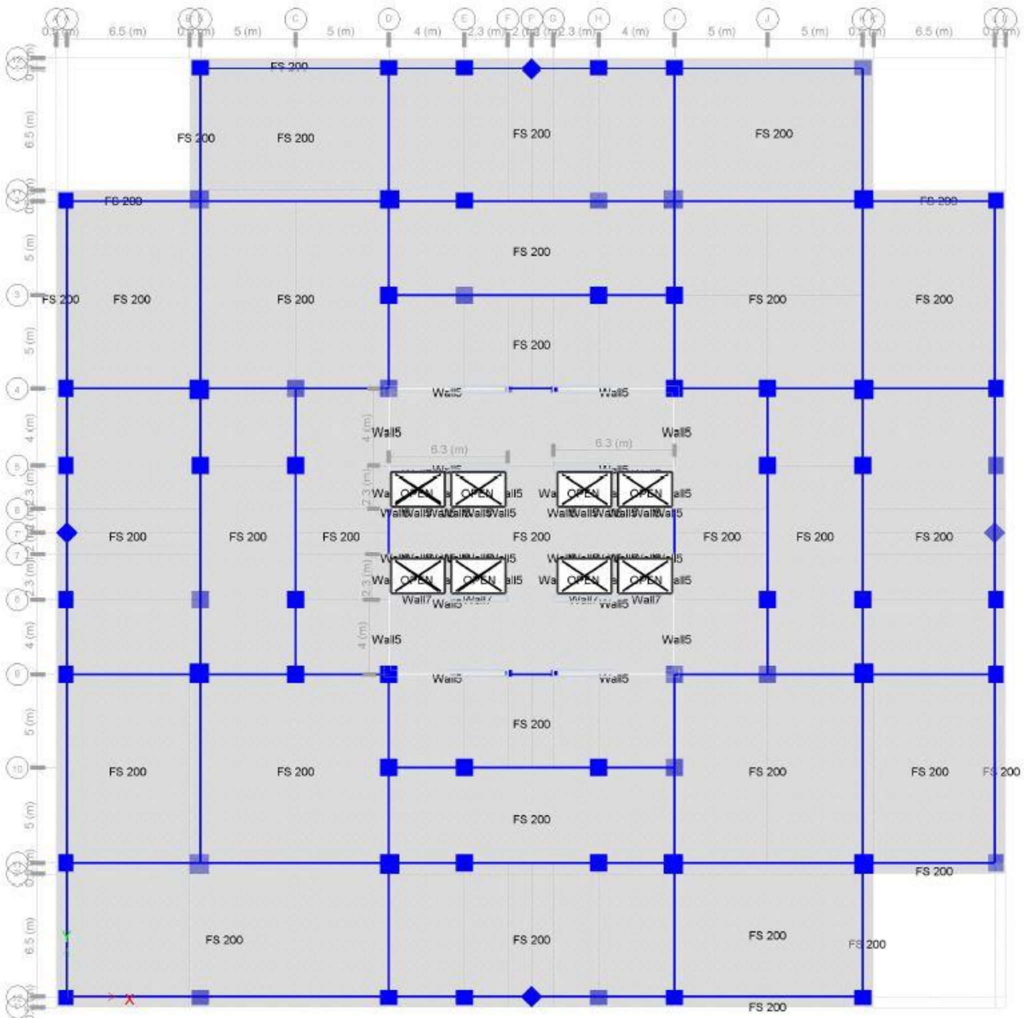


Figure 23: Slab sections for commercial floor (Illustrative)

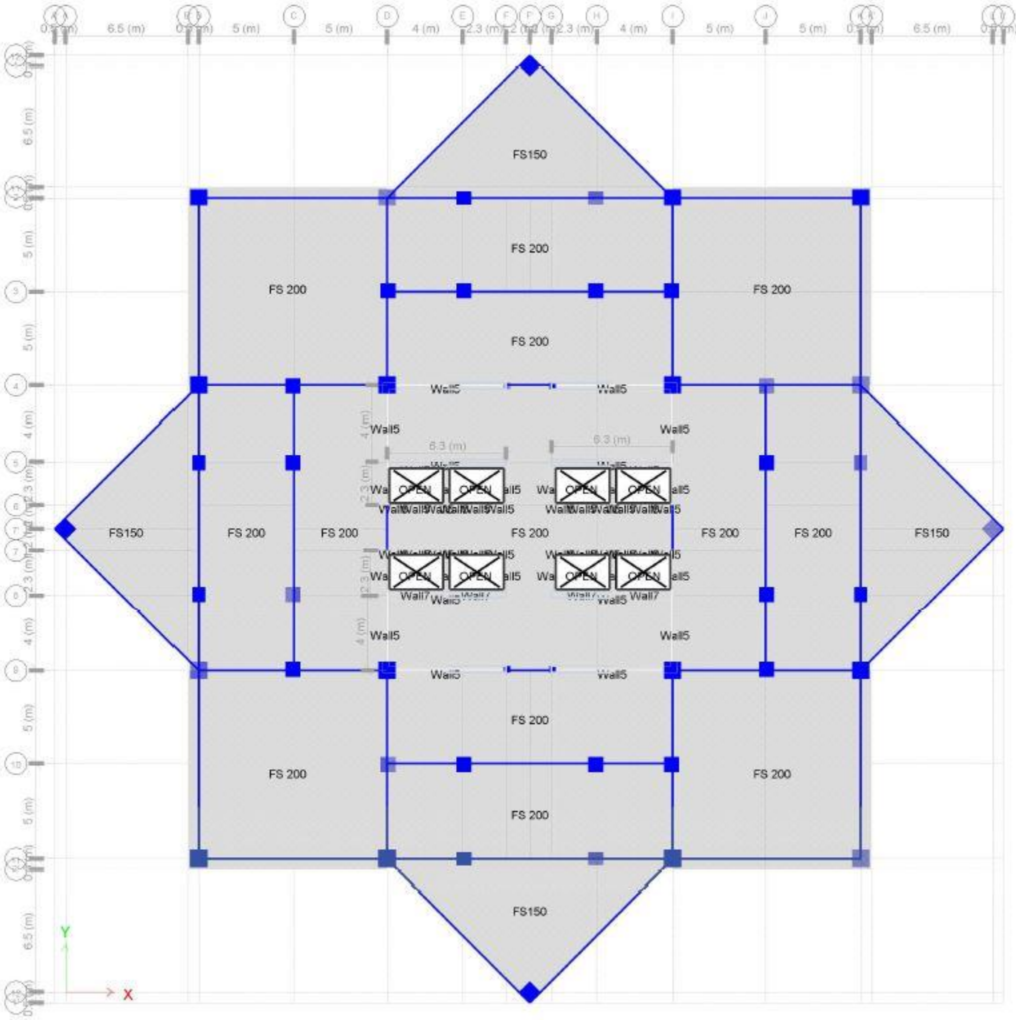


Figure 24: Slab sections for office and residential typical floors (Illustrative)

- **Load Patterns**

  The loadings are defined as three load patterns, Dead Load (D.L), Super Dead Load (S.D.L), Live Load (L.L) and Wind Load (W.L).

  - **Dead Load (DL)**

    The dead load is automatically assigned by the software itself provided, the self-weight multiplier for dead load pattern is given a value of 1. By doing so, the software assigns the dead weight of each concrete frame element with respect to the concrete design code chosen under 'design preferences' for analysis and design.

  - **Super Dead Load (SDL)**

    The S.D.L is assigned with a standard value assigned to a building as per RAK municipality recommendations. This value is 5.75 kN/m^2 which includes finishes, masonry and ceiling loads. This value theoretically sums up to only 4.5 or 5kN/m^2 (considering a floor finish load of 2kN/m^2, ceiling finish load of 0.5kN/m^2 and wall load of 3.6kN/m^2). However, considering maximum safety we assume a maximum possible value of 5.5kN/m^2 for design and analysis of both models. While this value is halved for balconies where the wall and finishes loads are comparatively lower than a normal floor slab. This value is 2.5kN/m^2 for cantilever slabs.

- **Live Loads (LL)**

  Live Loads as per code standards for different occupancies are given below.

Levels	Type of occupancy	ASCE 7-10	BS 6399-96
Level 1 - Level 8	**Commercial**		
	Commercial floor	6.00 kN/m2	4.00 kN/m2
	Commercial corridor	6.00 kN/m2	4.00 kN/m2
	Commercial stair	4.79 kN/m2	3.00 kN/m2
Level 9 - Level 21	**Office**		
	Office floor	2.4 kN/m2	2.5 kN/m2
	Office corridor	3.83 kN/m2	3.00 kN/m2
	Office stair	4.79 kN/m2	3.00 kN/m2
	Office balcony	3.6 kN/m2	4.00 kN/m2
Level 22 - Level 41	**Residential**		
	Residential floor	1.92 kN/m2	1.5 kN/m2
	Residential corridor	4.79 kN/m2	3.00 kN/m2
	Residential stairs	4.79 kN/m2	3.00 kN/m2
	Residential balcony	2.88 kN/m2	3.00 kN/m2
Level 42	**Roof**		
	Roof	1.92 kN/m2	1.5 kN/m2

Table 25: Live loads for different occupancies according to ASCE 7-10 and BS 6399-96

The live load values specified in the table above are values which have been taken from ASCE 7-10 (ASCE, 2010) and BS 6399-96 (BSI, 1996), code of practice for dead and imposed loads. The above loads were applied to the models according to their occupancy, while assigning shell load values for shell areas under the load pattern live load.

It can be inferred that the load values as per ASCE 7-10 is relatively greater than their corresponding values in BS 6399. This difference in load values is maximum for commercial spaces where the difference is as much as $2kN/m^2$.

However, if this variation will have a disparity in the design results has to be dealt with in further sections.

- **Wind Load**

  As mentioned in the previous sections, wind load application on the crosswind and along-wind directions requires pre-requisites that need to be evaluated and determined from the code provisions and tables for wind loads as per ASCE 7-10 and BS 6399-96. Some of these input parameters are, basic wind speed, its profile nature, terrain condition, intensity of gust factor etc. Considering this complexity, the analysis of the structure was done using program calculated parameters while modifying the lateral loads under defining load patterns. Hence, the wind loads were defined as Ex and Ey for wind loads acting along X and Y directions of the building respectively.

- **Load Combinations**

The following table gives an insight into the load combinations applied for design using ACI 318-14 and BS 8110-97. The load combinations for ACI 318 and BS 8110 are given below. The concrete frame design, slab design, shear wall design of two models are designed and analyzed with respect to the design load and combinations in ACI 318-14 and BS 8110 of which the results are interpreted and discussed in the following sub sections.

ASCE 7-10	BS 6399-96
1.4 DL + 1.4 SDL	1.4 DL + 1.4 SDL
1.2 DL + 1.2 SDL + 1.6 LL	1.4 DL + 1.4 SDL + 1.6 LL
1.2 DL + 1.2 SDL + 1.0 LL + 1.0 WX	1.2 DL + 1.2 SDL + 1.2 LL + 1.2 WX
1.2 DL + 1.2 SDL + 1.0 LL - 1.0 WX	1.2 DL + 1.2 SDL + 1.2 LL - 1.2 WX
1.2 DL + 1.2 SDL + 1.0 LL + 1.0 WY	1.2 DL + 1.2 SDL + 1.2 LL + 1.2 WY
1.2 DL + 1.2 SDL + 1.0 LL - 1.0 WY	1.2 DL + 1.2 SDL + 1.2 LL - 1.2 WY
0.9 DL + 0.9 SDL + 1.0 WX	1.4 DL + 1.4 SDL + 1.4WX
0.9 DL + 0.9 SDL - 1.0 WX	1.4 DL + 1.4 SDL - 1.4WX
0.9 DL + 0.9 SDL + 1.0 WY	1.4 DL + 1.4 SDL + 1.4WY
0.9 DL + 0.9 SDL - 1.0 WY	1.4 DL + 1.4 SDL - 1.4WY
	1.0 DL + 1.0 SDL + 1.4WX
	1.0 DL + 1.0 SDL - 1.4WX
	1.0 DL + 1.0 SDL + 1.4WY
	1.0 DL + 1.0 SDL - 1.4WY

Table 26: Load combinations used for analysis and design

The load combinations given above have been used for concrete frame design. It can be seen that the BS 6399-96 has greater number of load combinations than ASCE 7-10.

- **Sections That Passed Final Design Check**

The initial estimate of member sizes did not pass the design check. After a number of iterations, the following member sections passed the design checks.

ASCE 7-10	BS 6399-96
C1 1150 X 1150	C1 1050 X 1050
C2 1100 X 1100	C2 1000 X 1000
C3 1050 X 1050	C3 900 X 900
C4 1000 X 1000	C4 750 X 750
C5 900 X 900	C5 700 X 700
C6 750 X 750	C6 650 X 650
C7 650 X 650	C7 550 X 550
C8 550 X 550	C8 300 X 300
C9 300 X 300	CC 800 X 800
CC 800 X 800	B1 300 X 700
B1 350 X 800	LB 350 X 600
LB 350 X 825	CB 350 X 900
CB 350 X 900	TB1 200 X 600
TB1 200 X 600	

Table 27: Sections that passed the concrete design check

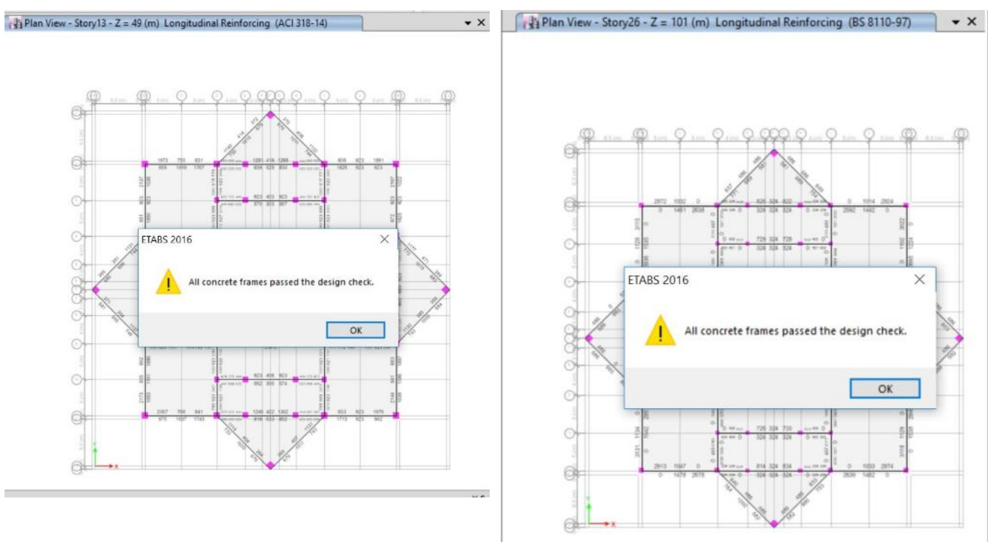


Figure 25: Pop up box which shows that members passed the concrete design check

It can be inferred that, the model designed to BS 8110-97 passed the design check with smaller sections than the ACI model. This can be attributed to the difference in load values for live loads, where, ASCE values were higher than BS values; that have been noticed in the previous subsections. This implies that the section sizes of frames are dependent and have a direct relation with the loads applied on them.

6.3 Discussion of Analysis and Design Results

The following sections critically review the analysis results and design outputs for building frame elements using either codes. Our scope of comparison extends to the flexural design results for beams, shear design results for beams, axial compression results of columns, the required area and minimum area of rebars suggested for concrete members, deflection, slab stresses and detailing.

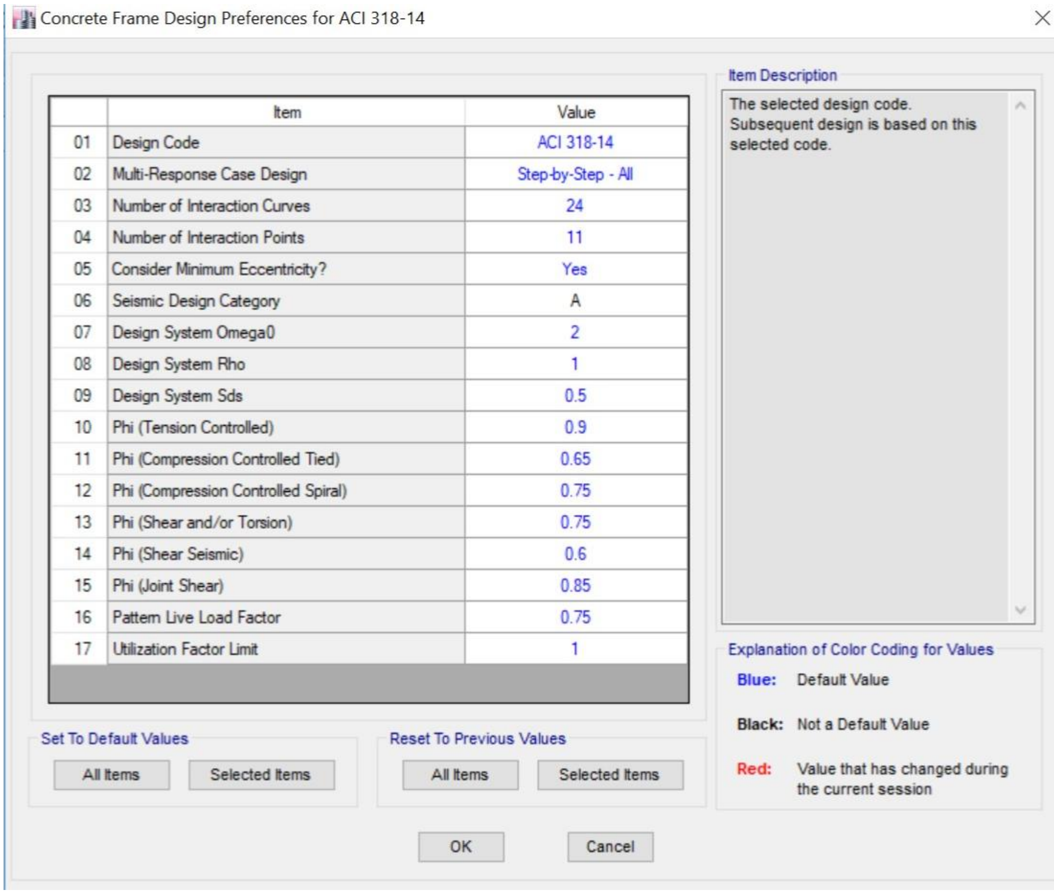
An attempt is made to compare similar concrete members of both codes and discuss the flexural, shear results and review the reason for this disparity.

Also, an attempt is made to compare the size of frame sections under each occupancy and discuss as to why the disparity in size of frame sections occur for the same occupancy when designed using both codes. It is also attempted to review if the load variation had a direct relation with the size of frame section.

6.3.1 Comparison of frame sections

Although the initial estimate of sections was estimated for design and analysis, repeated iterations were done until all sections passed the concrete frame design. Note that, here after the model designed to ACI 318-14 will be termed as 'ACI Model' and the model designed to BS 8110-97 will be termed as 'BS Model'.

6.3.1.1 ACI Model



	Item	Value
01	Design Code	ACI 318-14
02	Multi-Response Case Design	Step-by-Step - All
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	Yes
06	Seismic Design Category	A
07	Design System Omega0	2
08	Design System Rho	1
09	Design System Sds	0.5
10	Phi (Tension Controlled)	0.9
11	Phi (Compression Controlled Tied)	0.65
12	Phi (Compression Controlled Spiral)	0.75
13	Phi (Shear and/or Torsion)	0.75
14	Phi (Shear Seismic)	0.6
15	Phi (Joint Shear)	0.85
16	Pattern Live Load Factor	0.75
17	Utilization Factor Limit	1

Figure 26: Design Preferences for ACI 318-14

- **Columns**

  Each floor was designed with two types of columns to take up the load efficiently depending on their position. Each floor is assigned a column of larger section and the other of smaller sections. The former is termed as primary column and later is named secondary column respectively. All columns are square columns attributing to the almost square shape of the structure.

The commercial spaces which were noted to have greater live loads than that in BS model, required primary columns of sizes C1 1150 X 1150, C2 1100 X 1100, C3 1050 X 1050 and C4 1000 X 1000. These columns were assigned between Levels 1 to 10. On the other hand, secondary columns of size C5 900 X 900 was maintained constant between Levels 1 to 10.

The office spaces between Levels 11 to 21, needed primary columns of size C5 900 X 900 and secondary columns of size C6 750 X 750.

Residential floors, between Levels 21 to 42 required primary columns of size C7 650 X 650 and secondary columns of size C8 550 X 550.

The column at the vertex of the balcony was assigned a section C 800 X 800. It can be seen that this column required minimum reinforcement as the concrete section was itself capable of resisting the axial load. However, this section was chosen, firstly, to avoid the slenderness of the column all through its height of 165 meters and secondly to support the triangle shaped balcony which projects 7 meters from the floor edge.

- **Beams**

  As mentioned in the previous sections, although the peripheral beams around the flat slabs were assigned with rectangular sections of size 200 X 700, repeated iterations proved that it required section B1 350 X 800 to meet the design requirements.

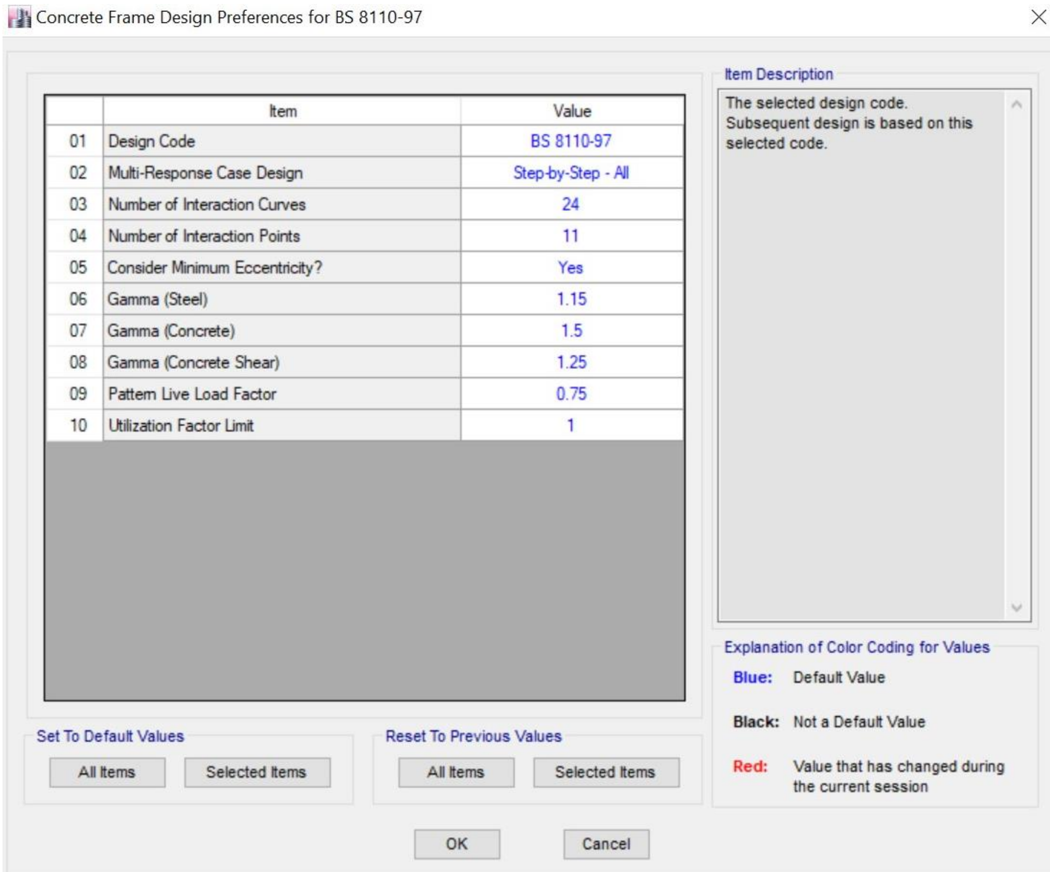
Similarly, the beams which connected cores named as lift beams, which were initially assigned with rectangular sections of size 300 X 500, required a sections of size LB 350 X 825.

However, it was noted that, the balcony beam and the tie beam required sections of 350 X 900 and 200 X 600 respectively.

- **Slabs**

  The initial sizes used for flat slabs were 200 mm thick slabs for floor areas and 150 mm thick slabs used for the balcony slabs. These slab sections passed the design check results.

6.3.1.2 BS Model



	Item	Value
01	Design Code	BS 8110-97
02	Multi-Response Case Design	Step-by-Step - All
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	Yes
06	Gamma (Steel)	1.15
07	Gamma (Concrete)	1.5
08	Gamma (Concrete Shear)	1.25
09	Pattern Live Load Factor	0.75
10	Utilization Factor Limit	1

Figure 27: Design Preferences for BS 8110-97

- **Columns**

  Just like the ACI Model, the BS Model also uses two types of columns the primary column and the secondary column. All columns are square in shape. The commercial spaces which were noted to have greater live loads than that in BS model, required primary columns of sizes C1 1050 X 1050 and C2 1000 X 1000. These columns were assigned between Levels 1 to 10. On the other hand, secondary columns of size C3 900 X 900 was maintained constant between Levels 1 to 10.

The office spaces between Levels 11 to 20, needed primary columns of size C3 900 X 900 and secondary columns of size C4 750 X 750.

Residential floors, between Levels 21 to 42 required primary columns of size C5 700 X 700 and C6 650 X 650 and secondary columns of size C7 550 X 550.

The column at the vertex of the balcony was assigned a section C 800 X 800. It can be seen that this column required minimum reinforcement as the concrete section was itself capable of resisting the axial load. However, this section was chosen, firstly, to avoid the slenderness of the column all through its height of 165 meters and secondly to support the triangle shaped balcony which projects 7 meters from the floor edge.

- **Beams**

As mentioned in the previous sections, although the peripheral beams around the flat slabs were assigned with rectangular sections of size 200 X 700, repeated iterations proved that it required section B1 300 X 700 to meet the design requirements.

Similarly, the beams which connected cores named as lift beams, which were initially assigned with rectangular sections of size 300 X 500, required a sections of size LB 300 X 600.

However, it was noted that, the balcony beam and the tie beam required sections of 350 X 900 and 200 X 600 respectively.

- **Slabs**

  The initial sizes used for flat slabs were 200 mm thick slabs for floor areas and 150 mm thick slabs used for the balcony slabs. These slab sections passed the design check results.

6.3.2 Parametric comparison of results of structural elements

To compare the parameters, we compare similar sections common to both ACI Model and BS Model. Sections similar in sizes help us to compare the parameters such as flexure, shear, axial strength, deflection and reinforcement required. We shall compare structural elements in the order beams, columns and slabs.

6.3.2.1 Beam section - BB 350 X 900

We have chosen the balcony beam of size BB 350 X 900 at Storey 32. The detailed concrete frame design report has been attached in the APPENDIX I and APPENDIX J. The following, is a gist of significant components of concrete frame design which is of importance for parametric comparison.

6.3.2.1.1 ACI Model

- *Flexure*

Live Load Reduction factor = 1

Φ_T = 0.9

Φ_{CTied} = 0.65

$\Phi_{CSpiral}$ = 0.75

Φ_{Vs} = 0.6

Φ_{Vns} = 0.75

For a Moment 3-3,

Design Moment = 250.24 kN-m

Required rebar for design moment = 1047 mm^2

Minimum rebar = 1007 mm^2

- *Shear*

Design Shear force = 42.6 kN

Required rebar for shear reinforcement = 311.71 mm^2 / m

6.3.2.1.2 BS Model

- *Flexure*

γ_c = 1.5

γ_s = 1.15

γ_m = 1.25

For a Moment 3-3,

Design Moment = 334.38 kN-m

Required rebar for design moment = 1165 mm^2

Minimum rebar = 486 mm^2

- *Shear*

Design Shear force = 31.64 kN

Required rebar for shear reinforcement = 389.18 mm^2 / m

Following were the comparisons made from the study:

- The Moment M_3 calculated as per BS Model provisions, 334.38 kN-m was greater than that of ACI Model, 250.24 kN-m. This can be attributed to the coefficients for dead loads in British Standards as compared to ACI standards.
- BS Model requires greater area of rebar 1165mm^2 for a given moment than ACI model value of 1047 mm^2. This can be attributed to the fact that Moments of British standards are higher than those of ACI standards.
- The minimum rebar required by ACI provisions, 1007 mm^2 is greater than BS provisions minimum reinforcement requirements of 486 mm^2.
- As far as the shear design provisions are considered, the shear force of beams V_2 of ACI Model is marginally higher than that of BS Model shear values. This can be due to the variation in load values of variable loads (Live Loads)
- The BS Model requires greater shear rebar than ACI Model.

It can be concluded that all the conclusions and inferences drawn for a beam section from the design and analysis of the above building closely agree with the results of review of literature and the hand calculations.

6.3.2.2 Column Section – C 650 X 650

We have chosen the column of size C 650 X 650 at Storey 32. The detailed concrete frame design report has been attached in the APPENDIX I and APPENDIX J. The following, is a gist of significant components of concrete frame design which is of importance for parametric comparison.

6.3.2.2.1 ACI Model

Load Reduction Factor = 0.417

Φ_T = 0.9

Φ_{CTied} = 0.65

$\Phi_{CSpiral}$ = 0.75

Φ_{Vs} = 0.6

Φ_{Vns} = 0.75

Axial Strength Pu = 2330.866 kN

Required area of rebar = 4225 mm^2 (1% rebar)

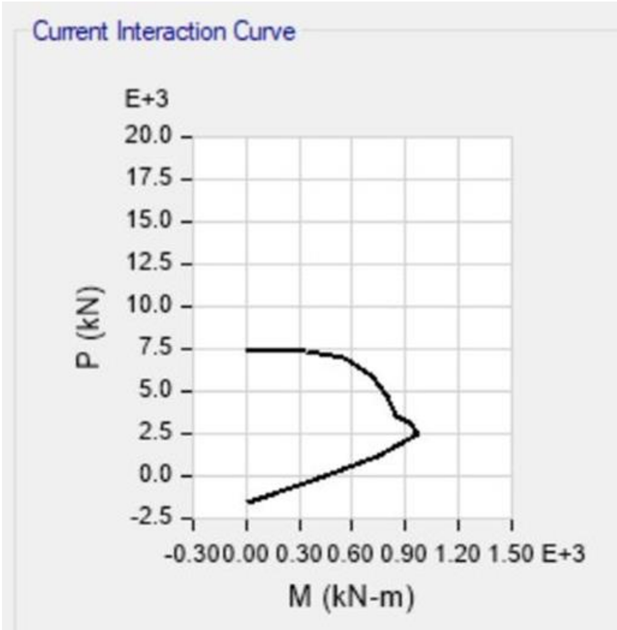


Figure 28: M-P Interaction curve for C 650 X 650 as per ACI 318-14 design

6.3.2.2.2 BS Model

γ_c = 1.5

γ_s = 1.15

γ_m = 1.25

Axial Strength N = 2200.53 kN

Required area of rebar = 1690 mm^2 (0.4% rebar)

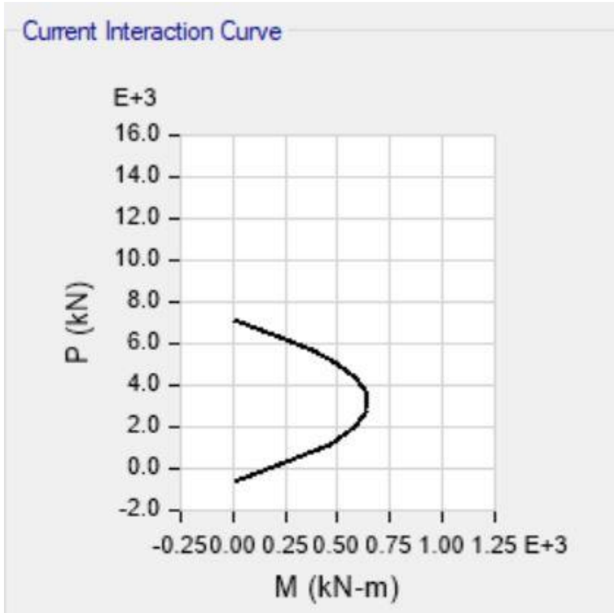


Figure 29: M-P Interaction curve for C 650 X 650 as per BS 8110-97 design

Following were the comparisons made from the study:

- The axial strength offered by ACI Model, 2330.86 kN was higher than that offered by BS Model, 2200.53 kN.
- The rebar area required for a given section is higher for ACI Model with a value of 4225 mm^2, while the rebar area required by BS Model was almost one-third of that required by ACI with a value of 1690 mm^2.
- As a result, the percentage of reinforcement required by ACI Model, 1% was higher than that of BS Model 0.4%.
- The M-P interaction curve for the column section 650 X 650 shows that, the curve for ACI Model is distorted and shows variation with respect to the BS Model. It can be seen that, ACI Model gives a higher Moment for a given P than the BS Model.

- It proves correct of the investigations made in the literature review which suggests that, the ACI Model curve deviates from BS Model curve and that ACI design specifications are less economical than BS.
- It can be deduced from our software aided design and analysis that, design of short columns with respect to ACI 318 is less economical and more conservative when compared with BS 8110-97 for a section of given area and cross section under respective loadings.

6.3.2.3 Slab Section

The design and analysis of slab showed that floor slabs required a depth of only 180 mm but a 200 mm slab was proposed as the slab was assumed to me a flat slab. The slab stresses showed that, they were minimal and passed the slab design check in ETABS.

Following are the observations made after the design and analysis of slabs:

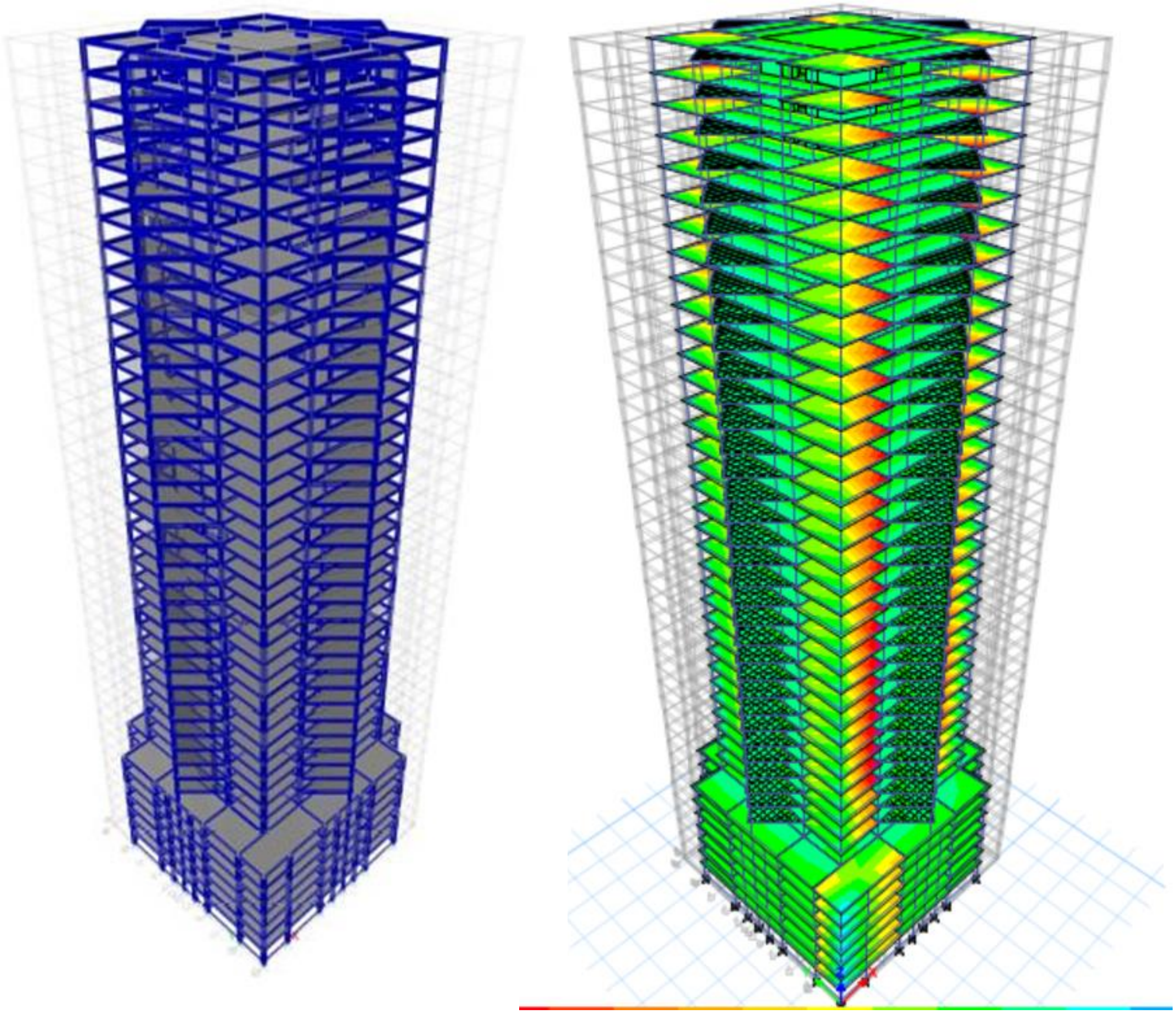


Figure 30: Maximum slab stresses at each floor (Illustrative)

6.3.2.3.1 ACI Model

The following are the slab stresses that were observed on the typical floors of ACI Model.

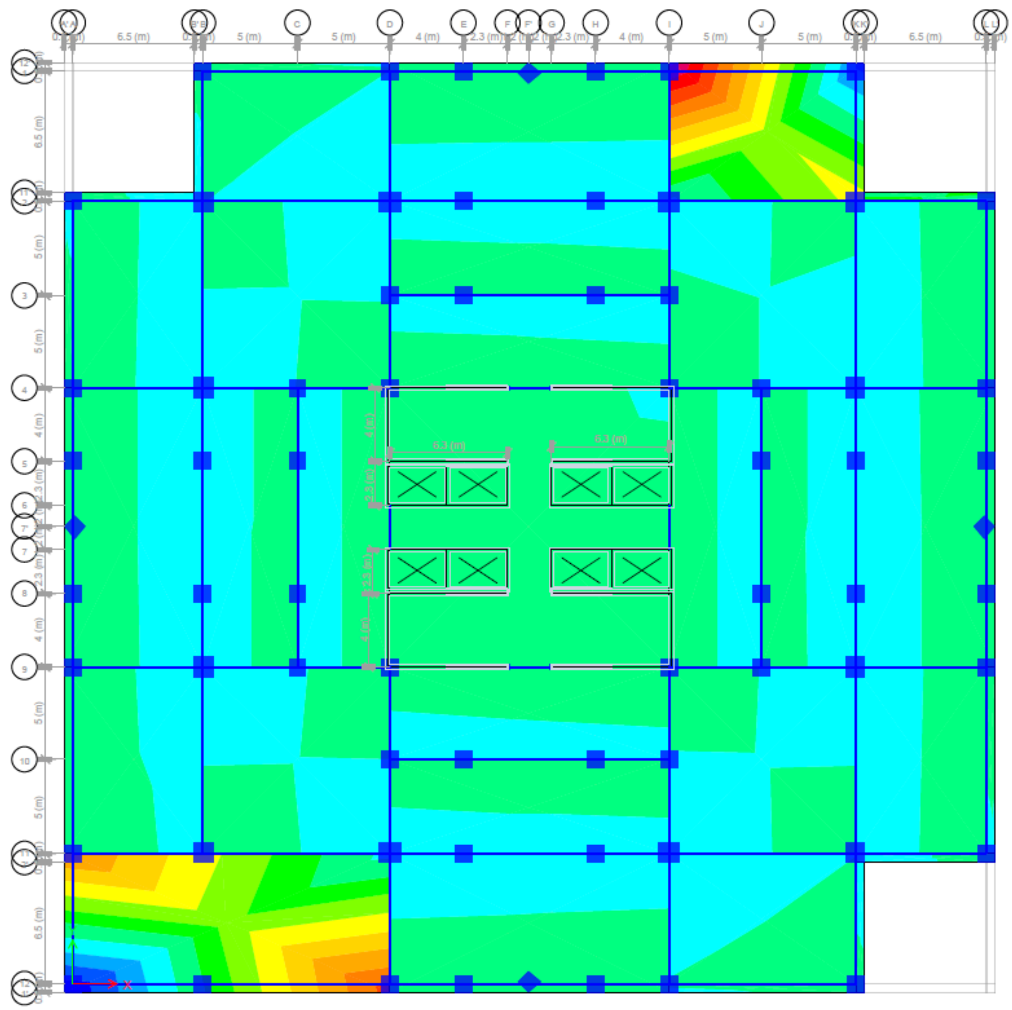


Figure 31: Slab stress at typical commercial levels (M11)

Storey 4 (Commercial space): Moment M11

Max + ve = 95.135 kN-m / m

Max – ve = -247.33 kN-m / m

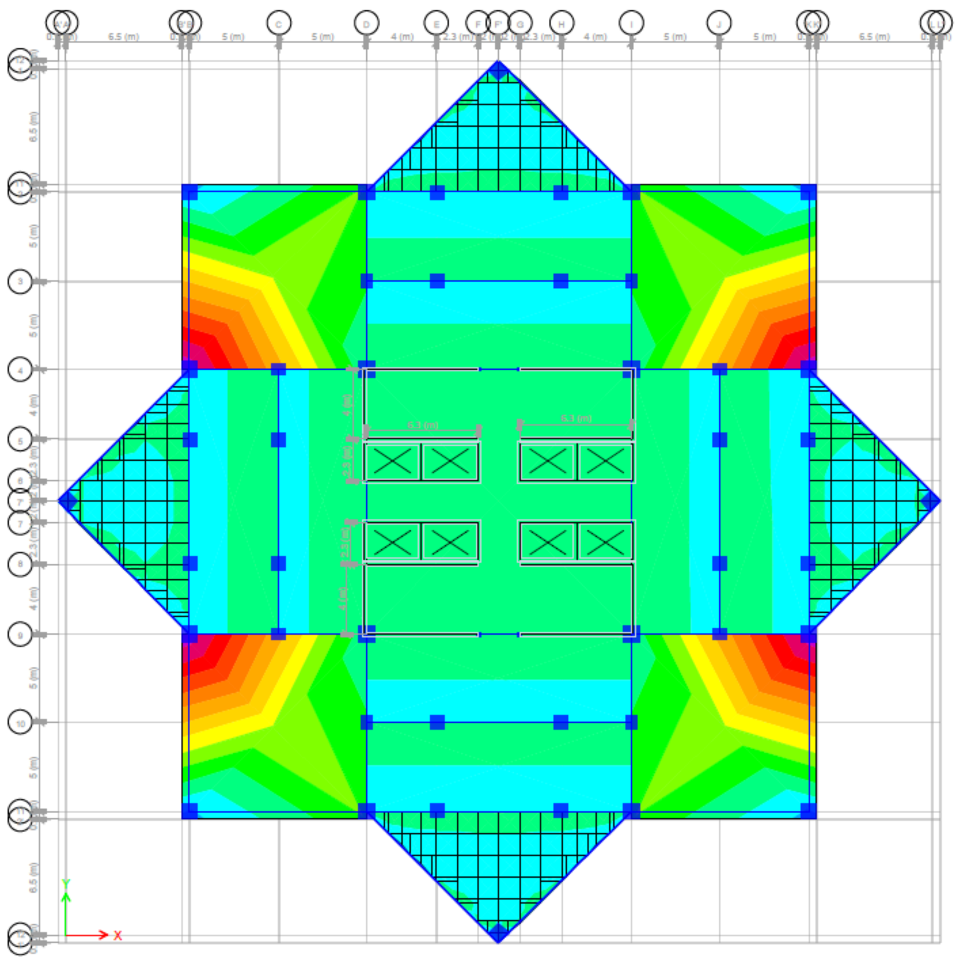


Figure 32: Slab stress at typical office levels (M22)

Storey 15 (Office space): Moment M22

Max + ve = 61.105 kN-m / m

Max – ve = -382.526 kN-m / m

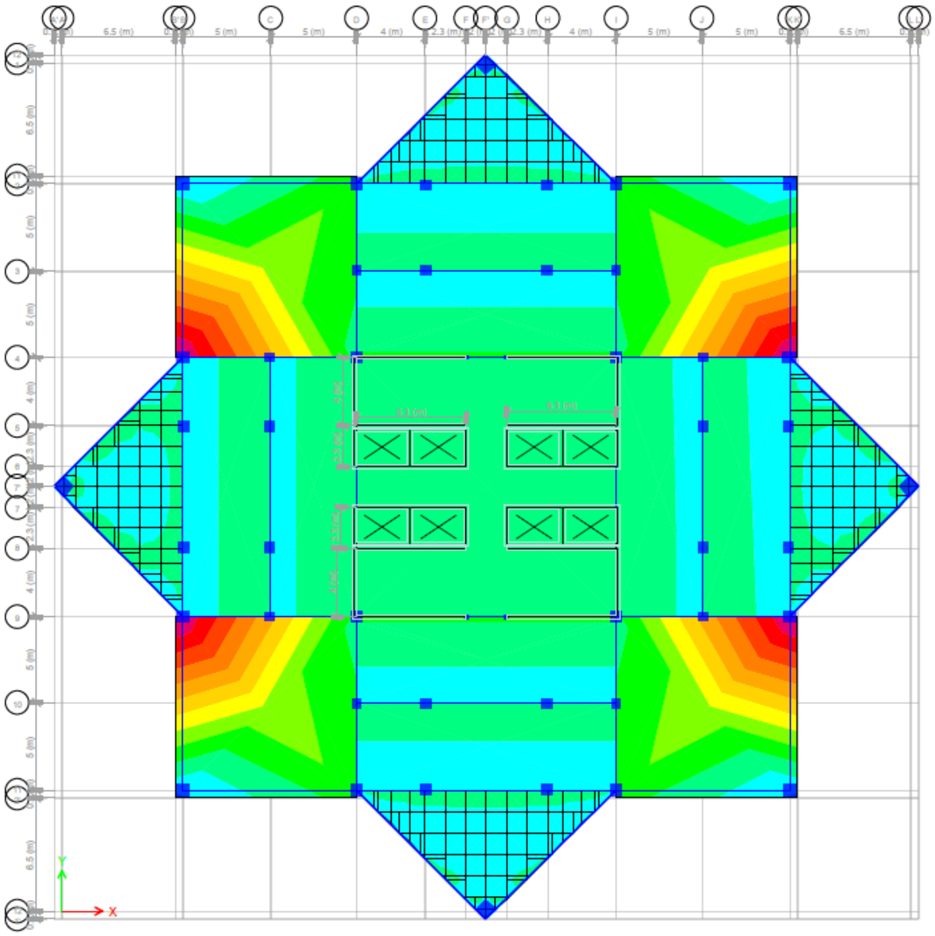


Figure 33: Slab stress at typical residential levels (M22)

Storey 26 (Residential): Moment M22

Max + ve = 90.18 kN-m / m

Max – ve = -361.773 kN-m / m

6.3.2.3.2 BS Model

The following are the slab stresses that were observed on the typical floors of BS Model.

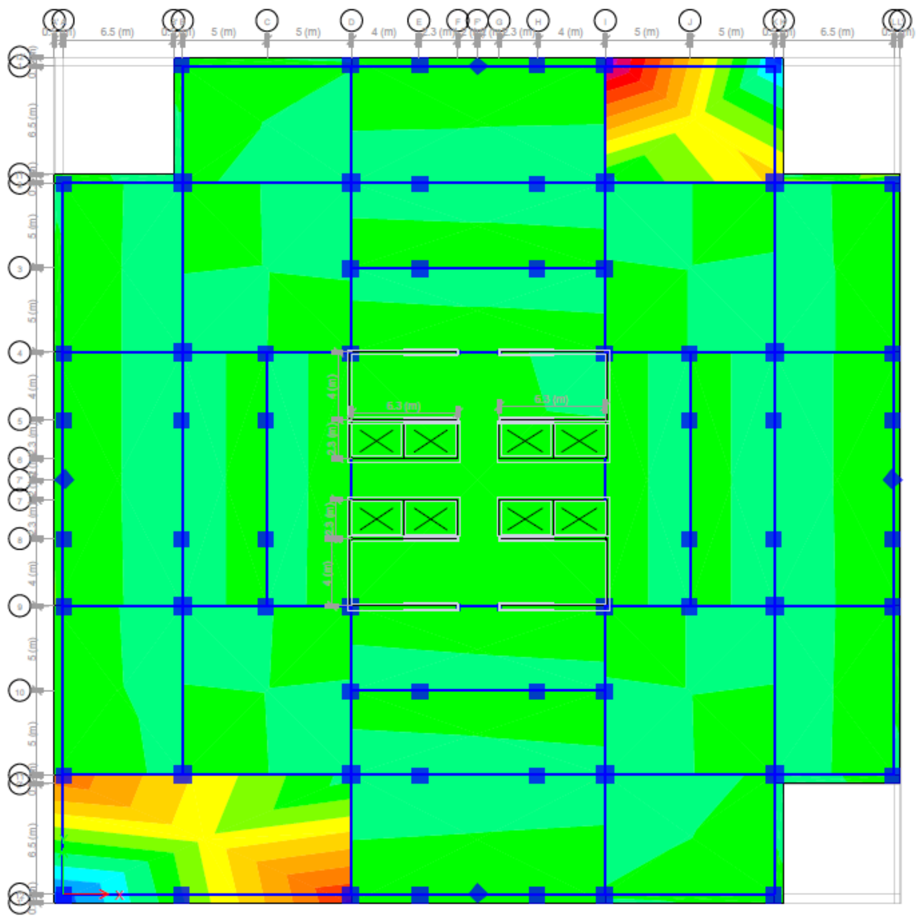


Figure 34: Slab stress at typical commercial levels (M11)

Storey 4 (Commercial): Moment M11

Max + ve = 91.433 kN-m / m

Max – ve = -237.238 kN-m / m

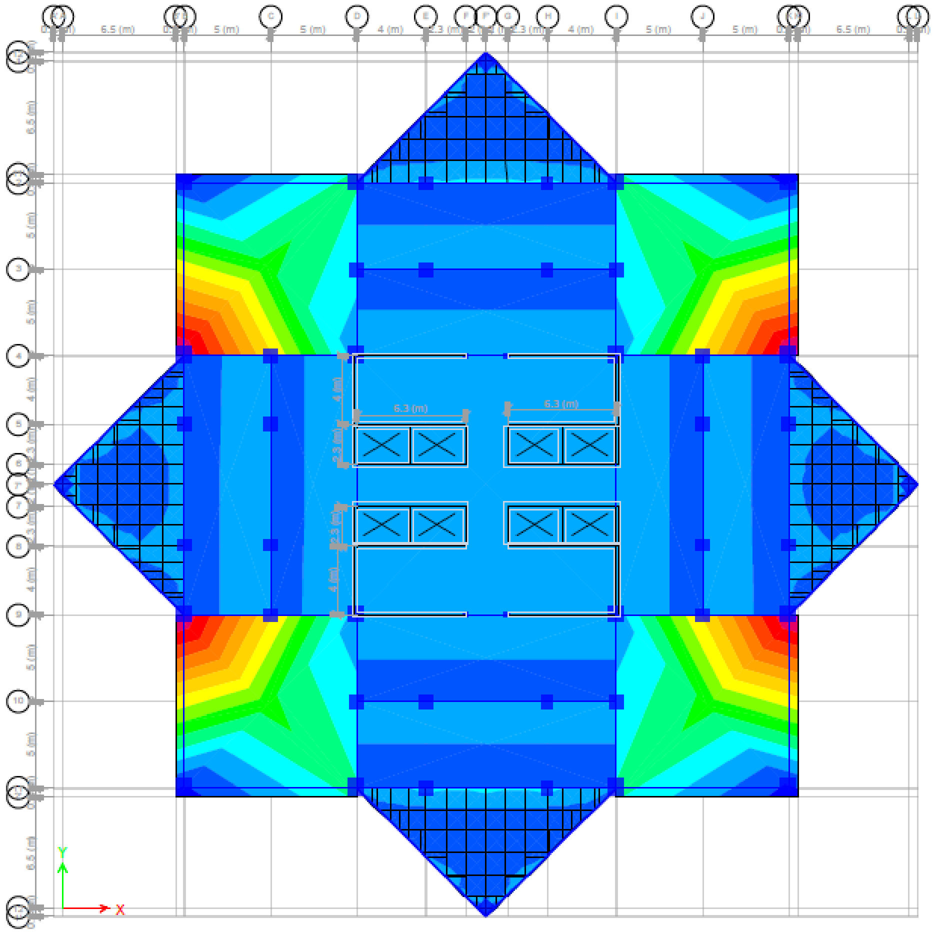


Figure 35: Slab stress at typical office levels (M22)

Storey 15 (Office space): Moment M22

Max + ve = 68.24 kN-m / m

Max – ve = -434.54 kN-m / m

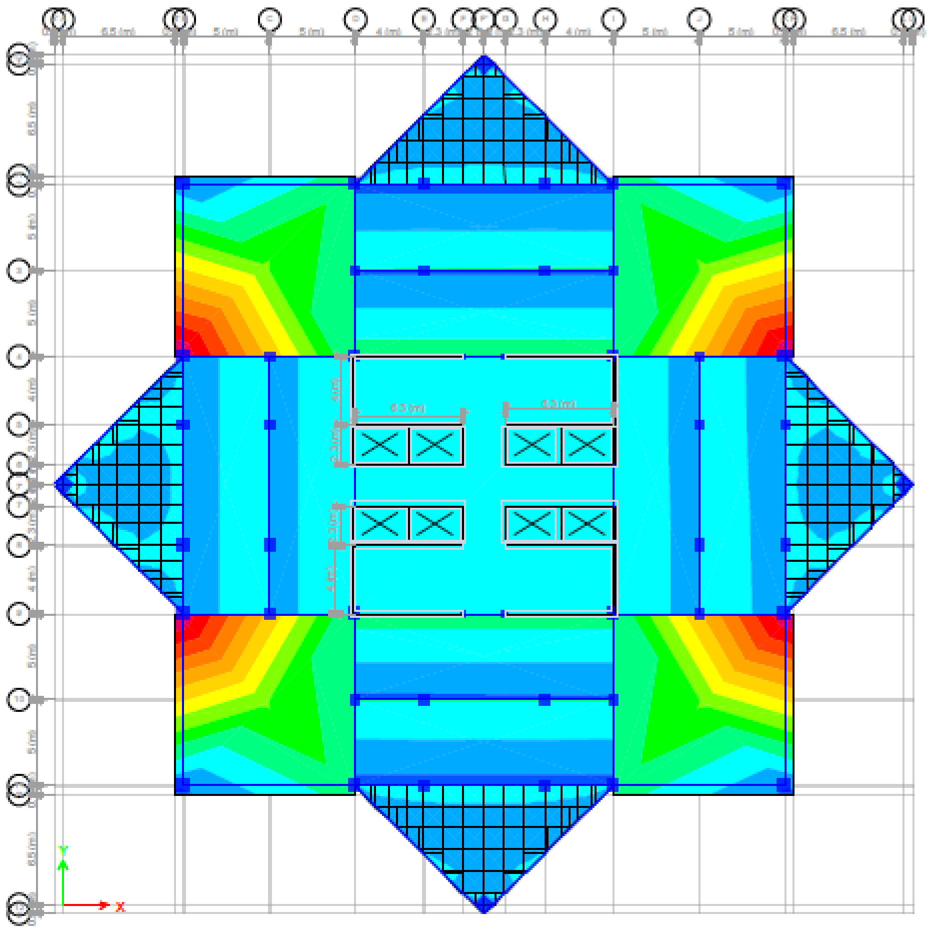


Figure 36: Slab stress at typical residential levels (M22)

Storey 26 (Residential): Moment M22

Max + ve = 97.22 kN-m / m

Max – ve = -392.98 kN-m / m

Following are the conclusions drawn by critically examining the slab stresses at different typical levels of occupancies with respect to ACI and BS model.

- Comparing the maximum positive and negative moments for a respective occupancy.
- At Storey 4, commercial space floor slab, the ACI Model gave maximum positive and negative moments as compared to the BS Model. This result can be attributed to the greater variation in live loads in ACI Model than BS Model for commercial occupancy.
- At Storey 15, Office floor slabs, The BS Model gave greater maximum positive and negative moments as compared to ACI Model.
- Similarly, at Storey 26, Residential floor slab, the BS Model slabs gave comparatively greater moments than ACI Model. However, this increase in marginal.
- Moments are taken as M11 and M22, as the maximum moment acting on a respective plane. M11 is the maximum moment acting along direction 1-1 and M22 is maximum moment acting along direction 2-2.
- Therefore, it can be deduced, from the given slab stress diagrams of commercial, office and residential floors of both codes ACI and BS respectively,
    - Slab stresses at the three typical levels in ACI Model are greater than BS Model.

- The BS Model slab results are observed to be more economical and safer.

6.3.2.4 Inferences from the comparison of frame sections of ACI and BS Models

Some of the inferences made from the critical review of the outputs of the concrete frame design are:

- The ACI Model requires larger sections to satisfy requirements of concrete frame design check as compared to the BS Model. This anomaly is significant among the frame elements that make up the commercial spaces. However, the columns, both primary and secondary, used in office and residential spaces are the same in ACI Model and BS Model. The peripheral beams that support the flat slabs in ACI Model require deeper sections than peripheral beams of BS Model.
- The frame sections used in BS Model although are comparatively smaller sections, the required reinforcement in frame sections of BS Model is greater than that required by ACI Model. This will be elaborated with an example in the upcoming section.
- The minimum reinforcement required by frame sections of ACI Model is greater than that required by BS Model. This will be elaborated with an example in the upcoming section.

- The shear reinforcement required by beams of BS Model is greater than that required by ACI Model.
- To compare the above results closely, a comparison of a frame section common to both BS Model and ACI Model is made below:
    - In Beam design, The Moment M_3 calculated as per BS Model provisions, was greater than that of ACI Model. This can be attributed to the coefficients for dead loads in British Standards as compared to ACI standards.
    - BS Model requires greater area of rebar for a beam, for a given moment than ACI model. This can be attributed to the fact that Moments of British standards are higher than those of ACI standards.
    - The minimum rebar in a beam, required by ACI provisions, is greater than BS provisions minimum reinforcement requirements.
    - As far as the shear design provisions are considered, the shear force of beams V_2 of ACI Model is marginally higher than that of BS Model shear values. This can be due to the variation in load values of variable loads (Live Loads).
    - The BS Model of beam design requires greater shear rebar than ACI Model.
    - The axial strength of a short column, offered by ACI Model, was higher than that offered by BS Model.

- The rebar area required for a given section is higher for ACI Model, while the rebar area required by BS Model was almost one-third of that required by ACI Model.
- As a result, the percentage of reinforcement required by ACI Model, 1% was higher than that of BS Model 0.4%.
- The M-P interaction curve for the column section, the curve for ACI Model is distorted and shows variation with respect to the BS Model. It can be seen that, ACI Model gives a higher Moment for a given P than the BS Model.
- It proves correct of the investigations made in the literature review which suggests that, the ACI Model curve deviates from BS Model curve and that ACI design specifications are less economical than BS.
- It can be deduced from our software aided design and analysis that, design of short columns with respect to ACI 318-14 is less economical and more conservative when compared with BS 8110-97 for a section of given area and cross section under respective loadings.
- Slab stresses at the three typical levels in ACI Model are greater than BS Model.
- The BS Model slab results are observed to be more economical and safer.

6.3.3 Parametric comparison of Deflection results

This section critically reviews the deflection of the frame elements and structure under both ACI and BS Models.

6.3.3.1 BS Model

The following image displays the deflected shape of the structure under service load conditions using BS design preferences.

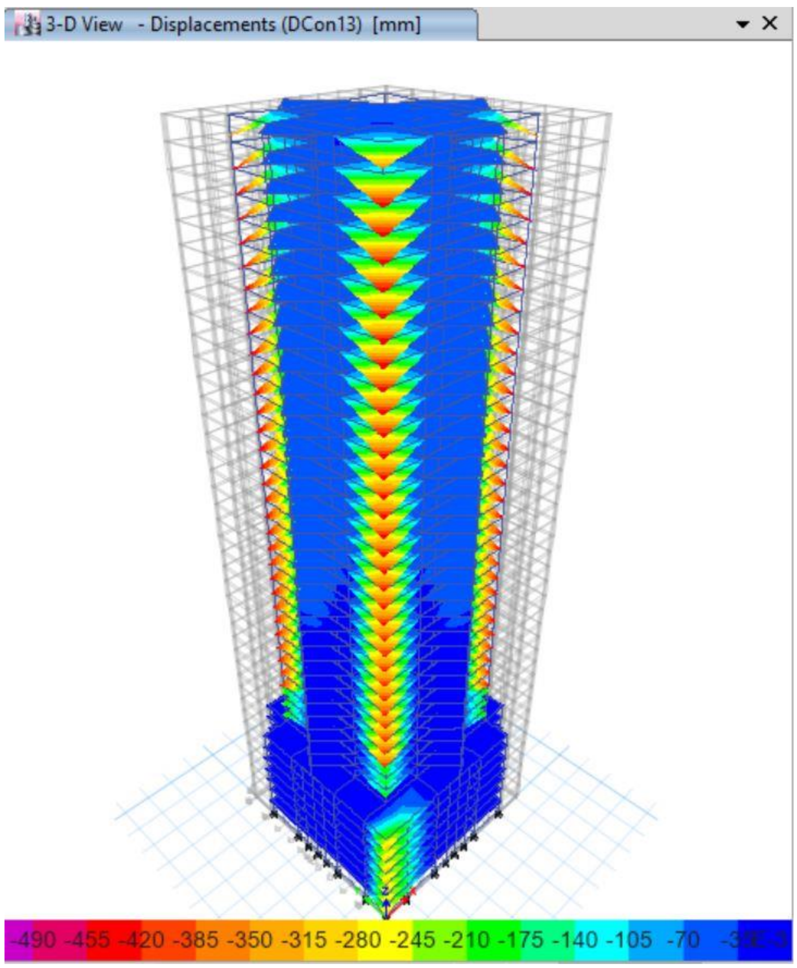


Figure 37: Deflected shape of structure under service load conditions

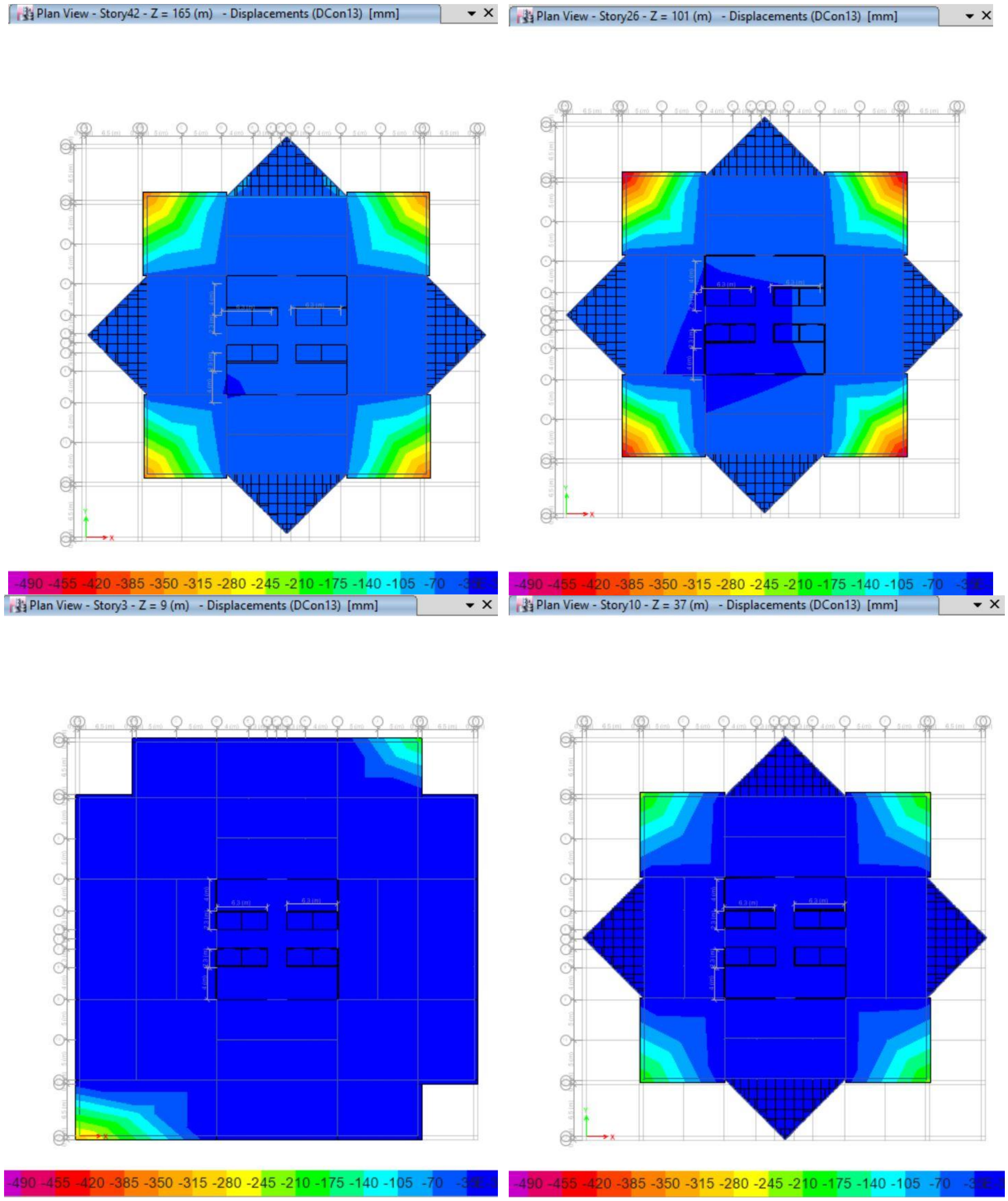


Figure 38: Deflection of slab in BS Model; (Clockwise from top left) Storey 42/Roof, Storey 26, Storey 10 and Storey 3

6.3.3.2 ACI Model

The following image displays the deflected shape of the structure under service load conditions using ACI design preferences.

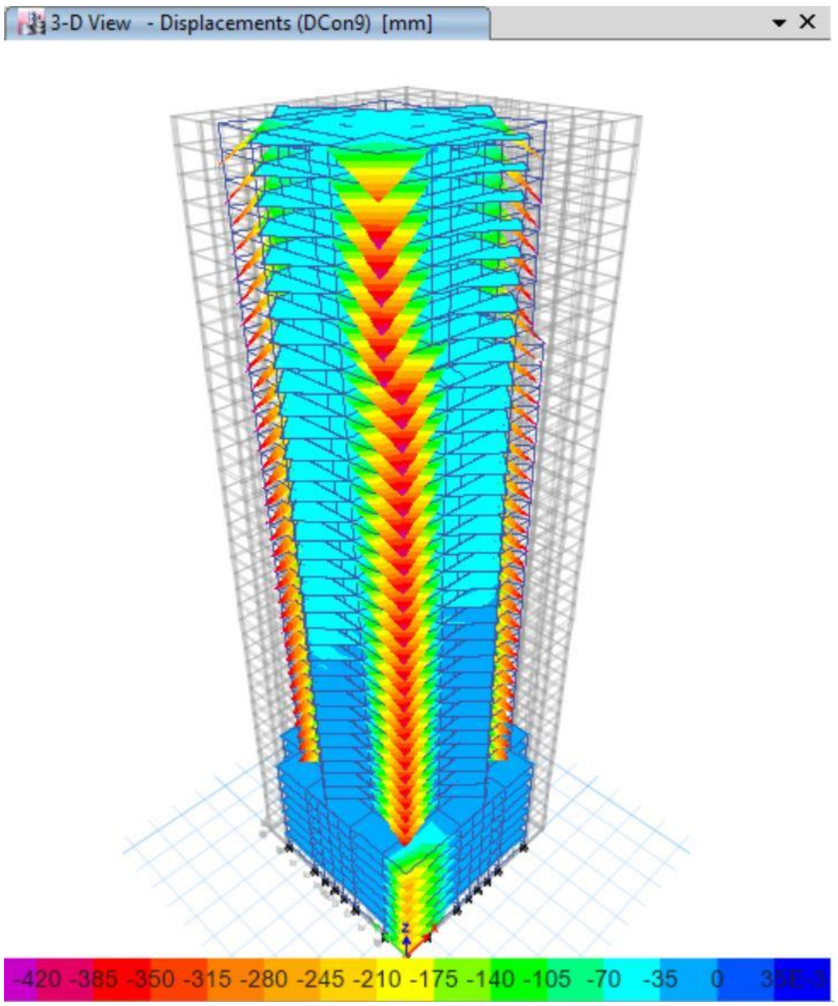


Figure 39: Deflected shape of structure under service load conditions

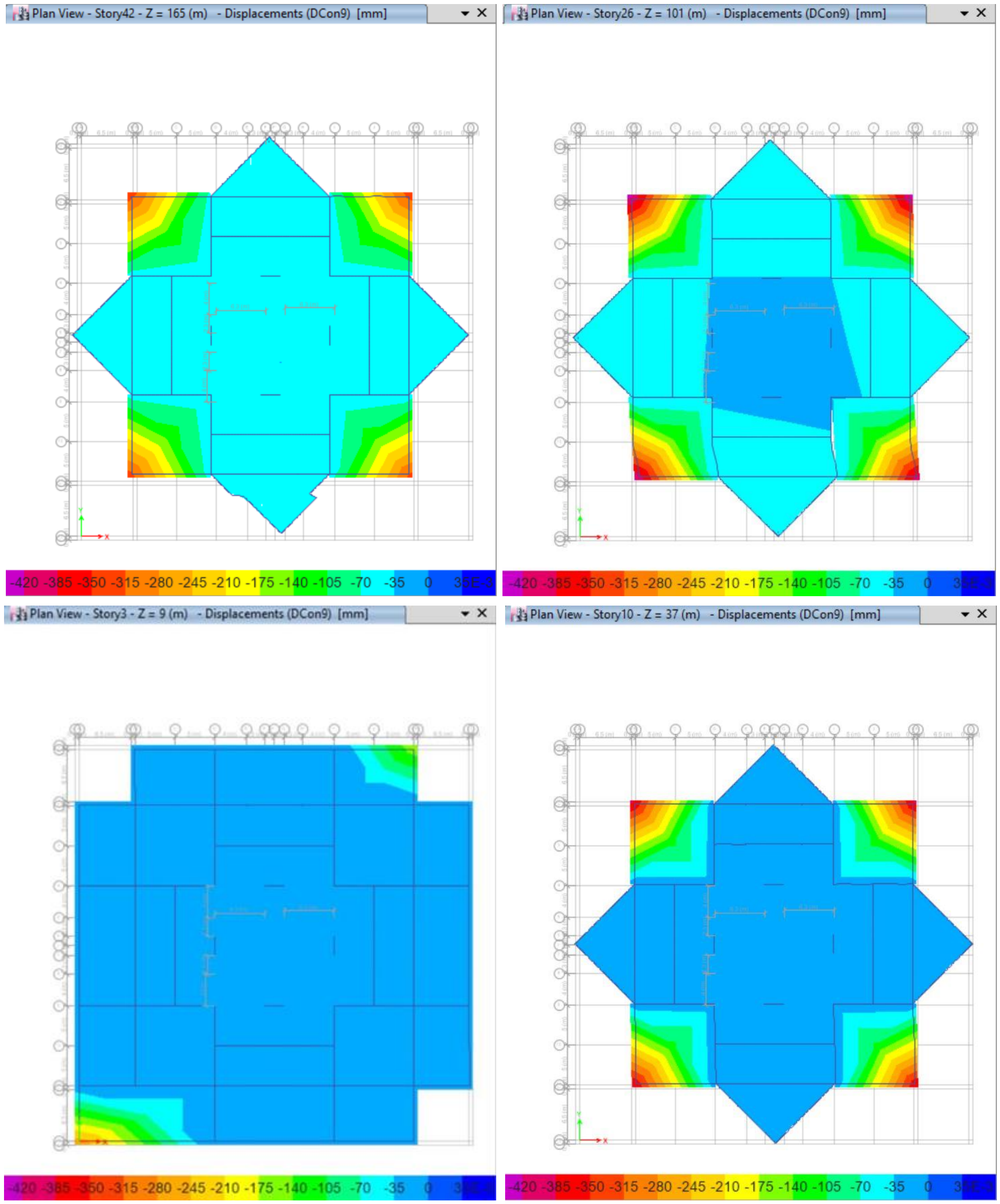


Figure 40: Deflection of slab in ACI Model; (Clockwise from top left) Storey 42/Roof, Storey 26, Storey 10 and Storey 3

6.3.3.3 Inferences of deflection comparisons in ACI and BS Models

The inferences that can be drawn from the above observations made on the maximum deflection that occurs:

- Deflection of structure designed using ACI concrete design preferences and loadings were greater than the deflections due to BS Model.
- A close examination of the ACI Model deflection, showed that, the deflection (displacement U_Z) of the slabs at serviceability condition was greater at top stories, however, this distinction did not vary much.
- A close examination of the BS Model deflection, showed that, the deflection (displacement U_Z) of the slabs at serviceability condition was greater at the top levels just like ACI Model. The deflection increased gradually above level 18.
- Although the deflection of slabs in BS Model at the bottom stories were low, they were minimal as compared to the deflections of the same slabs in ACI Model.
- The maximum deflection was observed at the corners of the star octagram shape in both models, with greater deflection observed in the ACI Model.
- Storey of BS Model gave lesser deflection as compared to ACI Model at all Levels of the structure.
- In short, it can be inferred that ACI design preferences and loads resulted in greater deflection in structural members as compared to a model designed to BS code provisions. However, it has already been discussed in the previous sections, that the BS 8110 provisions for maximum permissible deflection has

an upper hand over ACI provisions because it restricts deflection limit to a much lesser value, thus providing as extra margin of safety when the same span gave a greater deflection limit when analyzed with ACI 318.

6.3.4 Maximum moment – Moment about plane 3-3

This section discusses the moment acting on the entire structure.

6.3.4.1 ACI Model

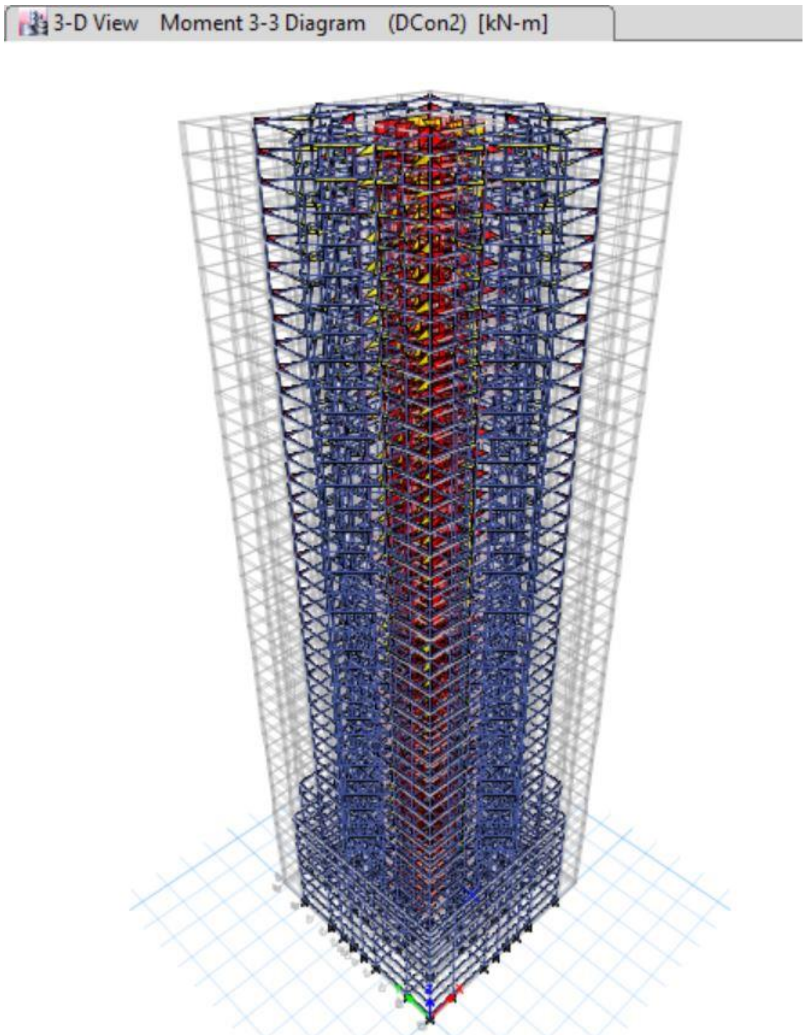


Figure 41: Maximum moment Moment 3-3 acting on the structure at Ultimate Design Loads – 3D View

6.3.4.2 BS Model

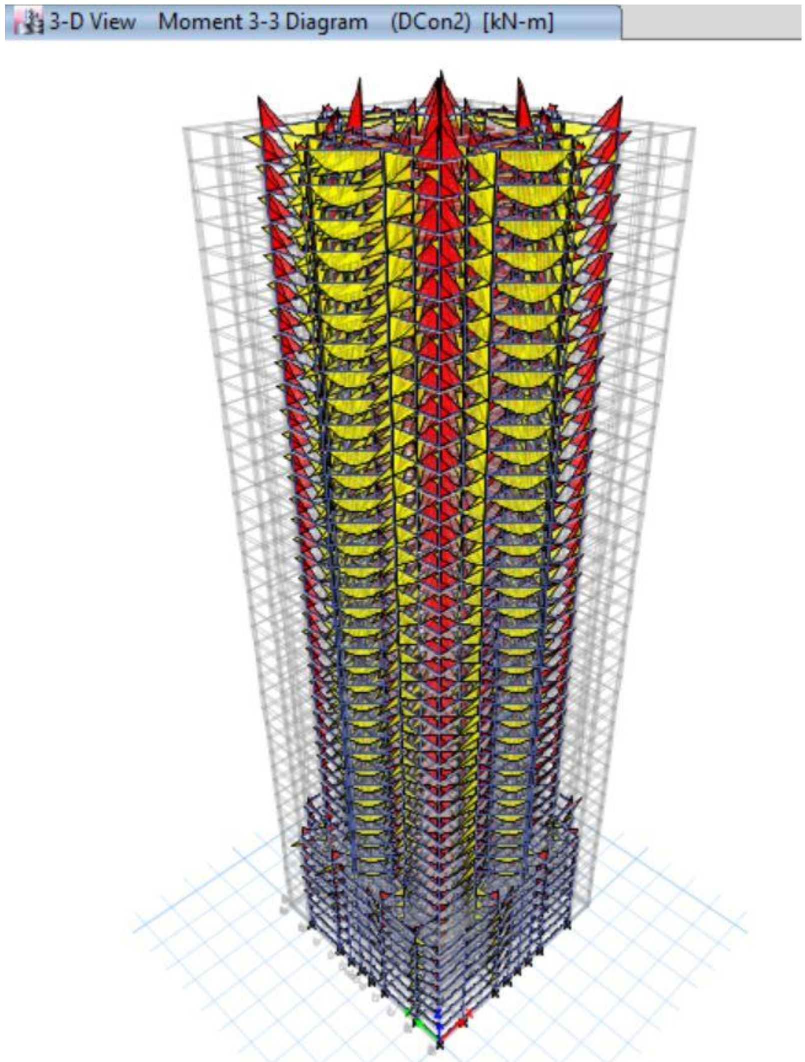


Figure 42: Maximum moment Moment 3-3 acting on the structure at Ultimate Design Loads – 3D View

6.3.4.3 Inferences from resultant design moment comparison

From the above figures it is evident that, the moments along plane 3-3 are greater in the BS Model as compared to the ACI Model. This maybe attributed to the partial factors of safety for ultimate loads using larger load safety factors. The following comparison of the moment diagram result of one of the structural members in the structure is elaborated below:

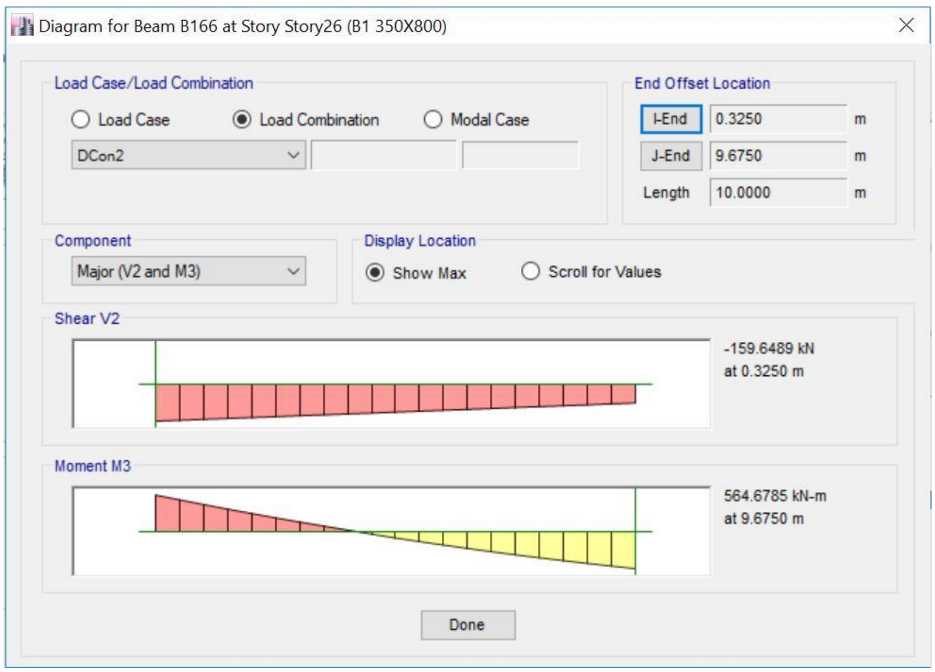


Figure 43: Shear force and Bending Moment Diagram for beam B1 350 X 800 at Storey 26 in ACI Model

The example presented is the periphery beam around the flat slab system which was initially given a section of 200 X 700 and after a number of iterations in analysis and design, this section needed to be increased in size as much as 300 X 850 for ACI Model and 300 X 700 for BS Model. The above image, shows the shear force and bending moment diagram for the 300 X 850 beam.

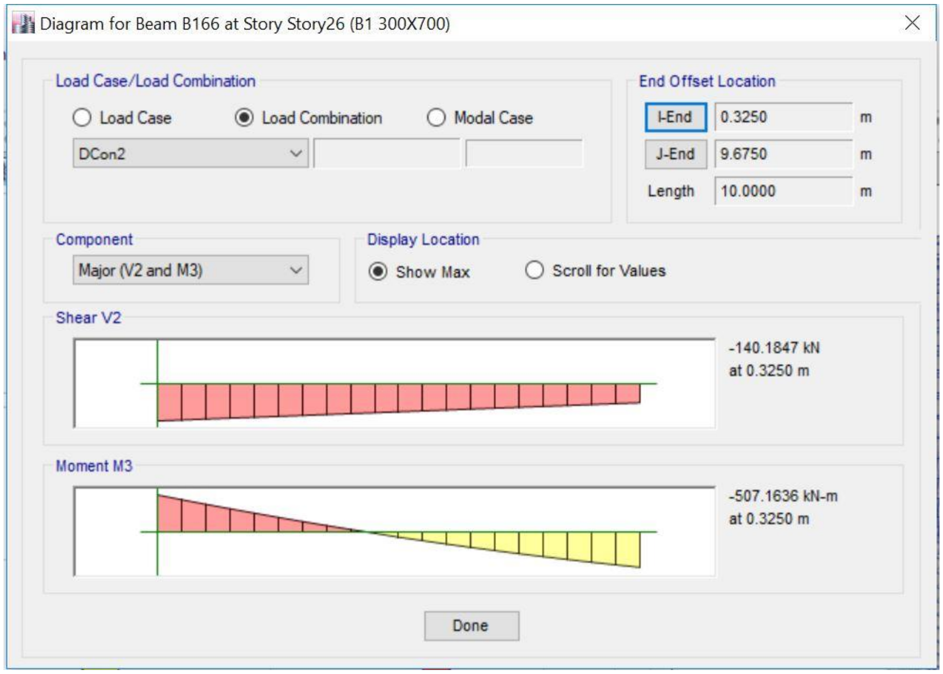


Figure 44: Shear force and Bending Moment Diagram for beam B1 300 X 700 at Storey 26 in BS Model

The above image, shows the shear force and bending moment diagram for the 300 X 700 beam. Although, the beam sections differ in size, their spans are the same which equals 10 m, and both beams belong to Storey 26 of their respective models. Hence, it may not be appropriate to compare the SF and BMD of the beams as their size influences their capacity to resist external loads and moments and it has been already discussed that BS Model beam requires a shorter size than ACI model. The following are the Maximum shear force and bending moments for the two model:

ACI Model

Shear Force (SF)	=	- 159.65 kN
Bending Moment Diagram (BMD)	=	564.68 kN-m

BS Model

Shear Force (SF)	=	- 140.18 kN
Bending Moment Diagram (BMD)	=	- 507.16 kN-m

The literature and reviews made in previous sections have shown that, there could be a trend of higher shear force and bending moments in BS Models attributing to the higher load safety factors and load combinations in BS Model.

6.3.5 Minimum area of longitudinal reinforcement and Required area of longitudinal reinforcements

There is a reasonable difference between the minimum area of reinforcement and required area of reinforcement. After analysis and design, the required reinforcement outputs at each level was displayed as shown below for both, ACI Model and BS Model.

6.3.5.1 ACI Model

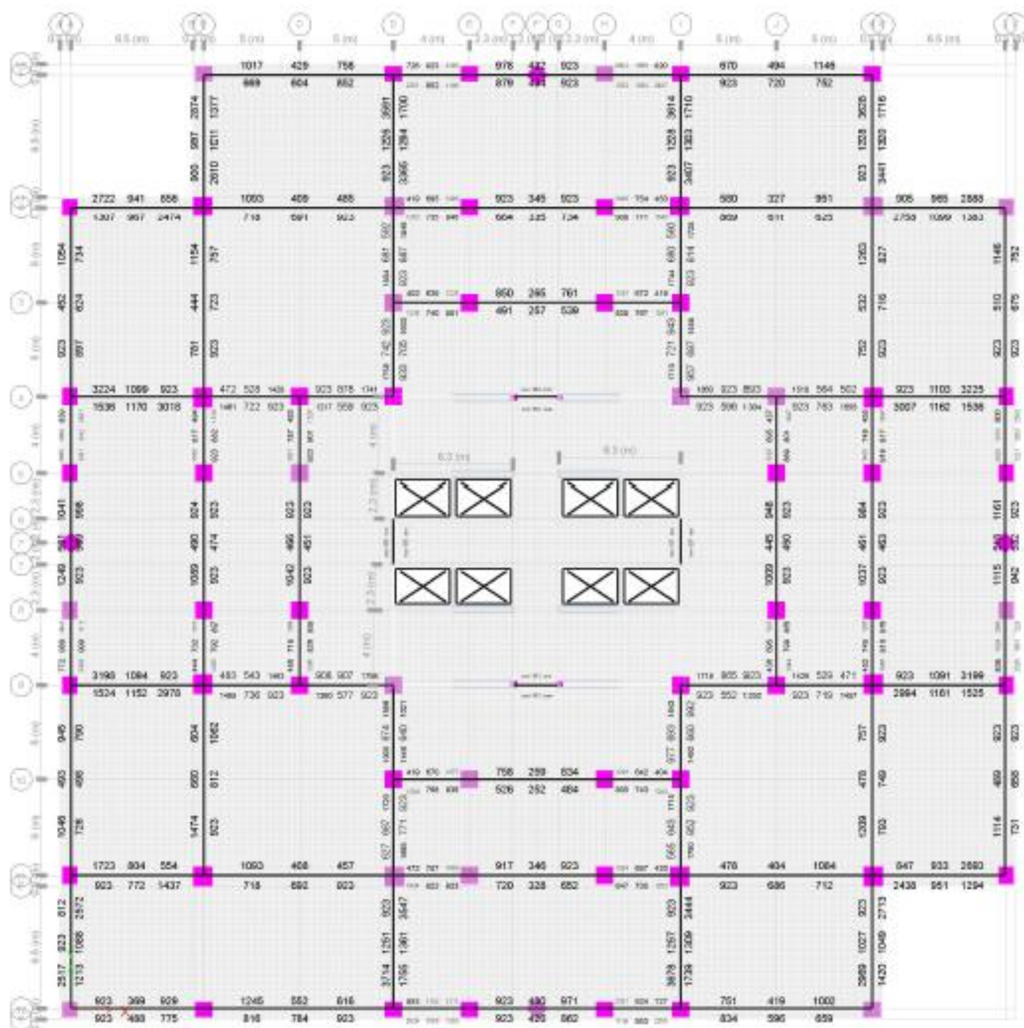


Figure 45: Required longitudinal reinforcement for frame elements at commercial level

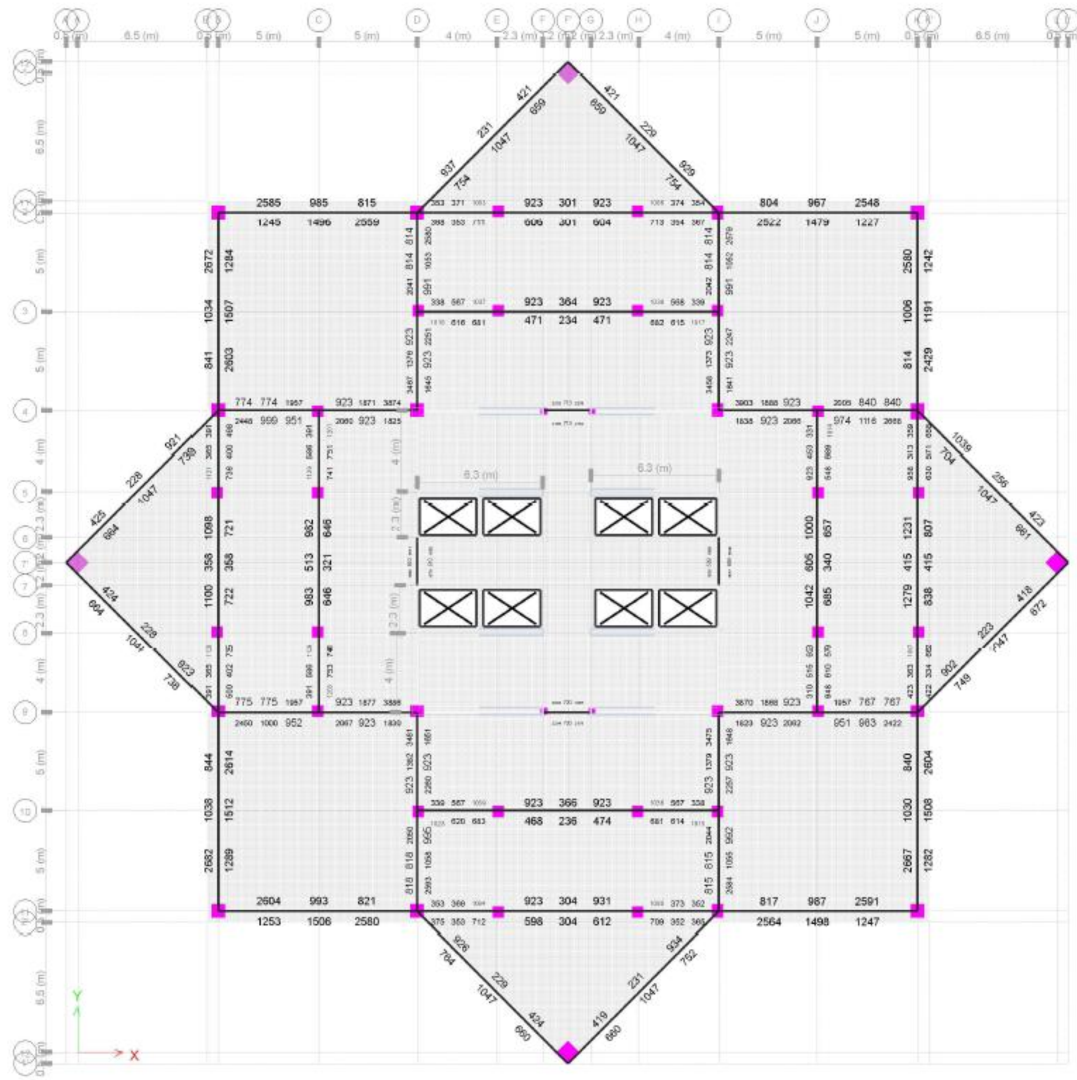


Figure 46: Required longitudinal reinforcement for frame elements at typical level

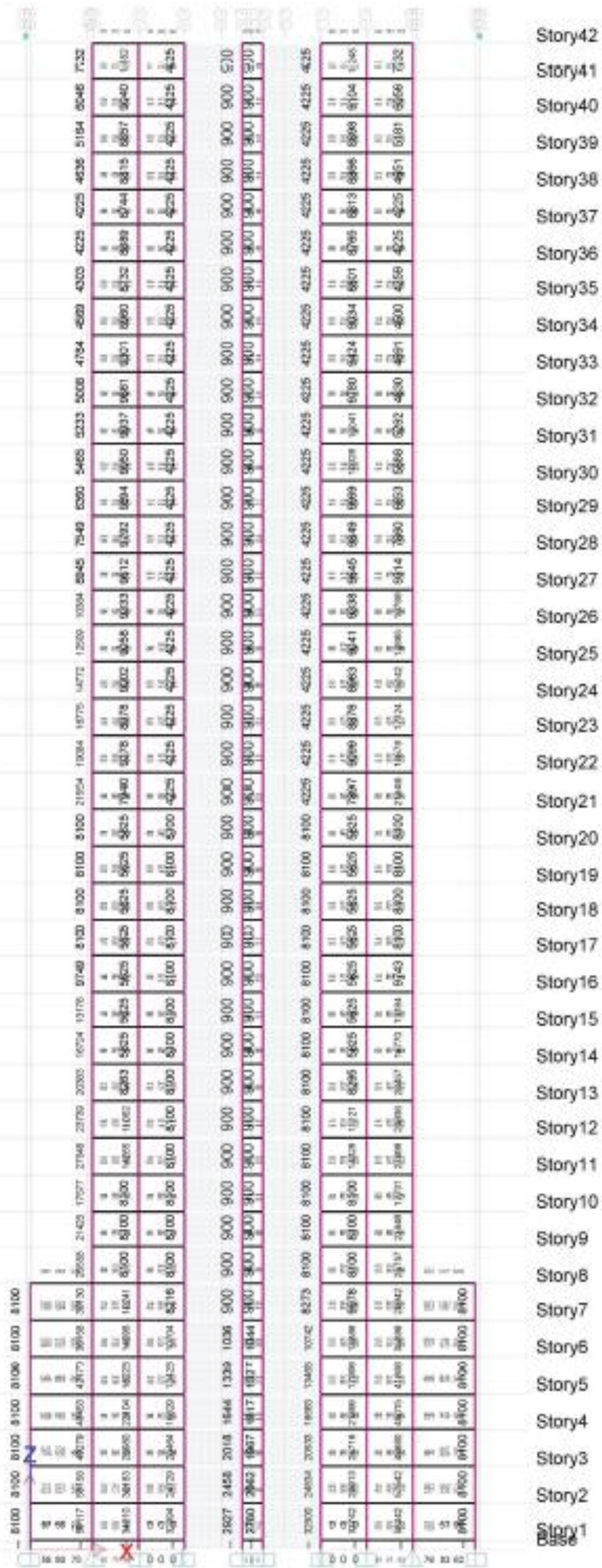


Figure 47: Required longitudinal reinforcement for columns in elevation

6.3.5.2 BS Model

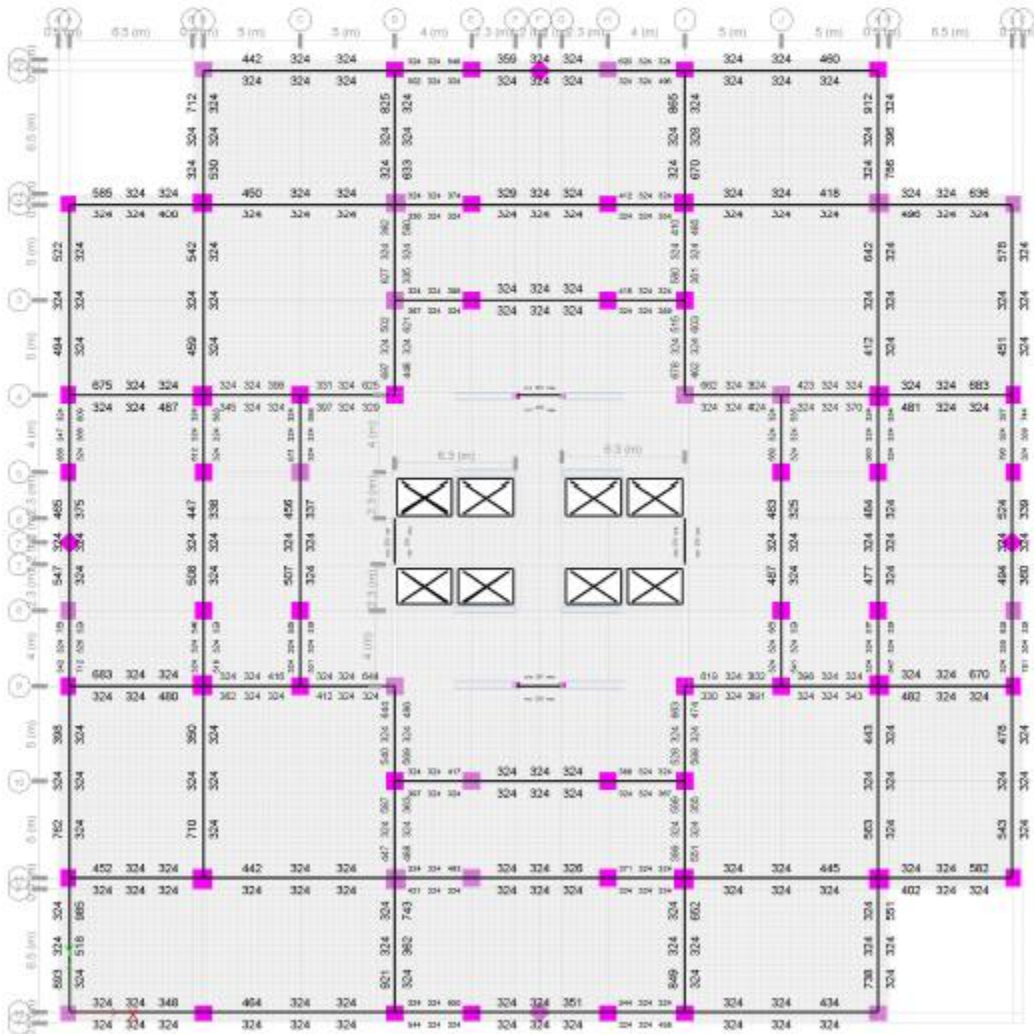


Figure 48: Required longitudinal reinforcement for frame elements at commercial level

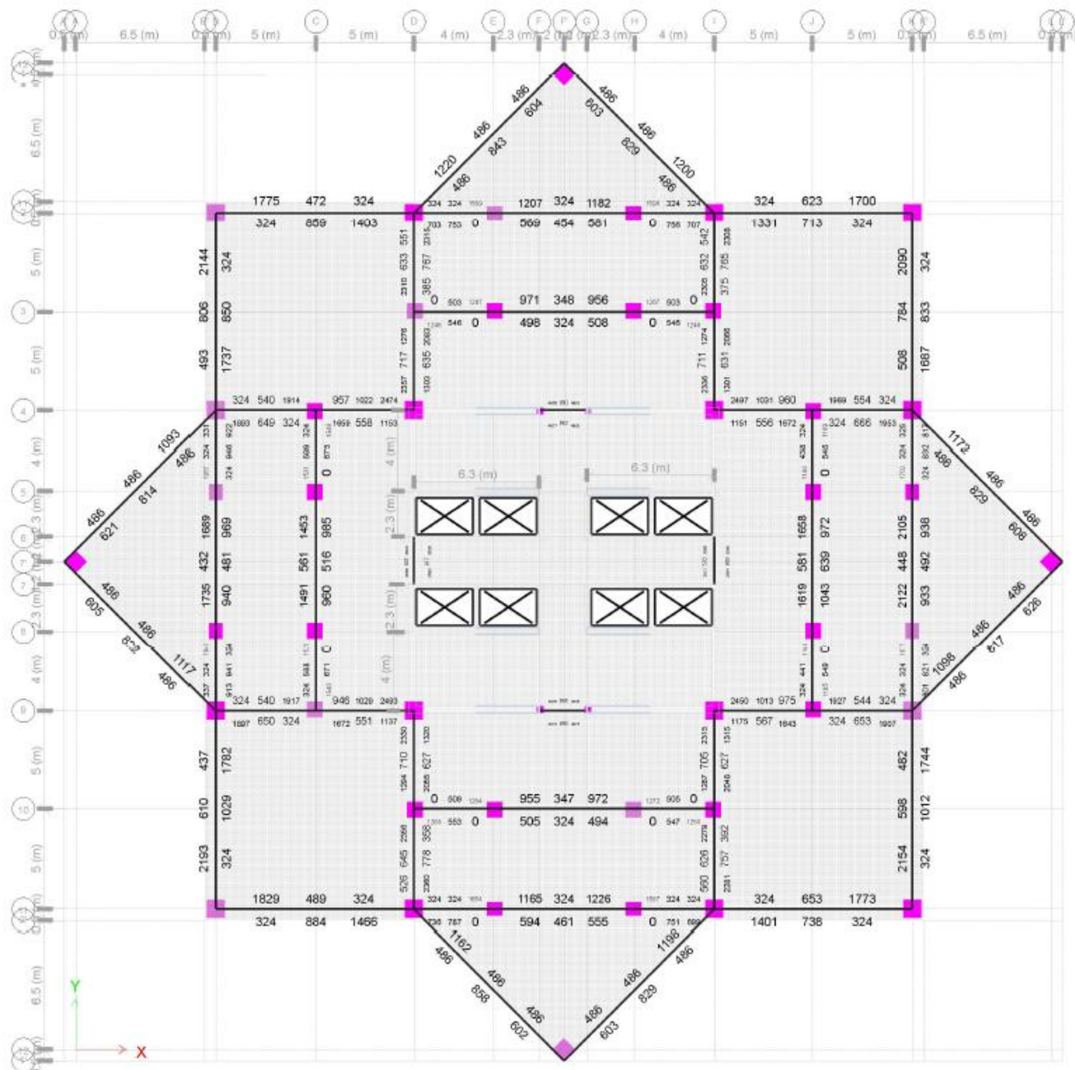


Figure 49: Required longitudinal reinforcement for frame elements at typical level

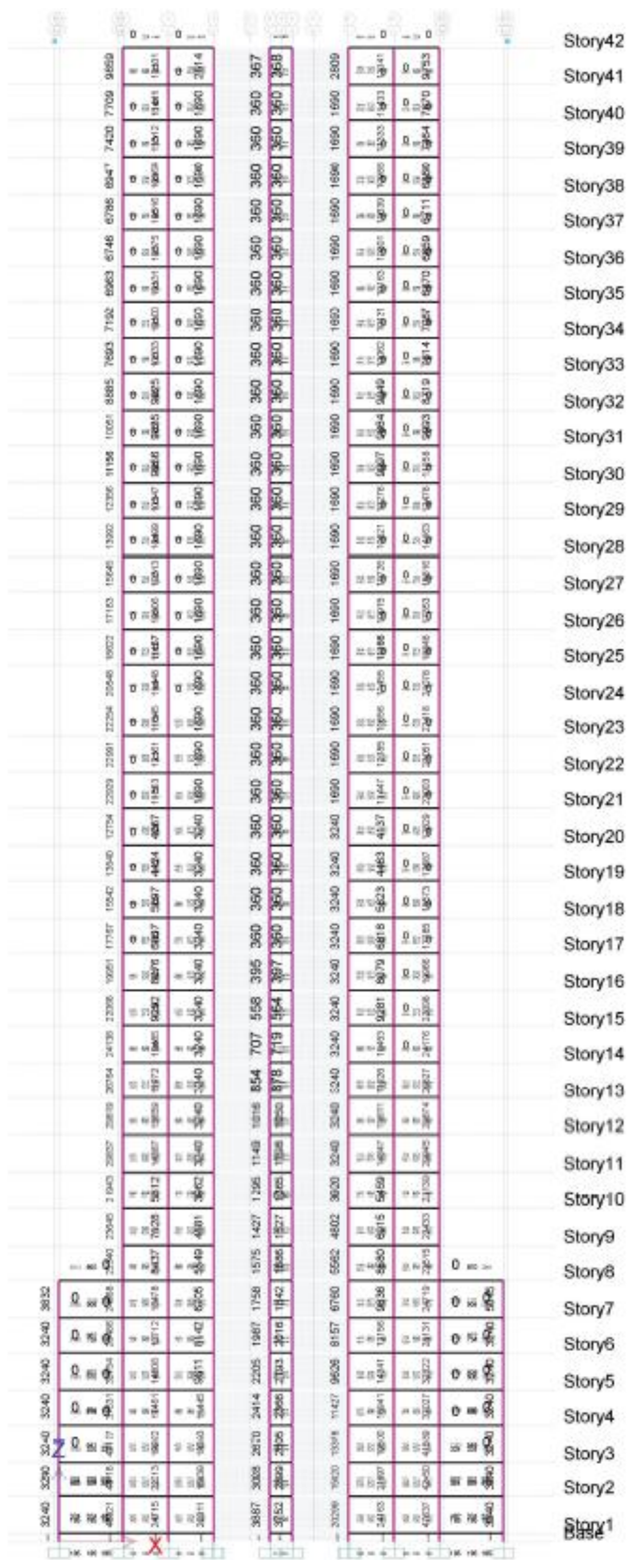


Figure 50: Required longitudinal reinforcement for columns in elevation

6.3.5.3 Inferences from comparison of minimum and required longitudinal reinforcement of different frame elements

6.3.5.3.1 Longitudinal and transverse reinforcement of beam B 350 X 900

BS Model

Design Moment and Flexural Reinforcement for Moment, M_3

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	0		0	0	0	0
Bottom (-2 Axis)		334.3793	1165	1165	486	0

Shear Force and Reinforcement for Shear, V_2

Shear V kN	Shear V_c / γ_M kN	Shear V_s / γ_M kN	Rebar A_{sv} /S mm²/m
31.647	152.6676	117.6	389.18

Figure 51: Flexural and shear reinforcement in BS Model for beam

ACI Model

Flexural Reinforcement for Moment, M_{u3}

	Required Rebar mm²	+Moment Rebar mm²	-Moment Rebar mm²	Minimum Rebar mm²
Top (+2 Axis)	87	0	65	87
Bottom (-2 Axis)	1047	816	0	1047

Shear/Torsion Design for V_{u2} and T_u

Rbar A_{vs} mm²/m	Rbar A_t /S mm²/m	Rbar A_l mm²	Design V_{u2} kN	Design T_u kN-m	Design M_{u3} kN-m	Design P_u kN
311.71	0	0	42.6404	40.2402	250.2396	0

Figure 52: Flexural and shear reinforcement in ACI Model for beam

- BS Model requires greater area of longitudinal reinforcement for a given moment than ACI model. This can be attributed to the fact that Moments of British standards are higher than those of ACI standards.
- The minimum longitudinal rebar required by ACI provisions, is greater than that required by BS provisions.

▪ The BS Model requires greater shear rebar than ACI Model.

▪ For the **BS Model**, the required rebar is, 1165 mm^2 for which we can provide,

Section	A	B	C	D	E	END	MIDDLE
B 350 X 900	4T20	2T20	4T20	2T20	-	ϕ10 @ 150C/C	ϕ10 @ 200 C/C

- For the **ACI Model**, the required rebar is, 1047 mm^2 for which we can provide,

Section	A	B	C	D	E	END	MIDDLE
B 350 X 900	3T20	2T20	3T20	2T20	-	ϕ10 @ 150C/C	ϕ10 @ 200 C/C

Hence, it is very evident from the bars we have provided that BS code provisions require greater number of bars although the diameter of bars remain the same.

6.3.5.3.2 Longitudinal and transverse reinforcement in column 650 X 650

BS Model

Axial Force and Biaxial Moment Design For N , M_2 , M_3

Design N kN	Design M_2 kN-m	Design M_3 kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
2200.5316	-106.8064	-85.7799	44.0106	44.0106	1690	0.4

Shear Design for V_2 , V_3

	Shear V kN	Shear V_c / γ_M kN	Shear V_s / γ_M kN	Rebar A_{sv} /s mm²/m
Major, V_2	79.0618	619.542	153.4005	722.77
Minor, V_3	73.9731	622.248	153.4005	722.77

Figure 53: Flexural and shear reinforcement in BS Model for column

ACI Model

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
2330.8662	-53.6837	-104.4873	80.9743	80.9743	4225	1

Shear Design for V_{u2}, V_{u3}

	Rebar A_v /s mm²/m	Design V_u kN	Design P_u kN	Design M_u kN-m	ΦV_c kN	ΦV_s kN	ΦV_n kN
Major Shear(V2)	0	126.6889	2860.6829	-154.0334	421.3406	0	421.3406
Minor Shear(V3)	0	98.9905	2846.5108	-121.4451	420.6532	0	420.6532

Figure 54: Flexural and shear reinforcement in ACI Model for column

- The rebar area required for a given section is higher for ACI Model, while the rebar area required by BS Model was almost one-third of that required by ACI.
- As a result, the percentage of reinforcement required by ACI Model, 1% was higher than that of BS Model 0.4%.
- It can be deduced from our software aided design and analysis that, design of short columns with respect to ACI 318 is less economical and more

conservative when compared with BS 8110-97 for a section of given area and cross section under respective loadings.

- These results were in contrast to the results obtained for beams.
- For the **BS Model**, the required rebar is, 1690 mm^2 for which we can provide

  C 650 X 650   6T20 bars
- For the **ACI Model**, the required rebar is, 4225 mm^2 for which we can provide,

  C 650 X 650   10T25 bars

6.3.5.3.3 Longitudinal reinforcement for 200mm slab

BS Model

For the slab at the commercial level, with a maximum positive moment of 95.135 kN-m and maximum negative moment of -247.33 kN-m, when calculated according to required reinforcement empirical equations, the Ast_{required} = 3614.12 mm^2. However, it was proposed to provide Ast_{provided} T20 @ 75mm c/c which resulted in a provided reinforcement of 4190 mm^2.

Ast_{required} = 3614.12 mm^2

Ast_{provided} T20 @ 75mm c/c = 4190 mm^2

Similarly, for the office floors where the values of moments were higher than those in the commercial floors. For the slab at the typical level, with a maximum positive moment of 61.105 kN-m and maximum negative moment of -382.526 kN-m, when calculated according to required reinforcement empirical equations, the Ast_{required} = 5589.7 mm^2. However, it was proposed to provide Ast_{provided} T20 @ 50mm c/c which resulted in a provided reinforcement of 6280 mm^2.

Ast_{required} = 5589.7 mm^2

Ast_{provided} T20 @ 50mm c/c = 6280 mm^2

ACI Model

For the slab at the commercial level, with a maximum positive moment of 91.433 kN-m and maximum negative moment of -237.238 kN-m, when calculated according to required reinforcement empirical equations, the $Ast_{required}$ = 3466.38 mm^2. However, it was proposed to provide $Ast_{provided}$ T20 @ 75mm C/C which resulted in a provided reinforcement of 4190 mm^2.

$Ast_{required}$ = 3466.38 mm^2

$Ast_{provided}$ T20 @ 75mm c/c = 4190 mm^2

Similarly, for the office floors where the values of moments were higher than those in the commercial floors. For the slab at the typical level, with a maximum positive moment of 68.24 kN-m and maximum negative moment of -434.54 kN-m, when calculated according to required reinforcement empirical equations, the $Ast_{required}$ = 6349.75 mm^2. However, it was proposed to provide $Ast_{provided}$ T20 @ 50mm C/C which resulted in a provided reinforcement of 6280 mm^2.

$Ast_{required}$ = 6349.75 mm^2

$Ast_{provided}$ T20 @ 50mm c/c = 6280 mm^2

Inferences from the slab designing:

- For the commercial levels, although, $Ast_{required}$ (required reinforcement) for the design moment in the slab of BS Model is greater than that of the ACI Model, the $Ast_{provided}$ (provided reinforcement) is the same for both slabs with T20 bars @ 75mm c/c spacing.
- For the typical levels, that accommodate offices and residential apartments, the trend was in contrast to that observed in the case of commercial levels. Here, the $Ast_{required}$ (required reinforcement) for the slabs in ACI Model was greater than that of BS Model slabs. However, slabs of both models were proposed to have T20 bars @ 50mm c/c spacing.

The following chapter, Chapter 8, discusses on the discussions drawn from the studies carried out in the last few chapters and their inferences.

CHAPTER 7

DISCUSSIONS AND RECOMMENDATIONS

The primary objective of this research work was to carry out a review of literature and compare it with the help of results from the analysis and design of a G+40 story building. An attempt was also made to theoretically evaluate the results and compare them with the available literatures and the designed models. This three-tier critical review of design code provisions was carried out step wise, by initially comparing the provisions different types of limit states, Ultimate and Serviceability according to different international design codes in general. This was followed by, a parametric study of the various parameters used in the empirical formulae that are applied to determine different physical quantities in design of beams, columns and slabs. The general and parametric study mentioned above, was performed for three different regional design codes, namely, ACI 318-14, BS 8110-97 and Eurocode2. For the next step of comparative study, the focus was placed primarily on two widely used design codes in the United Arab Emirates, the ACI 318-14 and BS 8110-97. These two codes were closely examined parametrically, in terms of permanent, variable and combination of loads, flexural and shear capacity provisions of beams, columns and slabs, deflection of frame elements and minimum and maximum amounts of longitudinal and transverse reinforcement. This review of parametric study and its findings and inferences were supported with a detailed analysis and design of a G+40 story, 165m tall, mixed-use building. This G+40 story building was designed according to both British and American concrete design standards. The software results from both codes were compared in terms of permanent, variable and combination of loads, flexural and shear capacity provisions of beams, columns and slabs, deflection of frame elements and minimum and maximum amounts of longitudinal and transverse reinforcement. The results of different sections were closely studied to provide enough evidences for the literature findings, theoretical results and software outputs to arrive at the best solution among the two codes in terms of adaptability to construction industry and environment and economy.

It was made sure to include maximum number of types of occupancies to get a vivid idea about the variation in results at different levels. The following are the discussion and inferences drawn from the three-tier critical review of BS 8110 (BS) and ACI 318 (ACI).

7.1 Loads and load combinations

- Variable load values of ACI are relatively greater for most of the occupancy types as compared to BS standards. It has wide variation specially for commercial occupancy type.
- BS 6399-96 uses greater number of load combinations than ACI.

7.2 Sectional capacity

- Both codes closely predict the flexural capacity of singly-reinforced sections.
- After the concrete frame design check, it was observed that ACI Model required larger sections to pass the design check. This was specially the case of frame elements of the commercial levels. However, the office and

residential floors of both models used the same column sections but different periphery beam sections. The beam section of ACI Model was greater than BS Model.

7.3 Beam Design

- Moment M3 of BS Model beams were greater in value than the moment value in ACI Model. These results were supported by the evidences from literature review and theoretical results.
- The 'required longitudinal reinforcement' in BS Model is greater than that required in ACI Model. These results were supported by the evidences from literature review and theoretical results.
- The 'required transverse reinforcement' is greater in the case of beams in BS Model as compared to their counterparts in ACI Model. However, these results contradicted the results calculated theoretically and evidences from the literature review, which suggested that ACI provisions required greater shear reinforcement as compared to BS provisions.
- Minimum amount of shear reinforcement required by BS Model was greater than ACI Model. However, literature evidences and theoretical investigations agreed with the minimum transverse reinforcement requirements.
- Maximum allowable spacing for shear reinforcements when calculated manually showed that ACI value of maximum transverse

reinforcement spacing was greater than the BS spacing recommendations.

7.4 Column Design

- With respect to column design results, axial strength offered by ACI Model was higher than axial strength offered by a column in BS Model. These results were supported by the evidences from literature review and theoretical results.
- The ACI Model column required a greater amount of longitudinal reinforcement as compared to a similar column designed using BS concrete design preferences. As a result, the percentage of longitudinal reinforcement required by ACI Model was higher than the BS Model value.
- The M-P interaction curve diagram, for a given column section was compared using either code provisions. And, it was concluded that ACI interaction curve was distorted and it deviated from the BS interaction curve. This implied that, columns designed to ACI design preferences gave a higher moment M for a given value of P.
- The results of the column design using software, agreed with the investigations in literature review and theoretical investigation.

7.5 Slab Design

- Slab stress at the three typical levels, commercial, office and residential levels, in ACI Model was greater than slab stress in BS Model.
- Ast $_{required}$ (required reinforcement) for the design moment in the commercial slab of BS Model is greater than that of the ACI Model, the Ast $_{provided}$ (provided reinforcement).
- Ast $_{required}$ (required reinforcement) for the typical slabs in ACI Model was greater than that of BS Model slabs.
- BS Model results were observed to be more economical.

7.6 Deflection

- Deflection of each level of the structure in ACI Model was higher than the deflections observed at each level of the structure in BS Model.
- The deflections were maximum at the corners of the star octagram shape as these corners supported the triangular balconies.

7.7 Maximum moment

The maximum moment 3-3 was observed to have higher values under ultimate load conditions using BS Model.

The above results prove that the BS design preferences has an upper hand over the ACI design preferences. However, in practical cases at consultants, in the event of conflict between various codes and standards, the most stringent conditions shall apply subject to the engineer's approval.

CHAPTER 8

CONCLUSION

8.1 Conclusion of research

It is believed that the objective of this research was met through a three-tier approach of critically reviewing the two design codes of interest – ACI 318 and BS 8110. Also, it was aimed at providing a general and parametric comparison of major code provisions of some regional design codes, namely, ACI 318, BS 8110 and EC2. This comparison was aimed at providing a helping hand to the structural engineers in the United Arab Emirates, accepting two facts. Firstly, that the United Arab Emirates is a home to structural engineers from different nations and expertise in different regional codes. Secondly, to maintain a uniformity and consistency in design and analysis of structures. This is mandatory, because this country does not have a complete national design building code and municipalities in the United Arab Emirates encourages design proposals conforming to mainly British and American standards. The BS 8110 is widely used by a large number of consultants. This trend began since late 1980's when most of the design works were carried out by UK nationals following BS 8110. However, by the late 1990's American structural consultants started to establish themselves in the United Arab Emirates and they practiced using the ACI 318 provisions. Since then, the BS and ACI provisions are equally followed in UAE.

With respect to the inferences from this research work, it was concluded that the British Standards (BS 8110 in this case) is preferred over American Standards. With respect to the comparison of design provisions of various types of limit states like under Ultimate and Serviceability conditions, and, because SI units are being of greater importance world-wide, it is voted that design of reinforced concrete sections using British Standards are preferred, owing to their adaptability to the construction industry and environment in this country favored by more economical solutions.

8.2 Scope for future research

This piece of research work focused primarily on comparing the code design provisions of ACI 318-14 and BS 8110-97 in terms of Ultimate Limit States of flexure and shear and, Serviceability Limit States of deflection. An attempt was also made to compare the section sizes and the required and minimum-maximum longitudinal and transverse reinforcement permitted under each code provisions. Future work could include research in terms of critically reviewing the untouched limit states like, torsion, axial compression, cracking, vibration etc. Also, this research carries out a general overview of some international codes and critical review of ACI 318-14 and BS 8110-97. There is plenty of scope to carry out above comparison in terms of any other international code as well. This research deals with Concrete Design Codes and can be extended to Steel and Composite design in future scopes. Hence, there is plenty of room for research as the subject of this dissertation is only a drop in an ocean of vast possibilities for research.

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APPENDICES

APPENDIX A

Ultimate factored design load

Considering a Dead Load (DL)	=	$2.5kN/m^2$
Live Load (LL)	=	$5kN/m^2$

- **ASCE 7-10 /ACI 318-14**

Ultimate Design Load	=	1.2 x DL + 1.6 x LL
	=	1.2 x 2.5 + 1.6 x 5
	=	$11\ kN/m^2$

- **BS 8110-97**

Ultimate Design Load	=	1.4 x DL + 1.6 x LL
	=	1.4 x 2.5 + 1.6 x 5
	=	$11.5\ kN/m^2$

- **Eurocode2**

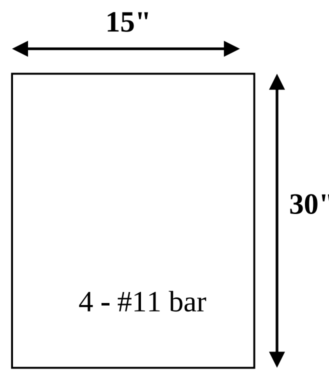
Ultimate Design Load	=	1.35 x DL + 1.5 x LL
	=	1.35 x 2.5 + 1.5 x 5
	=	$10.875kN/m^2$

Least ultimate design load is observed by EC2 provisions followed by ACI. Maximum ultimate design load is given by BS 8110 provisions.

APPENDIX B

Comparison of Ultimate Moment of Resistance using provisions of different design codes

ACI 318-14



$M_u = \phi A_S f_y (d - a/2)$

where, $(d - a/2) = 0.9d$

ϕ = 0.9

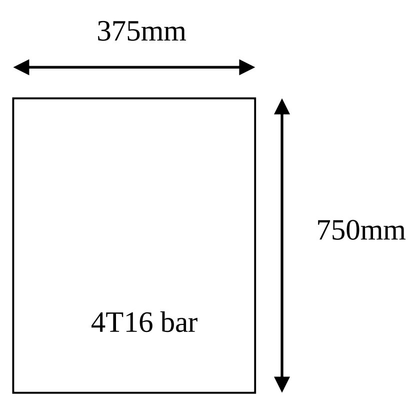
A_s = 6.24 in^2

d = 27.3"

fy = 60,000psi

Therefore, M_u = 0.9 x 6.24 x 60 x (0.9 x 27.3")

= 8279.1k-in (equivalent to 935.4kN-m)



BS 8110-97

Let

Fcu = 35MPa

Fy = 415MPa

d = 688mm

M_u = $0.156f_{cu}bd^2$

= $0.156 \times 35 \times 375 \times 688^2$

= 969.17kN-m

EC 2

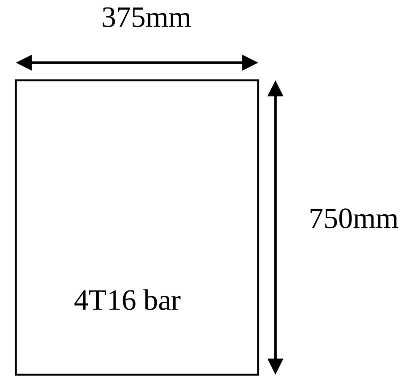
Let

Fcu = 28 MPa (cylinder strength is 0.8times cube strength)

Fy = 415MPa

d = 688mm

Assume 4T16 bars, $804mm^2$



M_{Rd} = $0.167fcu\ bd^2$

M_u = $0.167 \times 28 \times 375 \times 688^2$

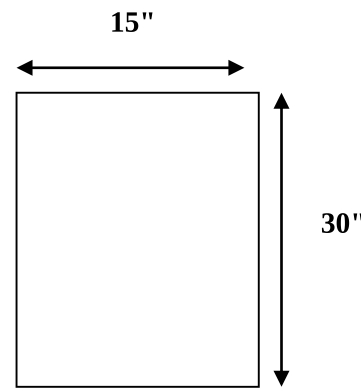
= 830 kN-m

APPENDIX C

Comparison of calculations for required area of longitudinal reinforcements in different design codes

ACI 318-14

We assume a design moment of M = 752.31kN-m



$M_u = \phi A_S f_y (d – a/2)$

where, $(d – a/2) = 0.9d$

ϕ = 0.9

A_s = 6.24 in^2

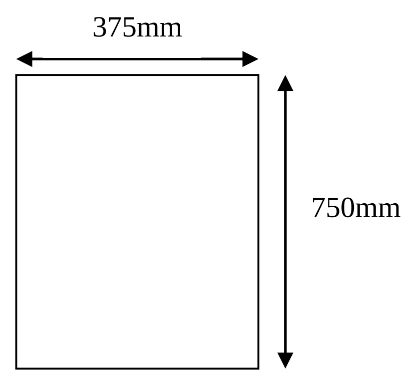
d = 27.3”

fy = 60,000psi

Therefore, A_s = $\frac{M}{\varphi\, fy\, (d - \frac{a}{2})}$

= (752.31) / (0.9 x 60 x 0.9 x 27.3”)

= 0.56 in^2 (which is equivalent 361.28 mm^2)



BS 8110-97

Let

Consider a design moment of M = 85 kN-m

Fcu = 35MPa

Fy = 415MPa

d = 688mm

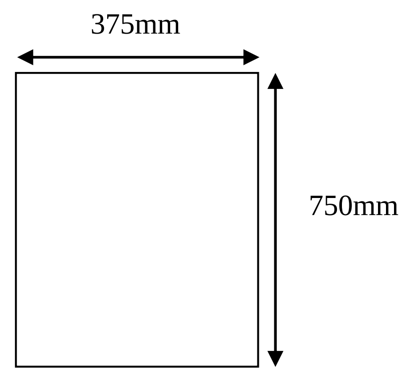
A_s = $\frac{Mu}{0.87\, fy\, z}$

= $(85 \times 10^6) / (0.87 \times 415 \times 653.6)$

= 360.19 mm^2

EC 2

Consider a design moment of M = 85 kN-m



Let,

Fcu = 28 MPa (cylinder strength is 0.8times cube strength)

Fy = 415MPa

d = 688mm

$$A_{st} = \frac{M}{0.87\, fy\, z}$$

$$= (85 \times 10^6) / (0.87 \times 415 \times 653.6)$$

$$= 360.19 \text{ mm}^2$$

APPENDIX D

Comparison of Axial Strength of columns of using provisions of different design codes

Assume, A_{st} = 1610 mm^2 (8T16 bars)

Area of concrete section = 200 x 500

ACI 318-14

f'_c = 5000 psi

f_y = 60,000psi

$P_{u,\,max}$ = 0.8 [0.85 f'_c (A_g - A_{st}) + f_y A_{st})]

= 0.8 [0.85 x 5000 ((500 x 200) – 1610) + 60 000 x 1610)]

= 411.806 klbs (which is equivalent to 1831.8 kN)

BS 8110-97

Let,

f_{cu} = 35MPa

f_y = 415MPa

N = 0.35 f_{cu} A_c + 0.7 A_{sc} f_y

= 0.35 x 35 x (200 x 500) + 0.7 x 1610 x 415

= 1692.7 kN

EC 2

Let,

f_{cu} = 28 MPa (cylinder strength is 0.8 times cube strength)

f_y = 415MPa

Pu = 0.57 f_{ck} A_c + 0.87 f_{yk} A_s

$$= \quad 0.57 \times 28 \times (200 \times 500) + 0.87 \times 415 \times 1610$$

$$= \quad 2177.29 \text{ kN}$$

APPENDIX E

Maximum permissible deflection limits

The following calculations elaborate on the comparison of maximum permissible deflection limits presented by ACI 318-14 and BS 8110-97 provisions.

For ACI 318-14 calculations, considering a beam of span 4.5m = 4500mm

For BS 8110-97 calculations, considering a beam of span 15' = 180"

- ❖ For those structural members which support or are attached to non-structural elements that are **not likely to be damaged by large deflections**

		ACI 318-14			**BS 8110-97**
$Deflection_{max}$	=	span / 240	$Deflection_{max}$	=	span / 250
	=	180" / 240		=	4500 /250
	=	0.75"		=	18mm
	= 19.05 mm (in Metric system)				

- ❖ For those structural members which support or are attached to non-structural elements that are **likely to be damaged by large deflections**

	ACI 318-14			**BS 8110-97**
Deflection_{max} =	span / 480		Deflection_{max} =	span / 500
=	180” / 240		=	4500 /500
=	0.38”		=	9mm
=	9.65 mm (in Metric system)			

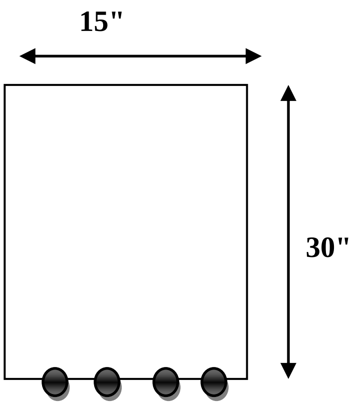
It shows that, for a concrete section with the same span, of beam/ slab, the ACI code gave a greater deflection limit than that of BS code limits. This can also be interpreted as, the lesser the maximum permissible deflection limit the more stringent the code provision is as far as deflection is concerned. In short, when there is a greater margin possible for deflection as in the ACI calculations, BS code restricts deflection limits at a lesser value, ie, it makes it safer.

APPENDIX F1

Flexural design of singly reinforced beam to ACI 318-14

Considering the example of a simple singly reinforced beam

Given,



$W_{LL} =$ 2.0 k/ft
$W_{DL} =$ 2.0 k/ft → including SW

L span = c/c spacing

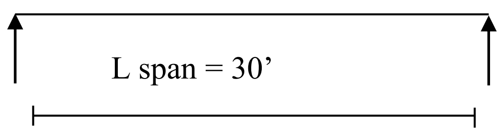
L span = 30’

W_{LL} = 2.0 kips/ft

W_{DL} = 2.0 kips/ft

f'_c = 5000 psi

f_y = 60000 psi



<u>Step1</u> - Compute factored load

$W_u = 1.2\ W_{DL} + 1.6\ W_{LL}$

$= 1.2\ (2) + 1.6\ (2) = \underline{5.6 \text{kips/ft}}$

$W_u = \underline{5.6 \text{kips/ft}}$

Step 2 - Compute Mu

$$\text{Mu @ midspan} = \frac{wl^2}{8}$$

$$= \frac{5.6 \text{ x } 30' \text{ x } 12''/\text{ft}}{8}$$

$$= 7560 \text{ k-in}$$

Step 3 - Compute Mn , req

$$\text{Mn, req} = \frac{\text{Mu}}{\varphi}$$

where, φ = 0.9 for flexure for a tension controlled section (as assumed)

$$\text{Mn, req} = \frac{7560}{0.9} = 8400\text{k-in}$$

Step 4 - Approximate value of d

Assume #9 bars

d = 30" – (1.128"/2) – 0.5" – 1.5"

= 27.4"

Step 5 - Calculate As req based on approx. Mn

Mn = As fy (d – a/2)

Assume, (d – a/2) = 0.9d

Mn = As fy (0.9d) = Mn, req

$$\text{Therefore, As,req} = \frac{Mn,req}{fy\ (0.9d)} = \frac{8400}{60\ (0.9*27.4")}$$

$$= 5.68 \text{ in}^2$$

Either use the design aids or with the help of knowledge of area of bars we calculate the number and size of bars

We consider 4 #11 bars , hence, As, prov = 6.24 in^2

→ Recalculating 'd'

$d = 30' – (1.41''/2) – 1.5''- 0.5''$

$= 27.3''$

<u>Step 6</u>– Spacing requirements

1. Spacing requirements

$$S_b = \text{max of} \begin{cases} d_{bar} = 1.41'' \\ 1'' \end{cases}$$

2. Diameter of bend

$d_{bend} = 4d_{stirrup}$

$= 4 \times 0.5''$

$= 2''$

ACI Checks

I. If bars fit within the base width

1. $2(cover) + 2d_{stirrup} + nd_{bar} + (n-1)s_b \leq b$

$= 2(1.5'') + 2(0.5'') + 4x1.41'' + 3(1.41'')$

$= 13.9'' < \longrightarrow 15''$ OK

2. $2(cover) + 2d_{stirrup} + d_{bend} + (n-1)s_b + (n-1)d_b \leq b$

$= 2(1.5'') + 2(0.5'') + 2'' + 3(1.41'') + 3(1.41'')$

$= 14.5'' < \longrightarrow 15''$ OK

This completes the check that all bars will fit within the available base width

II. Nominal moment capacity of actual section

1. β_1 = 0.85 -0.05 (f'c – 4) $\geq$ 0.65

   = 0.8

2. c = $\frac{As\ fy}{0.85\ f'c\ \beta 1 b}$

   $= \frac{6.24 \times 60}{0.85 \times 5 \times 0.8 \times 15''}$

   = 7.3"

C is used to determine if steel has yielded & if steel is in tension controlled section

ε_s = 0.003 ((d-c) / c)

= 0.0082 > is greater than required strain limit in steel. Therefore, section is tension controlled.

III. Moment capacity of actual section

$$Mn = As\ fy \left\{ \frac{d - \beta_1 c}{2} \right\}$$

$$= 6.24 \times 60 \times \left(27.3'' - \frac{0.8 \times 7.3''}{2} \right)$$

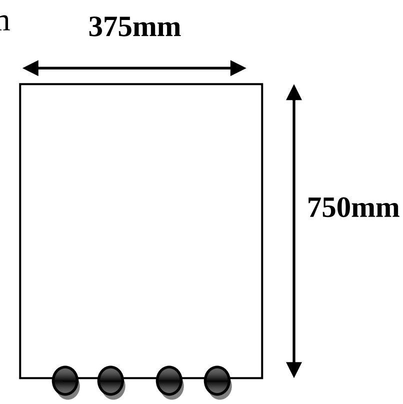
= 9128 k-in

Hence, Mn $\geq$ Mn, req

APPENDIX F2

Flexural design o singly reinforced beam to BS 8110-97

Considering the example of a simple singly reinforced beam



W_{LL} = 2.7kN/m
W_{DL} = 2.7kN/m→ including SW

L span = c/c spacing

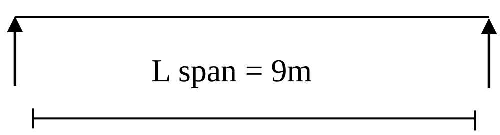
L span = 9m

W_{LL} = 2.7kN/m

W_{DL} = 2.7kN/m

f'_c = 35MPa

f_y = 415MPa



<u>Step1</u> - Ultimate design shear load

Wu = $1.4\ W_{DL} + 1.6\ W_{LL}$

= (1.4 x 2.7) + (1.6 x 2.7) = <u>8.1kN/m</u>

Wu = <u>8.1 kN/m</u>

Step 2 - Design moment at midspan

Considering a beam to be symmetrically loaded,

$$M = \frac{wl^2}{8}$$

$$= (8.1 \times 9^2) / 8$$

$$= 82.01\text{kNm}$$

Step 3 - Ultimate moment of resistance

$$Mu = 0.156 f_{cu} b d^2$$

$$= 0.156 \times 35 \times 375 \times 688^2 \times 10^{-6}$$

$$= 969.2\text{kNm}$$

Step 4 – Since Mu > M $\longrightarrow$ Singly reinforced beam

$$K = \frac{M}{f_{cu}\, bd^2}$$

$$K = 0.0132$$

Step 5 – Calculating d

$$d = h - \varphi/2 - d_{stirrup} - cover$$

$$= 750 - 8 - 4 - 50$$

$$= 688\text{mm.}$$

Step 6 – The maximum value of z = 653mm

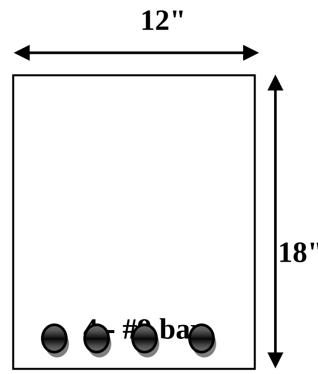
Step 7–As, req $= \dfrac{M}{0.87\, fy\, z} = \dfrac{82.01 \times 10^6}{0.87 \times 415 \times 653} = 400\ mm^2$

Provide 4T16 bars, As, prov = $804mm^2$

APPENDIX G1

Shear design of singly reinforced beam to ACI 318-14

Considering the example of a simple singly reinforced beam



$W_{LL} = 2.0$ k/ft
$W_{DL} = 4.0$ k/ft → including SW

L span = c/c spacing

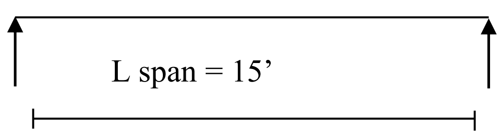
L span = 15'

W_{LL} = 2.0 kips/ft

W_{DL} = 4.0 kips/ft

f'_c = 4000 psi

f_y = 60,000 psi



<u>Step1</u> - Factored shear demand

Wu = 1.2 W_{DL} + 1.6 W_{LL}

= 1.2 (4) + 1.6 (2) = <u>8kips/ft</u>

Wu = <u>8kips/ft</u>

For a SSB, shear @ supports = reactions

(i) SUPPORT

Shear @ end of Beam,

$$\boxed{Vu_{,max} = \frac{Wu.l}{2}}$$

$$= \frac{8\ \text{kips/ft} \times 15'}{2} = 60\ \text{kips}$$

Shear @ ends of Beams

(ii) @CRITICAL SECTION

$$\boxed{Vu_{cr} = Vu_{max} - Wu \left(\frac{d + \frac{\text{width of supp}}{2}}{12''/\text{ft}} \right)}$$

$$= 60 - 8 \left(\frac{18'' + 3''}{12''/\text{ft}} \right)$$

$= \underline{46\ \text{kips}}$

<u>**Step 2**</u> - Required shear strength along beam and @ critical section

$$\boxed{Vn_{,req} = \frac{Vu}{\phi}}$$

0.75 for shear

$Vn,_{req\ max} = \dfrac{60\ kips}{0.75} = \underline{80\ kips}$

$Vn,_{reqcr} = \dfrac{46\ kips}{0.75} = \underline{61.3\ kips}$

Step 3 - Calculate concrete shear strength

$$\boxed{Vc = 2\sqrt{f'c}b_w d}$$

Simplified eq in ACI for concrete shear strength (Practical design consideration)

$Vc = 2\sqrt{4000} \times 12'' \times 18''$

$= \dfrac{27322.07\ psi}{1000\ lb/kips}$

$= \underline{27.3\ kips}$

WKT, $V_N = V_C + V_S$

Step 4 - Compare Vc with Vn req @ critical section to design steel

@ critical section ,

$Vc = 27.3$ kips

$Vn,_{req} = 61.3$ kips

$\therefore\ Vn_{req} > V_C$ ==== > Need steel

$$\boxed{Vs_{req} = Vn,_{req} - V_C}$$

$= 61.3 – 27.3 = \underline{34.0\ kips}$

Assume # 3 bar

2 legs

$Av = 2(0.11m^2) = \underline{0.22\ m^2}$

$$\boxed{S = \dfrac{Av.fy.d}{Vs,_{req}}}$$ ==== > max spacing

$= \dfrac{0.22 \times 60\ ksi \times 18''}{54.0\ kips}$

Max allow spacing = <u>6.99"</u>

Use # 3 bar @ 6" spacing c/c.

<u>Step 5</u> - Check ACI limits

- ACI max spacing : S_{max} = min $\begin{cases} d/2 \\ \quad \text{or} \\ 24" \end{cases}$

When $V_s < 4\sqrt{f'_c} b_w d$

∴ S_{max} = <u>9"</u>

Given spacing of 6" < 9"	O.K.

- ACI Min steel : $A_{v\,min}$ = max $\begin{cases} \dfrac{50\, b_w\, s}{f_y} \\[2ex] \dfrac{0.75\sqrt{f'_c}\, b_w\, s}{f_y} \end{cases}$

$$= \frac{50 \times 12" \times 6"}{60{,}000 \text{ psi}} = 0.060 \text{ m}^2$$

$$= \frac{0.75 \times \sqrt{4000} \times 12" \times 6"}{60{,}000 \text{ psi}} = 0.057 \text{ m}^2$$

∴ $A_{v\,min}$ = <u>0.06 m^2</u>

Provided Av	=	0.22 m^2	>	$A_{v\,min}$ O.K.

ACI Output review:

i. $V_C = 2\sqrt{f'c}\ bwd$

ii. $V_S = \dfrac{Av\ fyd}{s}$

iii. $V_N = V_C + V_S$

iv. $\phi V_N = V_U$

i. $\dfrac{2 \times \sqrt{4000 \text{ psi}} \times 12'' \times 18''}{1000 \text{ lb/k}} = 27.3 \text{ kips}$

ii. $\dfrac{0.22^2 \times 60 \text{ ksi} \times 18''}{6''} = 39.6 \text{ kips}$

iii. $V_N = 27.3 + 39.6 = 66.9 \text{ kips}$

iv. $\phi V_N = 0.75 \times 66.9 = 50.2 \text{ kips}$

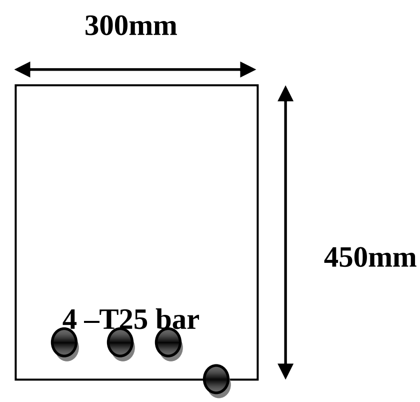
V_U @ crit $= 46$ kips <u>27 links Total</u>

$\boxed{\phi V_N > V_U \quad \text{O.K.}}$

APPENDIX G2

Shear design of singly reinforced section to BS 8110-97

Considering the example of a simple singly reinforced beam



$W_{LL} = 2.7$kN/m
$W_{DL} = 5.4$kN/m→ including SW

L span = c/c spacing

L span = 4.5m

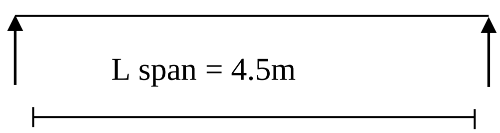
W_{LL} = 2.7kN/m

W_{DL} = 5.4kN/m

f'_c = 25MPa

f_y = 415MPa

As provided = 1964mm^2 4T25 bars



<u>Step1</u> - Ultimate design shear force

Wu = 1.4 W_{DL} + 1.6 W_{LL}

= (1.4 x 5.4) + (1.6 x 2.7) = <u>11.88kN/m</u>

Wu = <u>11.88kN/m</u>

Step 2 - Design shear stress υ

Considering a beam to be symmetrically loaded,

For a Simply Supported Beam , shear @ supports = reactions

(i) SUPPORT

Shear @ end of Beam,

$$\boxed{V_{u,@support} = \frac{W_u.l}{2}}$$

$$= \frac{11.88 \times 4.5}{2} = 26.73 \text{ kN}$$

Shear @ ends of Beams

$$\text{Therefore, } \upsilon = \left[\frac{V}{bd}\right]$$

$$= 26.73 \times \left[\frac{10^3}{300 \times 450}\right]$$

$$= 0.198 \text{ N/mm}^2$$

$\upsilon < 0.8\sqrt{fcu} \longrightarrow$ To limit diagonal compression failure

Step 3 - Design Concrete Shear stress

$$\boxed{\upsilon_c = \frac{100A_s}{bd}}$$

$$= \frac{100 \times 1964}{300 \times 450} = \underline{1.45}$$

Therefore, according to Table 3.8 BS 8110-97

$$\upsilon_c \quad = \quad \underline{0.7\ N/mm^2}$$

From Table 3.7, $\upsilon < \upsilon_c$

No shear reinforcement required. But still we provide minimum reinforcement.

Maximum spacing = 0.75d

Bs 8110-97 equation also comes from,

$$\boxed{V \quad < \quad V_{concrete} + V_{link}}$$

Nominal reinforcement $\longrightarrow$

$$\frac{Asv}{Sv} = \frac{0.4bv}{0.87fyv}$$

Let Sv = Sv max = 0.75 x 450 = 337.5mm

Therefore, Sv = 300mm

$$= \frac{0.4 \times 300 \times 300}{0.87 \times 415} = \underline{0.332}$$

Let links = φ 8 bars @ spacing 300mm c/c

$$\frac{Asv}{300} = 0.332$$

$Asv = 99.6mm^2$

Use 2T8 bars @ 300mm c/c

APPENDIX H1

Flexural design of one-way slab to ACI 318-14

Considering the example of a simply supported one-way slab

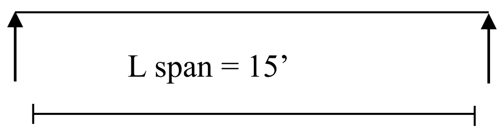
L span = c/c spacing

L span = 15’

W_{LL} = 100 lbs/ft^2

f'_c = 4000 psi

f_y = 60000 psi



Step 1 - Thickness of slab

Minimum thickness requirements based on deflection limits from ACI - (ACI 318-14 Sec. 7.3.1.1)

$h_{min} = \frac{L}{20}$ (simply-supported)

$= \frac{15'' \times 12''/ft}{20}$

$= 9''$

Step 2 - Loads based on thickness

⇨ D.L. = 9”/ (12”/ft) x (0.15 k/ft^3)

W_D = 0.113 kips/ft^2

Considering a 1ft strip of slab,

W_D = 0.113 kips/ft^2 x 1ft

= 0.113 kips/ft

⇨ L.L = 100 lbs/ft^2

$= 0.1\ \text{kips/ft}^2 \times 1\text{ft}$
$= 0.1\text{kips/ft}$

$W_u = 1.2\ W_{DL} + 1.6\ W_{LL}$

$= 1.2\ (0.113) + 1.6\ (0.1) \qquad = \qquad 0.14 + 0.16$

$W_u = \underline{0.3\ \text{kips/ft}}$

<u>Step 3</u> - Compute Mu

$$Mu \text{ @ midspan} = \frac{wl^2}{8}$$

$$= \frac{0.3 \times (15')^2}{8}$$

$$= 8.44\text{kip-ft/ft}$$

Mu = 101.28 k-in/ft (converting to inches)

<u>Step 4</u> - Compute Mn , req

$$Mn,\ req = \frac{Mu}{\varphi}$$

where, φ = 0.9 for flexure for a tension-controlled section (as assumed)

$$Mn,\ req = \frac{101.28}{0.9} = 112.54\ \text{k-in/ft}$$

<u>Step 5</u> - Approximate value of d

Assume #5 bars

d = 30" – 0.75" – 0.5x(0.625")

= 7.94"

Step 6 - Main reinforcement

i. Minimum steel: Considering shrinkage & temperature effects

$= \quad \rho = \quad 0.0018$

$A_{s,\,min} = 0.0018 \times 12" \times 9" = 0.19 in^2/ft$

ii. Maximum steel: Considering a tension controlled section

$= \quad \rho_{TC} = 0.85\beta_1(f'c/fy)(3/8)$

Let β_1 = 0.85 for 4000psi concrete

$= 0.018$

$A_{s,\,TC} = 0.018 \times 12" \times 7.94"$

$= 1.7 in^2$

iii. Actual steel required: As per demand & loading

We know that, $M_n = A_s f_y (d - a/2)$

Assume, $(d - a/2) = 0.9d$

$M_n = A_s f_y (0.9d) = M_{n,\,req}$

Therefore, $A_{s,req} = \dfrac{Mn,req}{fy\ (0.9d)} = \dfrac{112.5}{60\ (0.9*7.94")}$

$= 0.26\ in^2/ft$

Therefore, $A_{s,\,req} = 0.26 in^2/ft$

Either use the design aids or with the help of knowledge of area of bars we calculate the number and size of bars

We consider #5bars , hence, As, prov= $0.31\ in^2$

iv. Maximum spacing = 18" (According to Sec 7.7.2.3)

Hence, Actual steel required > Minimum steel required

And, Actual steel required < Maximum steel allowed

OK

Using $A_{s,req}$ and $A_{s,\,bar}$ we find required spacing $\dfrac{A_{s,req}}{A_{s,bar}} = \dfrac{12"}{S_{req}}$

S_{req} = 14”

Therefore, let spacing of bars be = 14” which is less than maximum spacing of 18”.

<u>Step 7</u> – Determine steel area per strip

We find the steel area @ a spacing of 14”

$$As = \frac{As_{,bar}}{s}(12")$$

$$= (0.31/14)(12")$$

$$= 0.266 \text{ in}^2\text{/ft of strip}$$

ACI Checks

IV. Nominal moment capacity of actual section

2. $\beta_1 = 0.85 - 0.05(f'c - 4) \geq 0.65$

$= 0.85$

2. $c = \frac{As\, fy}{0.85\, f'c\, \beta 1 b}$

$$= \frac{0.266 \times 60}{0.85 \times 4 \times 0.85 \times 12"} = 0.46"$$

V. $Mn = As\, fy \left\{ \frac{d - \beta_1 c}{2} \right\}$

$$= 0.266 \times 60 \times \left(7.94" - \frac{0.85 \times 0.46"}{2} \right)$$

$= 124$ k-in / ft

Hence, $Mn \geq Mn, req$

C is used to determine if steel has yielded & if steel is in tension controlled section

ε_s = 0.003 ((d-c) / c)

= 0.049 > is greater than required strain limit in steel 0.005. Therefore, section is tension controlled. OK.

φMn = 0.9 x 124 kip-in/ft

Therefore, φMn = 112k-in/ft

We know that, Mu = 101 k-in/ft

φMn > Mu

SAFE $\longrightarrow$ O.K.

APPENDIX H2

Flexural design of one-way slab to BS 8110-97

Considering the example of a simply supported one-way slab

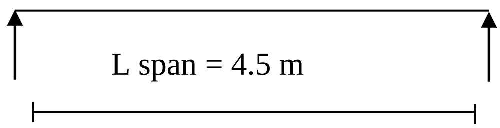
L span = c/c spacing

L span = 9m

W_{LL} = 5 kN/m^2

f'_c = 25MPa

f_y = 415MPa



Step 1 - Assuming a modification factor of 1.4

$$\text{Minimum effective depth, } d_{min} = \frac{\text{span}}{\text{Basic ratio x modification factor}}$$

$$d_{min} = \frac{4500}{20 \times 1.4} = 161\text{mm}$$

Hence, we assume a depth of slab (d) = 161mm

Assume dia of steel = 10mm

Let cover = 25mm

Therefore, overall depth of slab (h) = $d + \varphi/2 + c$

= 195mm

d = 165mm

h = 195mm

W_D = $0.195 \times 24kN/m^3$

= $4.68kN/m^2$

Step2 - Ultimate design load

Wu = $1.4\ W_{DL} + 1.6\ W_{LL}$

= $(1.4 \times 4.68) + (1.6 \times 5)$ = $\underline{14.6kN/m^2}$

Wu = $\underline{14.6kN/m^2}$

Therefore, considering a slab 1m width,

= $14.6kN/m^2 \times 1m$

= 14.6kN/m-m

Step 3 - Design moment at midspan

Considering a beam to be symmetrically loaded,

$$M = \frac{wl^2}{8}$$

= $(14.6 \times 4.5^2) / 8$

= 37 kNm

Step 4 - Ultimate moment of resistance

Mu = $0.156 f_{cu} bd^2$

= $0.156 \times 25 \times 1000 \times 165^2 \times 10^{-6}$

= 106.17 kNm / m

Since, M < Mu, no compression reinforcement required.

Step 5 – Main longitudinal steel, $\longrightarrow$

Since Mu > M Singly reinforced beam

$$K = \frac{M}{f_{cu}\ bd^2}$$

$$K = 0.054$$

$$Z = d\,[\,0.5 + \sqrt{(0.25 - \tfrac{K}{0.9})}\,]$$

$$= 165\,[\,0.5 + \sqrt{(0.25 - \left(\tfrac{0.054}{0.9}\right)}\,]$$

$$= 154.42mm < 0.95d$$

$$0.95d = 156.75mm$$

Hence, $z = 154.42mm$

Step 6 –As, req $= \dfrac{M}{0.87\,fy\,z} = \dfrac{37 \times 10^6}{0.87 \times 415 \times 154.42} = 663.64mm^2/m$

Provide T12 bars @150c/c / meter

Step 7 – Minimum area of reinforcement $= 0.0013 \times b \times h$

$$= 0.0013 \times 1000 \times 195$$

$$= 253.5\ mm^2/\ m$$

Therefore, $As,_{prov} > As,_{min}$

Hence design is OK