ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

COST EFFECTIVENESS OF COMPOSITE FRAMES IN CONDOMINIUM HOUSES.

BY BERGENE BASSA

MARCH, 2016

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A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF ADDIS ABABA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING.

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Declaration

I, the undersigned, declare that this thesis is my Original work and all sources of materials used for the thesis have been duly acknowledged.

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Date of submission: March, 2016

Acknowledgement

Thanks to God for each and every success in my life and satisfactory accomplishment of this thesis. My deepest gratitude goes to my thesis advisor Dr.Abrham Gebre for his professional, genuine guidance and valuable advice to accomplish the thesis on time.

I would like to extend my special appreciation to my families for their being with me in all ups and downs.

At last, but not least, I would like to express my profound heartfelt thanks to those people who have collaborated with me, especially my friends.

List of Symbols

Latin upper case letters

A_a = Cross-sectional area of structural steel

A_c = Cross-sectional area of concrete

A_g = Total cross-sectional area

A_s = Cross-sectional area of reinforcement

A_v = The shear area

A'_s = Area of reinforcements within the region of the steel web.

C_1 = Coefficient as a function of moment distribution

E_a = Design value of modulus of elasticity of structural steel

E_{cd} = Design value of modulus of elasticity of concrete

E_{cm} = Secant modulus of elasticity of concrete

EI =Flexural rigidity (per unit width for slabs)

E_s = Design value of modulus of elasticity of reinforcing steel

F_a = Tensile force in the structural steel necessary to resist the design Hogging bending Moment M calculated from plastic theory.

F_s = Tensile force in the Reinforcement necessary to resist the design Hogging bending Moment M calculated from plastic theory

F_c = Compressive force in the concrete flange necessary to resist the design sagging bending

I=Second moment of area of the total cross-section

I_c , I_s , I_a = second moments of area for the considered bending plane of the structural steel, the concrete (assumed to be un-cracked) and the reinforcement, respectively.

$M_{b,Rd}$ = The design buckling resistance moment of a laterally unrestrained beam

$M_{pl,Rd}$ = Plastic Moment Resistance of the Steel Section

M_{sd} = Design bending moment

N_{cr} = Critical Buckling load

$N_{pl,Rd}$ = Plastic resistance to compression

N_{sd} = Design value of the applied axial force

$P_{,Rd}$ = Design Resistance of Shear Connector Moment M calculated from plastic theory.

W_{pa} = Plastic section modulus for the structural steel

w_{pc} = Plastic section modulus for the concrete

W_{ps} = Plastic section modulus of the total reinforcement

$W_{pan}, W_{pcn}, W_{psn}$ = Plastic section moduli for the structural steel, the reinforcement and the concrete parts of the section respectively.

$V_{pl,Rd}$ = Plastic shear resistance

V_{sd} = Design Shear force

z_{cw} = Depth of Tension Zone

Latin lower case letters

a = Location of reinforcement from top surface of upper flange

b = Width of flange of the steel member

b_c =Total Cross section width of concrete

b_{eff} = Effective width of concrete slab

c_x = Minimum cover in x-direction for incased composite column

c_y = Minimum cover in y-direction for incased composite column

e =Eccentricity of the loading

f_{cd} = Design value of concrete compressive strength

f_{ck} = Characteristic compressive cylinder strength of concrete at the age of 28 days

f_{cm} =Mean compressive strength of concrete at the age of 28 days

f_y =Nominal tensile yield strength of structural steel, Yield strength 'of the structural steel

f_{yd} =Design yield strength of reinforcement

f_{yk} = Characteristic yield strength of reinforcement

f_{sk} = Characteristic tensile yield strength of reinforcement

h = Depth of web of the steel member

h_c = Total Cross section depth of concrete

h_n = Depth of the neutral axis in the web

i = Radius of gyration

l_o = Effective length

r=Radius

V_l =Total design longitudinal shear

Greek lower case letters

β_w= constant which is equal to 1.0 for plastic section

γ_a= Partial safety factor for structural steel

γ_c= Partial safety factor for concrete

γ_m=Partial safety factor for materials

γ_s= Partial safety factor for reinforcing steel

λ= Slenderness ratio

μ_{LT}= Factor for lateral-torsional buckling

ϕ= Diameter of the reinforcing bar

χ_{LT}= Reduction factor for lateral torsional buckling.

List of abbreviation

DD= Design dead Load

DL=Design Live Load

EBCS= Ethiopian Building Code Standard

Kg= Kilogram

L= Span length of a beam

mm = Millimeter

NAP= Position of Neutral axis

PNA=Plastic Neutral Axis

RC=Reinforced Concrete Structure

RCF=Reinforced concrete frame

S= Spacing of reinforcing bar

t_f = Thickness of flange

t_w = Thickness of web of the steel member

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Abstract

Steel-concrete composite construction has gained wide acceptance worldwide as an alternative to pure steel and pure concrete construction. However, this system is a relatively new concept for the construction industry in Ethiopia.

Reinforced concrete members are used in the framing system for most of the buildings since this is the most convenient & economic system for low-rise as well as short beam span when columns are closely spaced in buildings frame.

However, for medium to wide-spaced column in the buildings frame this type of structure is no longer economic because of increased dead load, less stiffness, span restriction and hazardous formwork.

Steel-concrete composite frame system can provide an effective and economic solution to most of these problems in medium to wide spaced column buildings.

An attempt has been made in this study to explore the cost effectiveness of steel-concrete composite construction by taking condominium houses currently undergoing in our country for maximum column spacing above which the steel-concrete composite frame structure is cost effective.

A cost versus frequently occurred maximum column spacing variation curves along both axes shows that for closely spaced column building frames structures, the conventional reinforced concrete frame system is cheaper than its equivalent steel-concrete composite system.

However, this curves show for building frames with column spacing between 9 to 10m and greater than this span along both axes, steel-concrete composite construction has economical advantages than reinforced concrete construction and 9 to 12 and greater than this storeys, again, steel-concrete composite construction has economical advantages than reinforced concrete construction in our country.

1. Introduction

1.1. Background

Numerous different structural systems are used today to meet performance or functional requirements in structures. One of these structural systems is steel-concrete composite construction. It is widely used in structural systems to achieve long spans, lower story heights, and provide additional lateral stiffness.

Composite construction uses the structural and constructional advantages of both concrete and steel. Concrete has low material costs, good fire resistance, and is easy to place. Steel has high ductility and high strength-to-weight and stiffness-to-weight ratios.

When properly combined steel and concrete can produce synergetic savings in initial and life-cycle costs. Currently, composite floor systems are widely utilized in steel buildings in the form of composite beams and joists/trusses **(Dong Keon Kim, 2005).**

Steel-concrete composite systems for buildings are formed by connecting the structural steel beam to the concrete slab or profiled deck slab with the help of mechanical shear connectors so that they act as a single unit **(Panchal & P. M. Marathe, 2011).**

Composite steel-concrete construction uses steel and concrete together to obtain a system with better performance, and/or lower cost, than using either material alone. A steel-concrete composite structural member contains both structural steel and concrete elements which work together **(Panchal & P. M. Marathe, 2011).** They have gained a wide acceptance worldwide as an alternative to pure steel and pure concrete construction. Pure steel as well as pure concrete has its own advantage and disadvantage. For example; Steel construction has many value benefits and advantage over traditional reinforced building construction. Steel in composite structures are used as permanent formworks in some parts of the structures, for example; in composite slab construction. Steel frame construction offers many advantages over traditional reinforced concrete with lower costs, sustainability and flexibility being amongst the many benefits of choosing steel framed buildings over the conventional reinforced concrete building construction. However, this system is venerable to fire and other sound insulation problems. To alleviate the above mentioned problems, it is good if the steel structures are covered with reinforced concrete (i.e. steel-concrete composite).But this is relatively new concept for construction industry in Ethiopia.

To bring this technology into practice, it requires a lot of works, giving awareness's to the different sectors of the society and conducting different researches around it.

1.2. Research Problem

At present day the costs of condominium houses are increasing at alarming rate especially that of the cost of concrete is currently very high as compared to its cost that was even a year ago. So, as known that the composite structures are light weighted and their thicknesses are low; it reduces the consumption of concrete material and accelerates the construction period, since it requires less formwork as compared to normal reinforced concrete structures.

Use of composite or hybrid material is of particular interest, due to its significant potential in improving the overall performance through rather modest changes in manufacturing and constructional technologies.

Thus, it is important to make a cost comparative study of steel-concrete composite structures with its equivalent normal reinforced concrete structure.

1.3. Scope of the Research

The scope of the research is limited to the followings:-

- Conducting structural analysis and modeling composite frames /structures with different column spacing.
- Reviewing Structural analysis of a reinforced concrete condominium structures currently used by the Addis Ababa Housing agency.
- Cost comparison of particular condominium buildings currently being constructed with that of composite structures.

1.4. Objectives of the Research

The objectives of this research are:-

- To obtain the cost effective spacing of columns/span of beams (minimum spacing of column in which composite structures is economical) below which the construction of normal reinforced concrete condominium building and above which the composite condominium building will be effective.
- To compare normal reinforced concrete condominium building frame with that of its equivalent composite condominium buildings on the bases of construction cost of materials.
- To obtain the demarcation number of storeys above which the steel-concrete composite frame will be cost effective.

1.5. Significance of the Study

A study on cost effectiveness of the composite frames and its comparison with normal reinforced concrete frames structures is very important for the following reasons:-

- It may solve the existing high cost problem associated with the current condominium building in Addis Ababa by introducing the construction of composite structures in our country with effective cost.
- It may help us to practice the use of light weighted structures that minimizes the consumption of the concrete and reinforcement as well as construction time.
- It may also enable to advertize the use and advantage of composite structures to the contractors and stockholder of the condominium buildings by exploring its cost effectiveness's.

1.6. Methodology

This study will primarily employ quantitative research methods (i.e. computer based structural analysis and use of EBCSs(Ethiopian Building Code Standards)) since the research objectives are aimed to obtain the economical/ cost effective spacing of columns and the height/Stories in which the composite condominium building will be cost effective.

The summarized methods for this research are as follows:-

- Reviewing different literatures.
- Reviewing structural analysis already done at Addis Ababa housing development agency for reinforced concrete condominium buildings and quantifying its component materials for frames of the building.
- Doing structural analysis and design of steel-concrete composite frames with different column spacing and quantifying its frames.
- Comparing the results of cost of the conventional reinforced concrete condominium building frames and steel-concrete composite condominium building frames.
- Selecting the demarcation point /economical column arrangement and spacing based on current cost analysis by considering their variation and;
- Selecting the demarcation storey above which the steel-concrete composite frame structure will be cost effective based on current cost analysis by considering their variation.

1.7. Content of the thesis

The thesis is consists of five chapters. Chapter one introduces the background, Research Problem, Scope of the research, Objectives of the research, Significance of the study and the methodology followed of the thesis work.

Chapter Two describes previous works done on the comparison of the conventional reinforced concrete frame with that of its equivalent steel-concrete composite frames/Literature Review/.

The third chapter focuses on analysis and design consideration and design example of composite frames with specific attention on loadings, material properties and design assumptions made. Moreover, it also addresses the design specification to be considered during analysis and design of composite frames as per the standards stated in the code. The core and specific input is presented under this chapter, indicating all the necessary steps and calculations used in designing the composite frames.

Chapter Four describes obtained design outputs/results and discussions which correlate the findings of this thesis with the stated economical demarcation span of beam/optimum column spacing above which steel-concrete composite frame will be cost effective/ in different literatures and codes.

The last chapter of this thesis is made to contain the conclusions drawn from the outputs based on the prevailing current material price, give the economical demarcation span of beam /optimum column spacing/ above which steel-concrete composite frame will be economical and the recommendations on the basis of the findings.

2. Literature Review

A number of studies dealing with what is steel-concrete composite structures, what is the economical span for steel-concrete composite beams as well as economical demarcation span for different types of beams and cost comparison of reinforced concrete structures with that of the steel-concrete composite structures was made in different countries of the world at different period of the time which are important for this study. A short review of different literatures and journals which will facilitate and give a clue for this study are presented below.

2.1.Background History of steel-concrete composite structures

The first study of steel-concrete composite members in building began as early as 1908 at Columbia University **(Viest et al. 1997).** The economical span for a uniform reinforced concrete slab is little more than that at which its thickness becomes sufficient to resist the point loads to which it may be subjected or, in buildings, to provide the sound insulation required. For spans of more than a few meters, it is cheaper to support the slab on beams, ribs or walls than to thicken it. Where the beams or ribs are also of concrete, the monolithic nature of the construction makes it possible for a substantial breadth of slab to act as the top flange of the beam that supports it. It used to be customary to design the steelwork to carry the whole weight of the concrete slab and its loading; but by about 1950 the development of shear connectors had made it practicable to connect the slab to the beam, and so to obtain the T-beam action that had long been used in concrete construction. The term composite beam refers to this type of structure **(R.P.Johnson, 2004).** Steel-Concrete Composite Column

Steel-concrete composite column is a compression member, comprising either a concrete encased hot-rolled steel section or a concrete filled tubular section of hot-rolled steel and is generally used as a load-bearing member in a composite framed structure. There are two basic kinds of composite columns: steel sections encased in concrete (steel-reinforced concrete sections or SRC) and steel sections filled with concrete (concrete filled tubes or CFT). The latter can be either circular (CCFT) or square/rectangular (RCFT) in cross-section. In composite columns additional synergies between concrete and steel are possible: in concrete-filled tubes, the steel increases the strength of the concrete because of its confining effect, the concrete inhibits local buckling of the steel, and the concrete formwork can be omitted; and in encased sections, the concrete delays failure by local buckling and acts as fireproofing while the steel provides substantial residual gravity load-carrying capacity after the concrete fails **(Dong Keon Kim, 2005).**

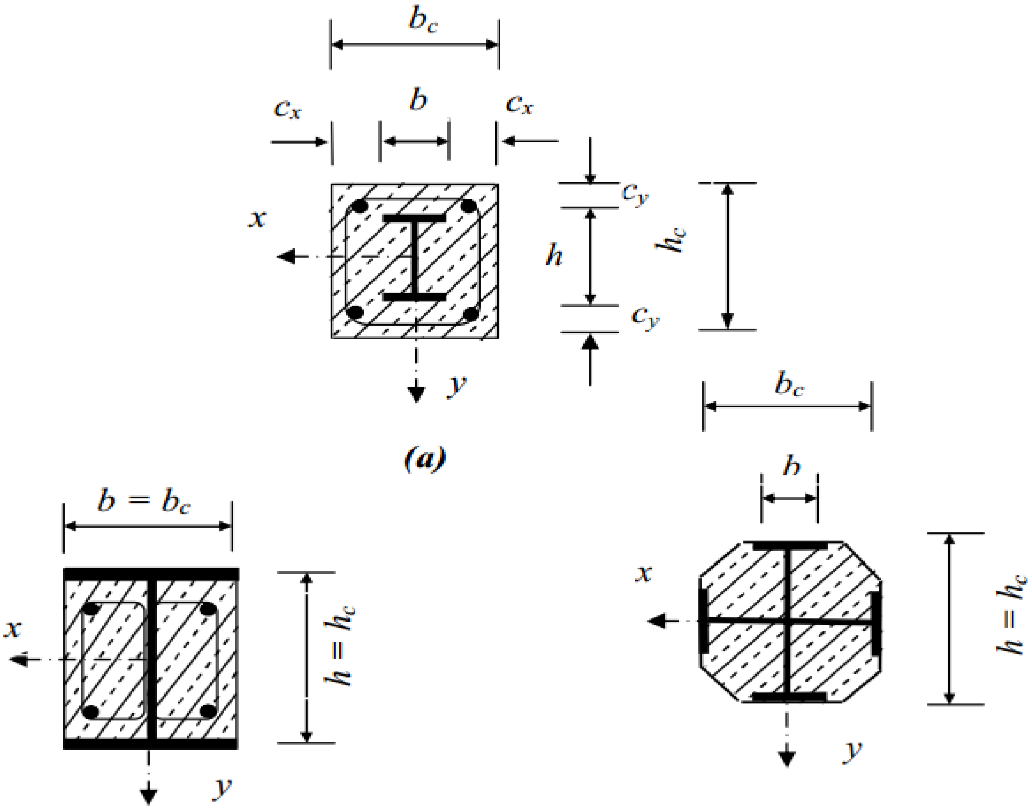


Fig. 1 Typical cross - sections of fully and partially concrete encased columns[**Dong Keon Kim, (2005)].**

Fig. 2: Concrete encased steel-concrete composite column during construction**[Roberto et al. (2013)]**

Note that there is no requirement to provide additional reinforcing steel for composite concrete filled tubular sections, except for requirements of fire resistance where appropriate.

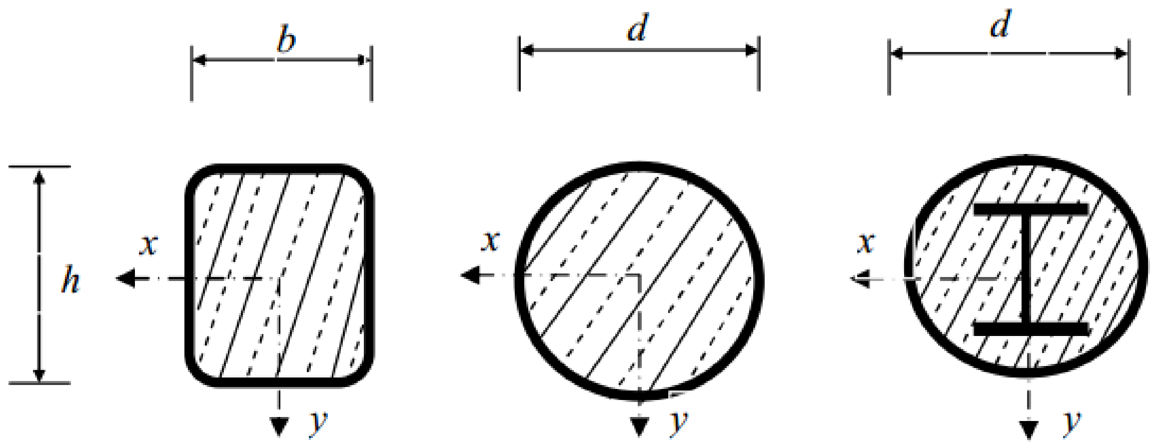


Fig. 3: typical cross-sections of concrete filled tubular sections**[Roberto et al. (2013)]**

Fig. 4: Concrete filled tubular steel-concrete composite column during construction **[Roberto et al. (2013)]**

In a composite column both the steel and concrete would resist the external loading by interacting together by bond and friction. Supplementary reinforcement in the concrete encasement prevents excessive spelling of concrete both under normal load and fire conditions.

In composite construction, the bare steel sections support the initial construction loads, including the weight of structure during construction. Concrete is later cast around the steel section, or filled inside the tubular sections. The concrete and steel are combined in such a fashion that the advantages of both the materials are utilized effectively in composite column. The lighter weight and higher strength of steel permit the use of smaller and lighter foundations. The subsequent concrete addition enables the building frame to easily limit the sway and lateral deflections.

With the use of composite columns along with composite decking and composite beams it is possible to erect high rise structures in an extremely efficient manner. There is quite a vertical spread of construction activity carried out simultaneously at any one time, with numerous trades working simultaneously. For example:

- One group of workers will be erecting the steel beams and columns for one or two storey at the top of frame.
- Two or three story's below, another group of workers will be fixing the metal decking for the floors.
- A few story's below, another group will be concreting the floors.
- As we go down the building, another group will be tying the column reinforcing bars in cages.
- Yet another group below them will be fixing the formwork, placing the concrete into the column moulds etc.

2.1.1. Steel-Concrete Composite Beam

In conventional composite construction, concrete slabs rest over steel beams and are supported by them. Under load these two components act independently and a relative slip occurs at the interface if there is no connection between them. With the help of a deliberate and appropriate connection provided between the beam and the concrete slab, the slip between them can be eliminated. In this case the steel beam and the slab act as a "composite beam" and their action is similar to that of a monolithic Tee beam. Though steel and concrete are the most commonly used materials for composite beams, other materials such as pre-stressed concrete and timber can also be used. Concrete is stronger in compression than in tension, and steel is susceptible to buckling in compression. By the composite action between the two, we can utilize their respective advantages to the fullest extent. Generally in steel-concrete composite beams, steel beams are integrally connected to prefabricated or cast in situ reinforced concrete slabs.

2.2. Advantage of composite frames

Patrous et al. (2007) summarized the following advantages of steel-concrete composite columns and beams.

- Increased strength for a given cross sectional dimension.
- Increased stiffness, leading to reduced slenderness and increased buckling resistance.
- Good fire resistance in the case of concrete encased columns.
- Corrosion protection in encased columns.
- Significant economic advantages over either pure structural steel or reinforced concrete alternatives.
- Identical cross sections with different load and moment resistances can be produced by varying steel thickness, the concrete strength and reinforcement. This allows the outer dimensions of a column to be held constant over a number of floors in a building, thus simplifying the construction and architectural detailing.
- Erection of high rise building in an extremely efficient manner.

Formwork is not required for concrete filled tubular sections.

- The most effective utilization of steel and concrete is achieved.
- Keeping the span and loading unaltered; a more economical steel section (in terms of depth and weight) is adequate in composite construction compared with conventional non-composite construction.
- As the depth of beam reduces, the construction depth reduces, resulting in enhanced headroom.
- Because of its larger stiffness, composite beams have less deflection than steel beams.
- Composite construction provides efficient arrangement to cover large column free space.
- Composite construction is amenable to "fast-track" construction because of using rolled steel and pre-fabricated components, rather than cast-in-situ concrete.
- Encased steel beam sections have improved fire resistance and corrosion**.**

Sijaria et al. (2014) mentioned in conventional composite construction, concrete slabs rest over steel beams and are supported by them. Under load, these two components act independently and a relative slip occurs at the interface if there is no connection between them. With the help of deliberate and appropriate connection provided between the beam and the concrete slab, the slip between them can be eliminated. In this case, the steel beam and the slab act as a "Composite beam" and their action is similar to that of a monolithic Tee beam. Since concrete is stronger in compression than in tension, and steel is acceptable to book ling in compression, by the composite action between the two, we can utilize their respective advantages to the fullest extent.

There are many advantages associated with steel-concrete composite construction.

Some of these are :

- The most effective utilization of steel and concrete is achieved.
- Keeping the span and loading unaltered, a more economical steel section (in terms of depth and weight) is achievable in composite construction compared with conventional non-composite construction.
- As the depth of beam reduces, the construction depth reduces, resulting in enhanced headroom.
- Because of its larger stiffness, composite beams have less deflection than steel beams.
- Composite construction is amenable to "fast-track" construction because of using rolled steel and prefabricated components, rather than case-in situ concrete.
- Encased steel beam sections have improved fire resistance and corrosion.
- Considerable flexibility in design, pre-fabrication and construction schedule in congested areas.

2.3. Economical span for steel-concrete composite structures.

The design of structures for buildings and bridges is mainly concerned with the provision and support of load-bearing horizontal surfaces. Except in long-span bridges, these floors or decks are usually made of reinforced concrete, for no other material has a better combination of low cost, high strength, and resistance to corrosion, abrasion, and fire. The economical span for a reinforced concrete slab is little more than that at which its thickness becomes just sufficient to resist the point loads to which it may be subjected or, in buildings, to provide the sound insulation required. For spans of more than a few metres it is cheaper to support the slab on beams or walls than to thicken it. When the beams are also of concrete, the monolithic nature of the construction makes it possible for a substantial breadth of slab to act as the top flange of the beam that supports it. At spans of more than about **10m**, and particularly where the susceptibility of steel to damage by fire is not a problem, as for example in bridges and multi-storey car parks, steel beams become cheaper than concrete beams **(R.P.JOHNSON,2004).**

Shweta A. Wagh et al.(2014) concluded that steel-concrete composite beam spans of 6 to 12 m can be created giving maximum flexibility and division of the internal space. Composite slabs use steel decking of 46 to 80 mm depth that can span 3 to 4.5 m without temporary propping. Slab thicknesses are normally in the range 100 mm to 250 mm for shallow decking, and in the range 280 mm to 320 mm for deep decking.

Column-free space has become an important design requirement in modern commercial buildings to achieve flexibility in use. Many long-span beam systems have been developed with spans of up to 18 m, which means that internal columns are not required for many building layouts **(Bouwen met Staal et.al 2008).**

Brian Uy and J.Y. Richard Liew (2003) pointed out that the economic advantage of fabricated beams that they can be designed to provide the required moment and shear resistance along the beam span in accordance with the loading pattern along the beam. They concluded that steel-concrete composite tapered beams are found to be economical for spans up to 20 m.

Liew and Richard Jat Yuen (2001) performed structural analysis and design for steel-concrete composite frames and stated that short to medium span (6m to 12m) composite floor beams perform quite well and are rarely found to transmit annoying vibrations to the occupants and particular care is required for long span beam more than 12m. They also stated that steel-concrete composite beams are highly efficient and economic with bay sizes in the range of 6m to 12m. They finally summarized different types of beams and their economic span as follows:

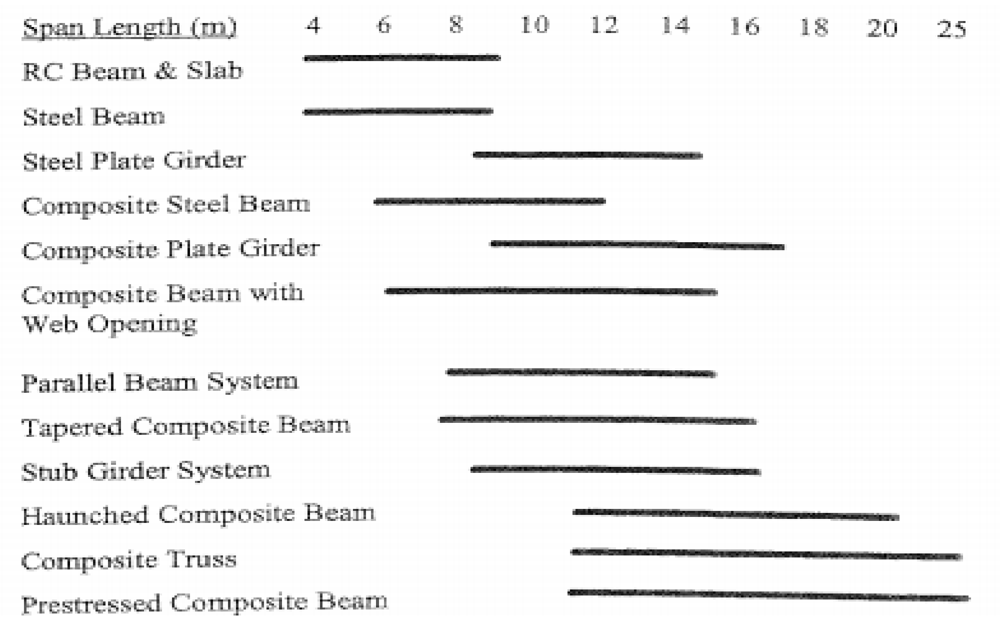


Table 1 Comparison of flooring Systems (Beams)**[Liew and Richard Jat Yuen (2001)]**

Erkan and AMHÂL (2005) performed analysis and design on steel-concrete composite structures and finally reached on the conclusion that composite floor construction is highly competitive if spans are increased to 12, 15 or even 20 m. There is, of course, a demand for larger column-free spans (**http://www.steel-insdag.org:80/TeachingMaterial**) in buildings to facilitate open planning or greater flexibility in building layout.

Sins (2009) reached a conclusion on long span steel-concrete composite beams that at spans up to 18m, economic, strength-governed solutions are possible. He also stated that for long span steel-concrete composite beams beyond 18m serviceability criteria will have an increasingly significant influence on design.

Sijaria et al. (2014) pointed out that the main economy in using profiled deck is achieved due to speed in construction. They notified that normally 2.5 to 4.0m spans can be handled without propping and spans in excess 4m will require propping.

They made cost comparison of G + 5 steel-concrete composite structure with that of an equivalent R.C.C. structure and these two structures has been analyzed, designed and cost per unit quantities worked out. They also concluded that Though, the cost comparison reveals that Steel-Concrete composite design structure is more costly, reduction in direct costs of steel composite structure resulting from speedy erection will make Steel-concrete Composite structure economically viable. They finally, showed that above 15 stories, steel-concrete composite structure is cost effective as shown below in their figure.

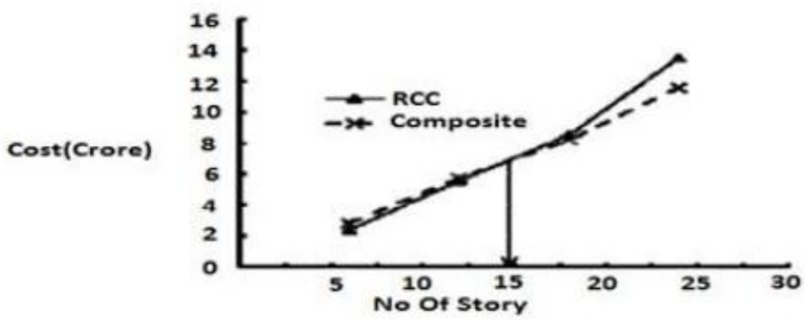


Fig. 5: Cost Versus number of storey curve for composite and RCC building **[Sijaria et al. (2014)]**

Sriramulu et.al (2003) stated that in an innovative composite beam design, the construction industry has been using steel sheets or decking that is deeply corrugated with a profile shaped like a trapezoid. This allows for large spans between beams in flooring systems, making it an economical design and reducing the amount of concrete needed. Simple construction of steel-concrete composite structure with rolled steel sections, for spans in the range of 6 to 10 m, perhaps the most appropriate and economical form of construction is rolled sections and simple, shear only connections. Secondary beams at 2.4 m or 3.0 m centers support lightweight composite floor slabs and span onto primary beams, which in turn frame directly into the columns. The same form of construction may also be used for longer span floors but beam weights and costs increase to the point where other forms of construction may be more attractive. Of increasing concern to developers is the provision of web openings as these are inflexible and they can create difficulties in meeting the specific needs of tenants or in subsequent reservicing during the life of the structure. But for fabricated sections are most likely to be economic for spans above 12 m. above this span length, rolled sections are increasingly heavy and a fine-tuned fabricated section is likely to be able to save on both flange size and web thickness **(Roland Bartschi, 2003).**

Begum et al. (2013) had objectives to provide a brief description to various components of steel concrete framing system for buildings and investigate the cost effectiveness of steel-concrete composite frames over traditional reinforced concrete frames for building structures. After analysis, design and cost comparison, they concluded that for medium to high rise buildings steel concrete composite frame system is a better choice over reinforced concrete frame system from both economy & serviceability point of view. For high rise buildings constructed with composite frames cost decreases due to the use of smaller cross sectional element, use of less steel, use of less formwork for concrete, low labor cost etc. Steel-concrete composite frame system can be an economically viable solution for high-rise buildings in Bangladesh.

Koppad and Itti (2013) tried to show the comparison of different parameters of the reinforced concrete and traditional steel-concrete composite structures. They concluded as follows: 1. the axial force in RC structure is on higher side of composite structure. 2. Composite structures are more economical than that of Reinforced concrete structure. 3. Weight of composite structure is quite low as compared to RC structure which helps in reducing the foundation cost. 4. Composite structures are the best solution for high rise structure as compared to RC structure. 5. Speedy construction facilitates quicker return on the invested capital and benefits in terms of rent. 6. The node displacement and deflection in composite structure is more compared to RC structure but the deflection is within permissible limit. 7. The maximum bending moment in composite beam is less compared to RC beam. 8. The maximum shear force in composite beam is less compared to RC beam. 9. The cost of composite beam is reduces by 27% compared to RC beam. 10. The cost of composite structure related RC column is reduces by 20.45% compare to RC column.

Shweta A. Et al (2014) took four various multi-storeyed commercial buildings i.e. G+12, G+16, G+20, G+24 and analysed by using STAAD-Pro software and made design and cost estimation by using MS-Excel programming and from obtained result they made comparison between the two structures. They have concluded that Composite action increases the load carrying capacity and stiffness by factors of around 2 and 3.5 respectively, In case of a composite structural system because of the lesser magnitude of the beam end forces and moments compared to an R.C. system, one can use lighter section in a composite structure. Thus, it reduces the self-weight and cost of the structural components. The downward reaction (F_Y) and bending moment in other two directions for composite structural system is less. Thus one can use smaller size foundation in case of composite construction compared to an R.C. construction. Under earthquake consideration because of inherent ductility characteristics, steel-concrete composite structures perform better than a R.C. structure. In the cost estimation for building structure no savings in the construction time for the erection of the composite structure is included. As compared to RC structures, composite structures require less construction time due to the quick erection of the steel frame and ease of formwork for concrete. Including the construction period as a function of total cost in the cost estimation will certainly result in increased economy for the composite structure. The cost comparison reveals that steel-concrete composite design structure is more economical in case of high rise buildings and construction is speedy.

Salunke et al (2013) made comparative study on analysis and design of steel-concrete composite structure with that of the ordinary reinforced concrete structures. They investigated four different storied (G+6, G+7, G+8, G+9, G+10) buildings and studied the Effect of each building with respect following parameters: Time period, base shear, total dead load and Most important cost of different schemes. They finally concluded that time period of building is decreased by 4% than normal R.C. building and 22 % increased than steel building, Completion period of the building came down by 21% when compared to R.C. Structure and 23 % up when compared to STEEL building, Dead Load of building is decreased by 23 % than normal R.C. building and 3 % increased than STEEL building, Base Shear of building is decreased by 13 % than normal R.C. building and 3 % increased than STEEL building, As per this work the total cost of structure for composite is increased by 55% than normal R.C. building and 44% decreased than steel building

Some study was made even on medium rise buildings; like a study by **Tedia1 and Maru (2013)** who took G+5 residential buildings and found that composite structure is more economical than traditional reinforced structures. Similar study by **Shah and Pajgade (2013)** showed that composite structure is more economical than traditional reinforced structures.

Comparative study of G+30 storied commercial building which is situated by **(Panchal and Marathe, 2011).** Their result is the same as the previous study. A study by **Dabhade et al**

(2009) showed that steel frame with composite deck floor saves 55.3% construction time than precast frame with precast concrete floor and 14.3% compared to steel frame with precast concrete floor. However, this required extra 23.10% of direct cost and 12.99% of net cost for precast frame with precast concrete floor while 0.52% and -2.34% for steel frame with precast concrete floor. Similar methodology was followed by **Tedia1 and Maru (2013)** with that of **Koppad and Itti (2013)** except the building type .i.e. G+5 (rigid joint regular frame).

Shahand Pajgade (2013) studied by taking the Office building having G+15 storied located in seismic zone 4 & wind velocity 39 m/sand made analysis and design of both composite and reinforced structures by STAAD-pro software. The methodology followed by **Panchal and Marathe (2011)** is similar to the **Shahand Pajgade (2013)** study except the building height i.e. G+30.

2.4. Summary

As can be seen from the above review of different literatures and journals, the following points can be extracted:

- Almost all literatures focus on the comparison of different parameters including cost of the traditional reinforced concrete and steel-concrete composite structures and concluded that for medium to high rise building, steel-concrete composite structures are cost effective and for low rise buildings, reinforced concrete structures are cost effective.
- Most literatures focus on the commercial buildings.
- Most literatures use software like STAAD-pro and ETABS software for modeling and analysis of both reinforced concrete and steel-concrete structures.
- Most literatures focus on the speedy construction of steel-concrete composite structure.
- Most literatures focus on the column spacing/beam span of 6-12m is both cost and structural effective span for steel-concrete composite structure.

Although, the method they followed and conclusion they arrived are different, it is can be concluded that at medium beam span (6-12m) steel-concrete composite structure become effective both interims of cost and safety (structurally).

3. Analysis and Design of Steel-Concrete Composite Frame

3.1. Analysis of Steel-Concrete Composite Frame

In this study, a reinforced concrete moment resisting building frame (20.7 meters in width and 30 meters in length and 9 storey, one basement and one terrace floor), representing conventional (condominium building) types of buildings in a relatively low risk earthquake prone zone (Zone two) has been collected from Addis Ababa Saving Houses Development Enterprise. This frame has been first analyzed and designed by equivalent steel-concrete composite frame. Second, the arrangement of columns (column spacing) has been changed twice without altering the plan as well as the whole shape of the building. Thirdly, the height of the original building has been changed to different storey for the feasibility study of steel-concrete composite construction in Ethiopia. A sample floor and roof plans of the building is shown in Fig. 5, 6, 7 and 8. The design and analysis of the frame is conducted with three different heights of the building. During this change an attempt is made to make the frames' span width and length to be conforming to architectural norms and constructional practices of the conventional buildings in Ethiopia.

Analysis of steel-concrete Composite frame is conducted using ETABS 2015 version 15.0.0. For earthquake analysis, dynamic method of analysis is used.

A 9-storey (G+7 with terrace floor) of Reinforced Concrete building analyzed and designed for gravity and earthquake loads in Addis Ababa housing development Enterprise, Ethiopia is also studied. The rectangular plan of the building is 20.7 m by 30 m. The story height of the building is varying but the most frequently occurred story height is 3.23 m with a total height of 29.23 m. Sample structural system and its plan layout are shown in figure 6, 7 and 8.

The equivalent steel-concrete composite frame is designed in ten different models in order to achieve the objective of the research work. The 1st model is designated as model-1X which is with column spacing of 6m along x-direction. The 2nd model is designated as model-2X which is with column spacing of 7.5m along x-direction. The 3rd model is designated as model-3X with column spacing of 10m along x-direction. The 4th model is designated as Model-1Y which is with column spacing of 5m along y-direction. The 5th Model is designated as Model-2Y with column spacing of 7.5m along y-direction. The 6th model is designated as Model-3Y with column spacing of 10m along y-direction. The 7th model is designated as Model-1Z with number of storey of 7. The 8th model is designated as Model-2Z with number of storey of 9. The 9th model is designated as Model-3Z with number of storey of 12 and the last model is designated as Model-4Z with number of storey of 15.

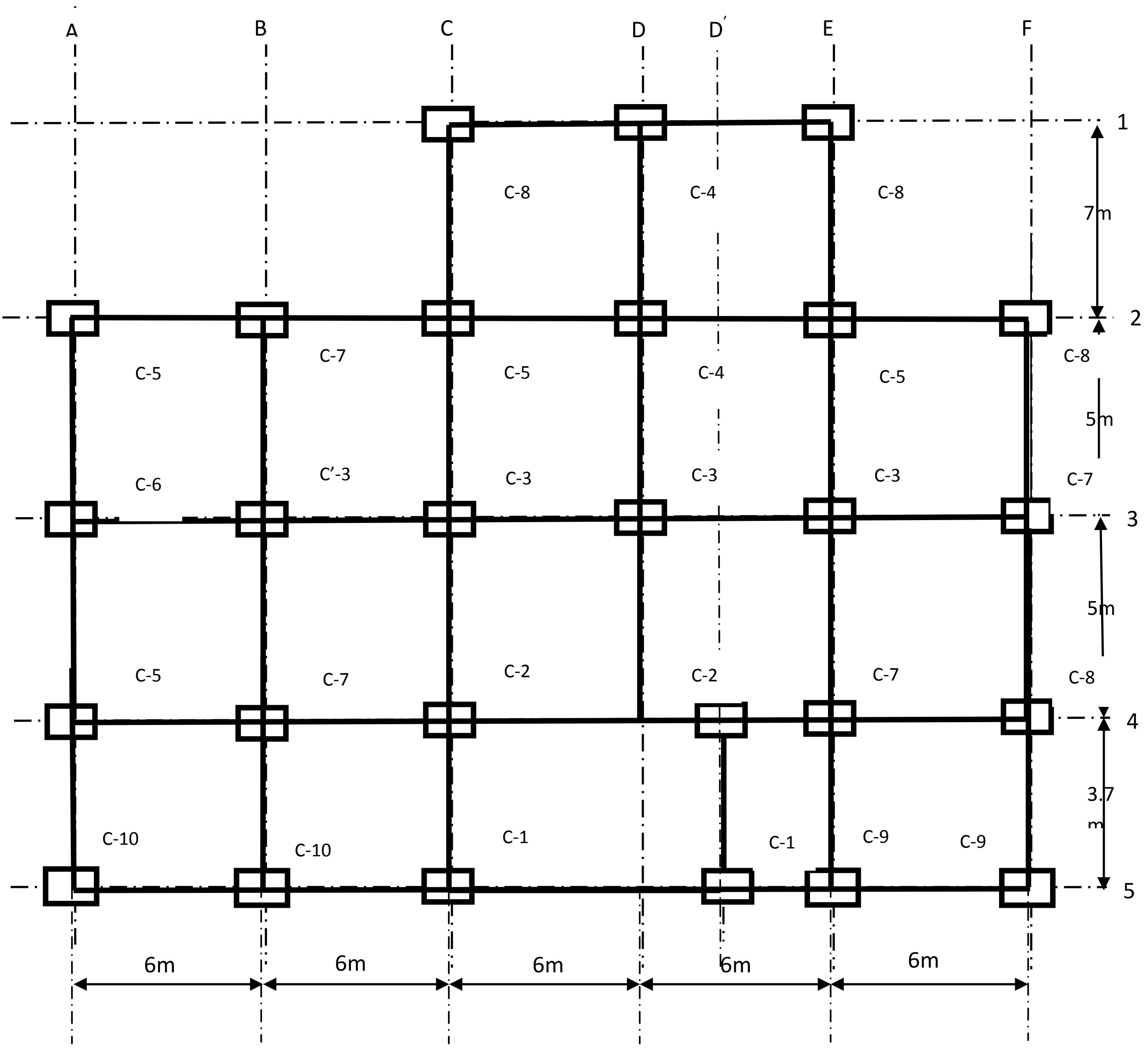


Fig. 6: Ground Floor Plan

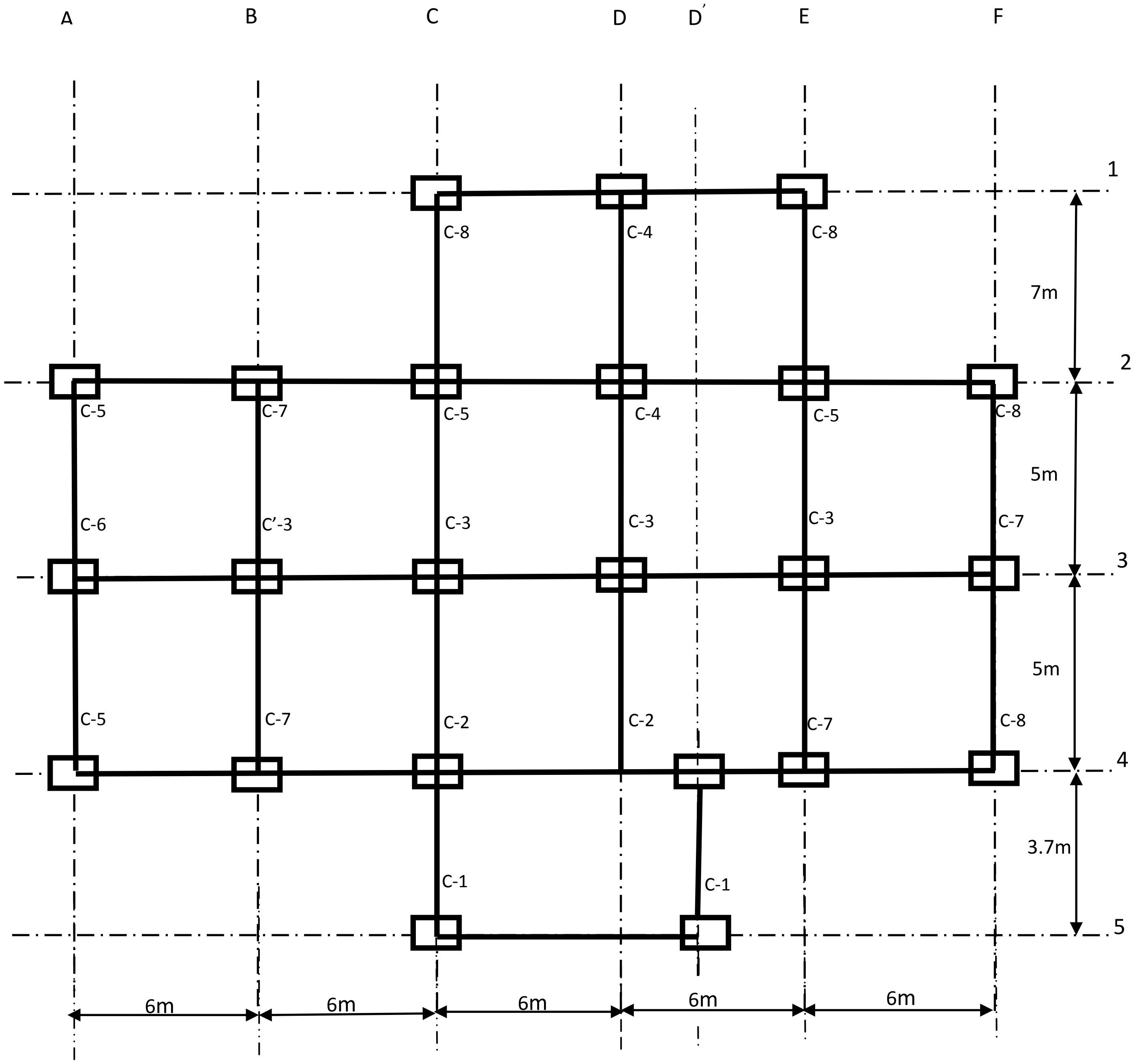


Fig. 7: Typical First to seventh Floor

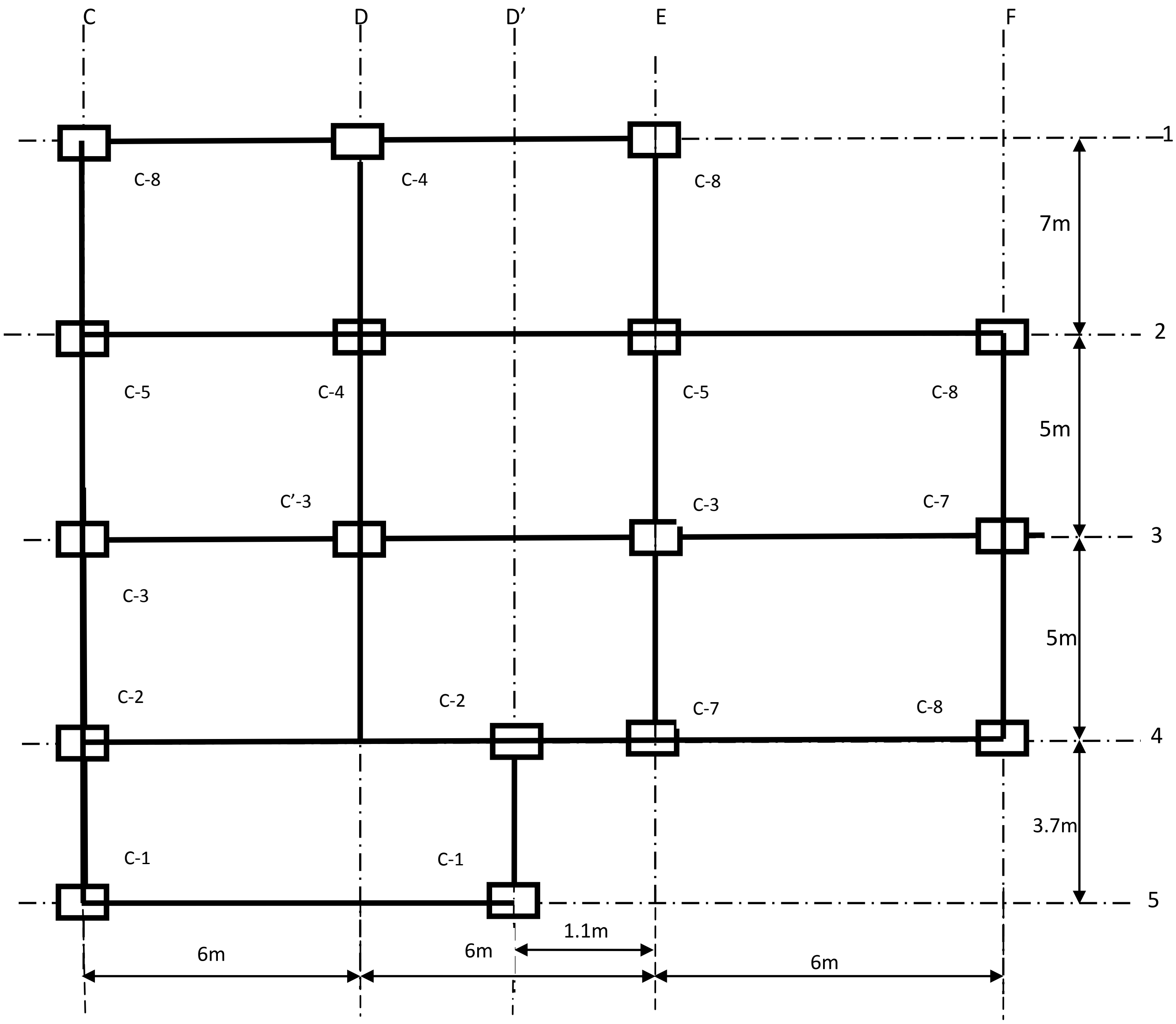


Fig. 8: Roof Plan

3.2. Design of steel-concrete composite frame

The design axial force and bending moments on both axis for column is taken from the software analysis result and the sample trial cross-section (for each model for column the terrace and ground column) is checked for different criteria as per EUROCODE-4, EN 1994-1-1:2004 (Plastic Resistance of the section, short term loading, long term loading, Resistance the composite column under axial compression for both major and minor axis, Resistance the composite column under axial compression plus biaxial bending for both axis, column resistance against combined compression and biaxial bending).

The optimum, economic and safe section for both main beam as well as secondary/infill beam is selected from Auto select section list by the software using EUROCOD-4 and the sample axis(X-axis) beam is manually checked these sections for different criteria as per EUROCODE-4, EN 1994-1-1:2004 for construction stage (plastic moment resistance of steel section, plastic shear resistance, Bending moment and vertical shear interaction, lateral torsional buckling of steel beam) and composite stage(moment resistance of the cross-section, vertical shear and bending moment and shear force interaction, shear buckling, shear connector resistance).

The analysis and design is conducted for ten models of composite frames. Since the most frequently occurred maximum column spacing along x-axis is 6m, Therefore, the 1st model along this axis is the original dimensions/model; the 2nd model is for 7.5m column spacing along this axis; the 3rd model is for 10m column spacing along this axis again. the most frequently occurred maximum column spacing along y-axis is 5m, therefore ; the 4th model is for column spacing of 5m along y-axis; the 5th model is for column spacing of 7.5m along this axis ; the 6th model is for the column spacing of 10m along again this axis ; the 7th model is frame with number of storey of 7; the 8th model is frame with number of storey of 9; the 9th model is with number of storey of 12 and the last 10th model is frame with number storey of 15. The detailed sample design is presented in this chapter and step by step design procedures in more detail are presented in Annexes for the selected sample trials/models.

3.2.1. Design Example of composite column

The column along axis 4C is with maximum axial force therefore, selected to be designed, since this column is assumed to carry maximum moment and axial force. The 4th floor column along axis 4C in model-1X is arbitrarily selected to clarify the design steps and procedures followed for steel-concrete composite column.

Length=3.03m

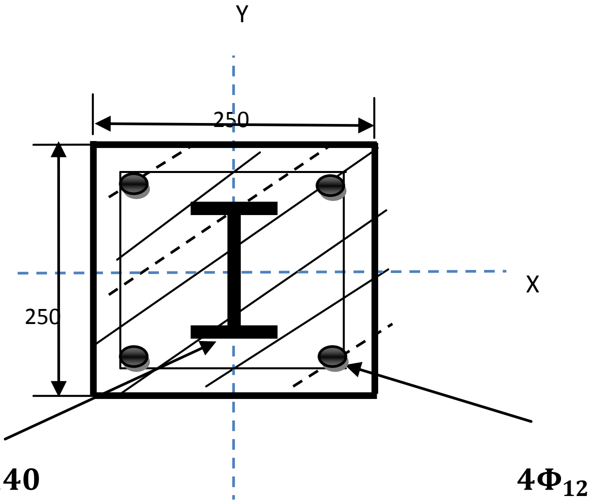


Fig. 9: Cross-Section Properties of Terrace Column for Model-1X

Step-1: Details of the section

Column dimension	250x250
Length	3030
Concrete grade	C-25
Structural steel section	IPE 140

Step-2: Load data

Design Axial Force$(N_{sd}) = 1199.722$ KN (from ETABS Analysis)

Design Bending moment about X-X axis $(M_{sd}, x) = -11.3KNm$ (from ETABS Analysis)

Design Bending moment about Y_Y axis $(M_{sd,}y) = 6.46KNm$(from ETABS Analysis)

Step-3: Column cross section data

$h_f = 2500mm$ $\quad A_a = 16.4 \times 10^2\ mm^2$

$b_f = 250mm$ $\quad I_{ax} = 541.2 \times 10^4 mm^4$

$b = 73mm$ $\quad I_{ay} = 44.92 \times 10^4 mm^4$

$h = 140mm.$ $\quad W_{px} = 88.34 \times 10^3 mm^3$

$t_f = 6.9\ mm$ $\quad W_{py} = 19.25 \times 10^3 mm^3$ $\quad t_w = 4.7\ mm$

$C_y = \frac{250-140}{2} = 55\text{mm}$ $\quad\quad$ $C_x = \frac{250-73}{2} = 88.5 \text{ mm}$

Reinforcing steel : $4\Phi_{12}; A_s = 452.4 \text{ mm}^2$

$A_c = A_{gross} - A_s = (250^2) - (4 \times 113.1) - 16.4 \times 10^2 = 60{,}407.6 \text{ mm}^2$

Step-4: Design checking

- **Plastic axial load capacity of cross-section**

$$N_{pl,Rd} = \frac{A_g f_y}{\gamma_s} + A_c\left(\frac{0.85 f_y}{\gamma_c}\right) + A_s\left(\frac{f_s}{\gamma_s}\right)$$

$$N_{pl,Rd} = \left(\frac{62{,}500 \times 355}{1.1} + \frac{0.85 \times 20 \times 60{,}407.6}{1.5} + \frac{452.4 * 300}{1.15}\right) \times 10^{-3} = 20{,}973.1\text{KN}$$

- **Effective elastic flexural stiffness of the section for short term loading**

About the major axis

$$(EI)_{ex} = E_a I_{ax} + 0.8 E_{cd} I_{cx} + E_s I_{sx}$$

$$E_{cd} = \frac{E_{cm}}{\gamma_c} = \frac{29000}{1.35} = 21481.48 \, {}^{\text{N}}\!/_{\text{mm}}$$

$$I_{sx} = A_s h^2 = 452.4 \times (\frac{250}{2} - 25 - 6)^2 = 399.74 \times 10^4 \text{mm}^4$$

$$I_{cx} = \frac{(250)^4}{12} - (399.74 \times 10^4 + 541.2 \times 10^4) = 31.6 \times 10^7 \text{mm}^4$$

$$(EI)_{ex} = (210{,}000 \times 541.2 \times 10^4) + (0.8 \times 21481.48 \times 31..6 \times 10^7) + (210{,}000 \times 399.74 \times 10^4) = 7.4 \times 10^{12} \text{Nmm}^2$$

About the minor axis

$$(EI)_{ey} = E_a I_{ay} + 0.8 E_{cd} I_{cy} + E_s I_{sy}$$

$$E_{cd} = \frac{E_{cm}}{\gamma_c} = \frac{29000}{1.35} = 21481.48 \, {}^{\text{N}}\!/_{\text{mm}}$$

$$I_{sy} = A_s h^2 = 452.4 * (\frac{250}{2} - 25 - 6)^2 = 399.74 \times 10^4 \text{mm}^4$$

$$I_{cy} = \frac{(250)^4}{12} - 10^4 \times (399.74 + 44.92) = 32.1 \times 10^7 \text{mm}^4$$

$$(EI)_{ey} = (210{,}000 \times 44.92 \times 10^4) + (0.8 \times 21481.48 \times 32.1 \times 10^7) + (210{,}000 \times 399.74 \times 10^7) = 8.45 \times 10^{14} \text{Nmm}^2$$

- **Related slenderness ratio**

$$\lambda = \left(\frac{N_{pl,R}}{N_{cr}}\right)^{\frac{1}{2}} \text{ with } N_{pl,R} = A_a f_y + A_c(0.85 f_{ck}) + A_s f_{yk} \text{ and } N_{cr} = \frac{\pi^2 EI}{L^2}$$

$$N_{pl,R} = [(16.4 \times 10^2 \times 355) + (0.85 \times 20 \times 60{,}407.6) + (452.4 \times 300)]10^{-3}$$

$$= 1{,}744.79 \text{ KN}$$

$$(N_{cr})_x = \frac{\pi^2 (EI)_{ex}}{l^2} = \frac{\pi^2 (7.4 \times 10^{12})}{(3030)^2} = 7{,}955.11\text{KN}$$

$$(N_{cr})_y = \frac{\pi^2 (EI)_{ey}}{l^2} = \frac{\pi^2 (8.45 \times 10^{14})}{(3030)^2} = 908{,}387.6\text{KN}$$

$$\lambda_x = \left(\frac{N_{pl,R}}{(N_{cr})_x}\right)^{\frac{1}{2}} = \left(\frac{1{,}744.79}{7{,}955.11}\right)^{\frac{1}{2}} = 0.47$$

$$\lambda_y = \left(\frac{N_{pl,R}}{(N_{cr})_y}\right)^{\frac{1}{2}} = \left(\frac{1{,}744.79}{908{,}387.6}\right)^{\frac{1}{2}} = 0.05$$

- **Check for the effect of long term loading**

The effect of long term loading can be neglected if anyone or both the following conditions are satisfied:

Eccentricity, e given by:

$e = \frac{M_{sd}}{N_{sd}} \geq 2$ times the cross section dimension in the plane of bending considerede

$$e_x = \frac{M_{sd,x}}{N_{sd}} = \frac{-11.29}{1199.722} = 0.094 < 2(0.250) = 0.5$$

$$e_y = \frac{M_{sd,y}}{N_{sd}} = \frac{6.46}{1{,}199.722} = 0.054 < 2(0.25) = 0.5$$

$$\lambda_{max} < 0.8\,;\text{i.e } 0.47 < 0.8$$

Since both conditions are satisfied, the influence of creep and shrinkage on the ultimate load need not be considered.

- **Resistance of the composite column under axial compression**

Design against axial compression is satisfied if the following condition is satisfied:

$N_{sd} < \chi_x N_{pl,Rd}$

$N_{sd} = 1{,}199.722$ KN and $N_{pl,Rd} = 20{,}973.1$KN

About major axis

Imperfection factor (α_x) =0.34 (buckling curve "b" for Encased profiles buckling about the strong axis.

$$\chi_x = \frac{1}{\phi_x + (\phi x^2 - \lambda_x^2)^{\frac{1}{2}}} \quad \text{and}$$

$$\phi_x = 0.5[1 + \alpha_x(\lambda x - 0.2) + \lambda_x^2) = 0.5[1 + (0.34(0.47 - 0.2) + (0.47)^2)] = 0.66$$

$$\chi_x = \frac{1}{0.66 + (0.66^2 - 0.47^2)^{\frac{1}{2}}} = 0.59$$

$N_{sd} < \chi_x N_{pl,Rd} \Rightarrow 1{,}199.722\ \text{KN} < 0.59 \times 20{,}973.1 = 12374.3\text{KN}$

About minor axis

Imperfection factor (α_y) =0.49 (buckling curve "c" for Encased profiles buckling about the weak axis

$\chi_y = \frac{1}{\phi_Y + (\phi y^2 - \lambda_y^2)^{\frac{1}{2}}}$ and

$\phi_y = 0.5[1 + \alpha_y(\lambda_y - 0.2) + \lambda_y^2) = 0.5[1 + (0.49(0.05 - 0.2) + (0.05)^2)] = 0.465$

$\chi_y = \frac{1}{0.465 + (0.465^2 - 0.05^2)^{\frac{1}{2}}} = 0.530$

$N_{sd} < \chi_y N_{pl,Rd} \Rightarrow 1{,}199.722\text{KN} < 0.53 \times 20{,}973.1 = 11{,}115.743\text{KN}$

Therefore, the design is OK for axial compression.

- **Check for second order effects**

Isolated non – sway columns need not be checked for second order effects if:

$\frac{N_{sd}}{N_{cr,x}} \leq 0.1$ For major axis bending i.e $\frac{1{,}199.722}{7{,}955.11} = 0.09 \leq 0.1$

$\frac{N_{sd}}{N_{cr,y}} \leq 0.1$ For minor axis bending i.e $\frac{1{,}199.722}{908{,}387.6} = 0.002 \leq 0.1$

Therefore, check for second order effects are not necessary.

- **Resistance of the composite column under axial compression and bi-axial bending.**

Axial plastic capacity of concrete section;

$$N_{pm,Rd} = A_c f_{cd} = \left(60{,}407.6 \times 0.85 \times \frac{20}{1.5}\right) \times 10^{-3} = 684.62\text{KN}$$

About Major axis

Plastic moment capacity of column cross – section $M_{pl,Rd}$

Plastic section modulus of reinforcing bars;

$W_{ps} = \sum A_{si} \times |ei| = 4 \times \left(\pi \times \frac{12^2}{4}\right) \times \left(\frac{250}{2} - 25 - \frac{12}{2}\right) = 42.52 \times 10^3\text{mm}^3$

Plastic section modulus of structural steel section $W_{pa} = 541.2 \times 10^3\text{mm}^3$

Plastic section modulus of the concrete cross-section (section assumed un-cracked).

$$W_{pc} = b_c \times \frac{h_c^2}{4} - W_{pa} - W_{ps} = \frac{250^3}{4} - 42.52 \times 10^3 - 541.2 \times 10^3$$

$$= 3{,}322.53 \times 10^3\text{mm}^3$$

By trial and error; Check that the position of PNA is 1st in the web of the structural steel.

$$h_n = \frac{\left(A_c \times f_{cd} - A'_{sn} \times (2 \times f_{sd} - f_{cd})\right)}{(2 \times b_c \times f_{cd} + 2 \times t_w \times (2 \times f_{yd} - f_{cd})}$$

$$h_n = \frac{\left(60{,}407.6 \times \frac{0.85 \times 20}{1.5}\right)}{(2 \times 250 \times \frac{0.85 \times 20}{1.5} + 2 \times 4.7 \times (2 \times \frac{355}{1.1} - \frac{0.85 \times 20}{1.5})} = 58.87\text{mm}$$

Since $h_n < \frac{h}{2} - t_f = \frac{140}{2} - 6.9 = 63.1\text{mm}$ Therefore; the PNA is in the web

A’s= 0 as there is no reinforcement within the region of the steel web.

Section modulus about neutral axis

$W_{plsn} = 0$ (As there is no reinforcement within the region of $2h_n$ from the middle line of the cross section).

$$w_{pan} = t_w \times h_n^2 = 4.7 \times 58.87^2 = 16.29 \times 10^3\text{mm}^3$$

$$w_{pcn} = b_c \times h_n^2 - w_{pan} - w_{psn} = 250 \times 58.87^2 - 16.29 \times 10^3 - 0 = 850.13 \times 10^3\text{mm}^3$$

Plastic moment resistance of section($M_{pl,Rd}$) which is defined as:

$$M_{pl,Rd} = f_{yd} \times \left(w_{pa} - w_{pan}\right) + 0.5 \times f_{cd}\left(w_{pc} - w_{pcn}\right) + f_{sd}\left(w_{ps} - w_{psn}\right)$$

$$= \left[\frac{355}{1.1} \times (541.2 \times 10^3 - 16.29 \times 10^3) + 0.5 \times \frac{0.85 \times 20}{1.5}(3{,}322.53 \times 10^3 - 850.13 \times 10^3) + \frac{300}{1.15} \times (42.52 \times 10^3 - 0)\right] \times 10^{-6}$$

$= 194.5$KNm

About Minor axis

Plastic moment capacity of column cross-section($M_{pl,Rd}$).

Plastic section modulus of reinforcing bars.

$$W_{ps} = \sum A_{si} \times |ei| = 4 \times \left(\pi \times \frac{12^2}{4}\right) \times \left(\frac{250}{2} - 25 - \frac{12}{2}\right) = 42.52 \times 10^3\text{mm}^3$$

Plastic section modulus of structural steel section $W_{pa} = 19.25 \times 10^3\text{mm}^3$

Plastic section modulus of the concrete cross-section (section assumed un-cracked).

$$W_{pc} = b_c \times \frac{h_c^2}{4} - W_{pa} - W_{ps} = \frac{250^3}{4} - 42.52 \times 10^3 - 19.3 \times 10^3 = 3{,}844.5 \times 10^3 \text{ mm}^3$$

By trial and error; Check that the position of PNA is 1st in the web.

$$h_n = \frac{\left(A_c \times f_{cd} - A'_{sn} \times (2 \times f_{sd} - f_{cd})\right) + t_w(2t_f - h)(2fyd - fcd)}{(2 \times hc \times f_{cd}) + 4t_f \times (2 \times fyd - fcd)}$$

$$h_n = \frac{\left(60{,}407.6 \times \frac{0.85 \times 20}{1.5}\right) + 4.7 \times (2 \times 6.9 - 140) \times (2 \times \frac{355}{1.1} - \frac{0.85 \times 20}{1.5})}{(2 \times 250 \times \frac{0.85 \times 20}{1.5}) + 4 \times 6.9 \times (2 \times \frac{355}{1.1} - \frac{0.85 \times 20}{1.5})}$$

$$= 13.31\text{mm} \qquad \text{since; } \left(\frac{t_w}{2} < h_n < \frac{b}{2}\right) = (\frac{4.7}{2} < 13.31 < \frac{73}{2})$$

$A'_{sn} = 0$ As there is no reinforcement within the region of the steel web.

Therefore; the PNA is in the web.

Section modulus about neutral axis

$W_{psn} = 0$ (As there is no reinforcement within the region of $2h_n$ from the middle line of the cross section).

$$w_{pan} = 2 \times t_w \times h_n^2 + \frac{(h-2\times t_f)}{4} \times t_w^2 = 2 \times 4.7 \times 13.31^2 \times + \frac{(140-2\times 6.9)}{4} \times 4.7^2 = 2{,}362.21\text{mm}^3$$

$$w_{pcn} = b_c \times h_n^2 - w_{pan} - w_{psn} = 250 \times 13.31^2 - 2{,}362.21 - 0 = 41.93 \times 10^3\text{mm}^3$$

Plastic moment resistance of section ($M_{pl,Rd}$) which is defined as:

$$M_{pl,Rd} = f_{yd} \times \left(w_{pa} - w_{pan}\right) + 0.5 \times f_{cd}\left(w_{pc} - w_{pcn}\right) + f_{sd}\left(w_{ps} - w_{psn}\right)$$

$$= \left[\frac{355}{1.1} \times (19.25 \times 10^3 - 2{,}362.21) + 0.5 \times \frac{0.85 \times 20}{1.5}(3{,}844.5 \times 10^3 - 41.93 \times 10^3) + \frac{300}{1.15} \times (42.52 \times 10^3 - 0)\right] \times 10^{-6}$$

$= 38.1\text{KNm}$

- **Checking Column Resistance against Combined Compression and Bi-Axial Bending.**

The design against combined compression and bi-axial bending is adequate if following conditions are satisfied: $M_{sd} \leq 0.9\mu M_{pl,Rd}$

About major axis

$M_{sd,x} = 11.29\text{KNm}$ and $M_{pl,Rd} = 194.5\text{KNm}$

$$\mu_x = 1 - \frac{(1-\chi_x) \times \chi_d}{(1-\chi_c) \times \chi_x} = 1 - \frac{(1-0.59) \times 0.06}{(1-0.033) \times 0.59} = 0.96 \text{ where } \chi_d = \frac{N_{sd}}{N_{pl,Rd}} = \frac{1{,}199.722}{20{,}973.1}$$

$$= 0.06 \text{ and } \chi_c = \frac{N_{pm,Rd}}{N_{pl,Rd}} = \frac{684.62}{20{,}973.1} = 0.033$$

$$0.9\mu M_{pl,Rd} = 0.9 \times 0.96 \times 194.5 = 167.5\text{KNm} > 11.29\text{KNm} \ldots\ldots\ldots\ldots\ldots..\text{ok}$$

About minor axis

$M_{sd,y} = 6.46\text{KNm}$ and $M_{pl,Rd} = 38.1\text{KNm}$

$$\mu_y = 1 - \frac{(1-\chi_y) \times \chi_d}{(1-\chi_c) \times \chi_y} = 1 - \frac{(1-0.53) \times 0.06}{(1-0.033) \times 0.53} = 0.952$$

$0.9\mu\ M_{pl,Rd,y}$=0.9×0.952×38.1=32.64KNm>6.46KNm---------------ok

$$\frac{1,199.722}{20,973.1} + \frac{11.29}{194.5} + \frac{6.46}{38.1} \leq 1.0 \leftrightarrow 0.85 \leq 1.0 \text{ ---------- ok}$$

Since design check is satisfied, the composite column is acceptable.

3.2.2. Design example of composite beam

The 1st floor beam along x-axis (axis 3) from model-1X has been selected and checked/Designed .The auto selected section from ETABS 2015 version 15.0.0 which is economical and safe section is shown in the following figure. This auto selected section is designed manually following the code procedure. Step by step design procedure is presented below. Material used and design assumption are shown in annexes.

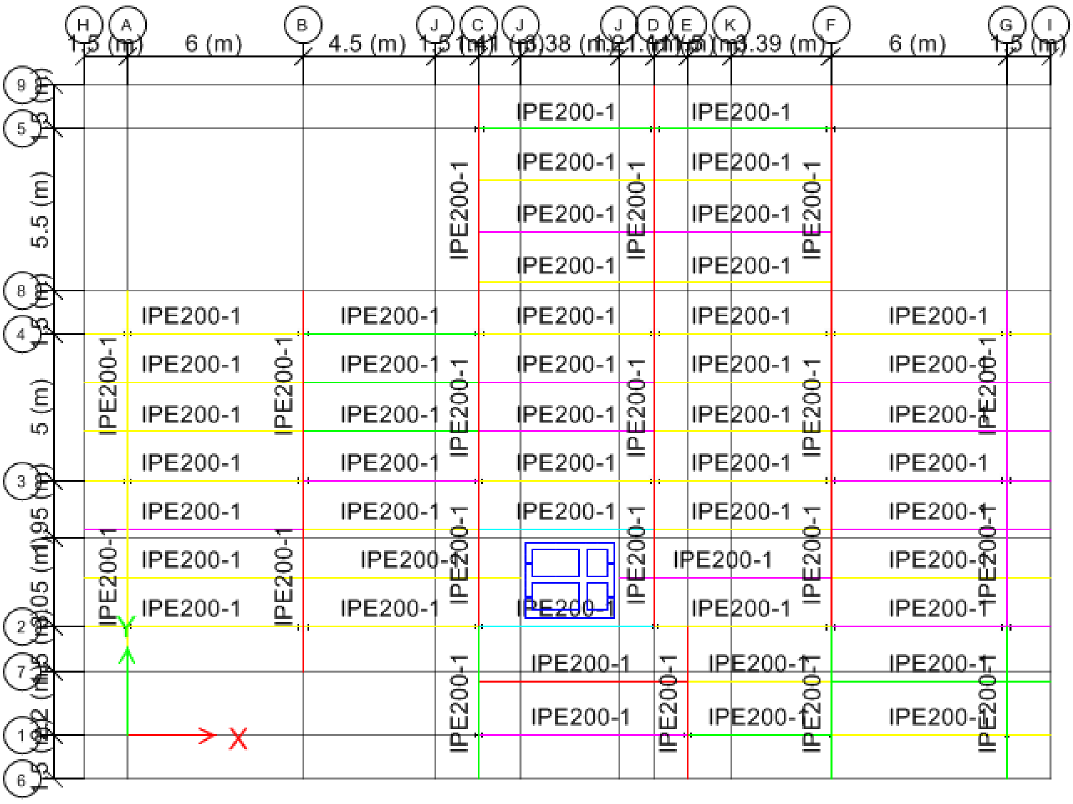


Fig. 10: Beam layouts for ground floor for model-1X

Step-1: Load Calculation

Span lengths: L=6m+6m+6m++6m+6m

- **Construction stage:**

Dead Load

Self-weight of slab=2x0.15x25= 7.5kN/m

Self-weight of beam =0.246kN/m (IPE 200)

Total design dead load =1.3x 7.75=10.07kN/m

Live Load

Construction Load =1x 2=2 kN/m

Total design live load =1.6x2=3.2kN/m

- **composite stage**

Dead load

Self-weight of slab=2x0.15x25=7.5 kN/m

Self-weight of beam=0.246kN/m

Load from floor finish =0.5x 2=1 kN/m

Total design dead load =1.3x8.746= 11.37kN/m

Live Load

Live Load =2x2=4 kN/m

Partition Load =1x2=2 kN/m

Total design live load =1.6x (6) =7.2 kN/m

Step-2: Bending Moment and Shear Force Calculation

- **Construction Stage**

Maximum positive moment $M_{sd}(+) = \left(\frac{DD(L^2)}{12} + \frac{DL(L^2)}{10}\right) = \left(\frac{10.07 \times 6^2}{12} + \frac{3.2 \times 6^2}{10}\right) = 41.73KNm$

Maximum negative moment $M_{sd}(-) = -\left(\frac{DD(L^2)}{10} + \frac{DL(L^2)}{9}\right) = -\left(\frac{10.07 \times 6^2}{10} + \frac{3.2 \times 6^2}{9}\right) = -49.1KNm$

Maximum shear force $(V_{sd}) = 0.6[(DD) \times L) + L \times (LD)] = 0.6[(10.07 \times 6) + (3.2 \times 6)] = 47.77KN$

- **Composite stage**

Maximum positive moment $M_{sd}(+) = \left(\frac{DD(L^2)}{12} + \frac{DL(L^2)}{10}\right) = \left(\frac{11.37 \times 6^2}{12} + \frac{7.2 \times 6^2}{10}\right) = 66.85KNm$

Maximum positive moment $M_{sd}(-) = -\left(\frac{DD(L^2)}{10} + \frac{DL(L^2)}{9}\right) = -\left(\frac{11.37 \times 6^2}{10} + \frac{7.2 \times 6^2}{9}\right) = -69.7KNm$

Maximum shear force $(V_{sd}) = 0.6[DDL + DLL] = 0.6[(11.37 \times 6) + (7.2 \times 6)] = 67.93KN$

Step-3: Section Selection

Selected Section Properties is IPE 200:

$h = 200mm$ $b = 100mm$ $t_f = 8.5mm$ $t_w = 5.6mm$

$r = 12mm$ $w = 0.246\ \frac{KN}{m}$ $A_a = 28.5 \times 10^2\ mm^2$ $W_{pl,x} = 220.6 \times 10^3\ mm^3$

$i_x = 8.26 \times 10mm$ $I_x = 1943 \times 10^4\ mm^4$ $W_{pl,y} = 44.61 \times 10^3\ mm^3$

$i_y = 2.24 \times 10mm$ $I_y = 142.4 \times 10^4\ mm^4$ $I_t = 6.98 \times 10^4\ mm^4$

$I_w = 12.99 \times 10^9\ mm^6$

Step-4: **Classification of composite section**

Steel section is classified in the most unfavorable class resulting from its elements that are under compression.

Flange: $\frac{0.5b}{t_f} = \frac{0.5 \times 100}{8.5} = 5.88\ ; \varepsilon = \sqrt{\frac{250}{355}} = 0.84 \rightarrow 10\varepsilon = 10 \times 0.84 = 8.4$

$\Rightarrow 5.88 \leq 8.4$, Therefore, Flane is class I

web : $\frac{d}{t_w} = \frac{183}{5.6} = 32.67$; $\varepsilon = \sqrt{\frac{250}{355}} = 0.84 \rightarrow 72\varepsilon = 72 \times 0.84 = 60.48$

$\Rightarrow 32.67 \leq 60.48$, Therefore, Web is class I

Therefore, whole cross- section is Class I

Step -5: Ultimate Limit State

- **Construction Stage**

  - **Analysis (Design action effect)**

Analysis based on linear elastic theory. Resistance based on plastic section capacity →method of analysis and design Elastic-Plastic

- **Plastic Moment Resistance of the Steel Section.**

$$M_{pl,Rd} = f_y \times \frac{W_{pl,X}}{\gamma_a} = \left[355 \times \frac{220.6 \times 10^3}{1.1}\right] \times 10^{-6} = 71.2\text{KNm} > 41.73\text{KNm}$$

- **Design shear resistance of steel beam**

$$A_v = 1.04 h_a t_w = 1.04 \times (200 - 2 \times 8.5) \times 5.6 = 1{,}065.792\text{mm}^2$$

$$V_{pl,Rd} = \frac{A_v(\frac{f_y}{3^{\frac{1}{2}}})}{\gamma_a} = \left[\frac{\left[1{,}065.792 \times \left(\frac{355}{3^{\frac{1}{2}}}\right)\right]}{1.1}\right] \times 10^{-3} = 198.585\text{KN} > 47.77\text{KNm}$$

- **Moment-shear interaction**

$V_{sd} < 0.5 \times V_{pl,Rd} = 99.3\text{KN} \leftrightarrow 47.77 < 99.3$

Therefore, effect of shear on moment capacity of steel beam can be neglected.

- **Checking for lateral torsional buckling of the steel Beam**

$$M_{pl,Rd} = \frac{\chi_{LT} \times \beta_w \times W_{pl,y} f_y}{\gamma_{M1}} = \left[\frac{0.28 \times 1.0 \times 220.6 \times 10^3 \times 355}{1.1}\right] \times 10^{-6}$$

$$= 65.89\text{KNm} > 41.73\text{KNm}$$

Where $\chi_{LT} = \frac{1}{[\phi_{LT} + (\phi_{LT}^2 - \overline{\lambda_{LT}^2})^{\frac{1}{2}}]} = \frac{1}{2.06 + (2.06^2 - 1.39^2)^{1/2}} = 0.38$

Where $\phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda_{LT}} - 0.2) + \overline{\lambda_{LT}^2}] = 0.5[1 + 0.21(1.39 - 0.2) + 1.39^2]$

$= 2.06$

$\alpha_{LT} = 0.21\ for\ rolled\ sections.$

$$\overline{\lambda_{LT}} = \left(\frac{\lambda_{LT}}{\lambda_1}\right) \beta_w^{\ 0.5} = \frac{106.34}{76.41} \times 1^{1/2} = 1.39$$

$$\lambda_{LT}=\frac{0.9(\frac{L}{i_y})}{C_1^{1/2}\left[1+\left(\frac{1}{20}\right)\times\frac{(\frac{L}{i_y})^2}{\left(\frac{h}{t_f}\right)^2}\right]^{0.25}}=\frac{0.9\left(\frac{6000}{2.24\times10}\right)}{(1.879)^{0.5}\left[1+\left(\frac{1}{20}\right)\times\frac{\left(\frac{6000}{2.24\times10}\right)^2}{\left(\frac{200}{8.5}\right)^2}\right]^{0.25}}=106.34$$

$$\lambda_1=\Pi\left(\frac{E}{f_y}\right)^{0.5}=\Pi\left(\frac{210}{355}\right)^{0.5}=76.41$$

Where $C_1=1.879$(coefficient as a function of moment distribution); $\beta_w=1.0$(class 1 cross-section); $\gamma_{M1}=1.1$

- **Composite stage**

Moment Resistance of the cross section:

Negative Bending Moment

At internal support negative bending moment of resistance is obtained by considering the tensile resistance of the reinforcement. Concrete area is neglected.

- Effective width of the concrete flange

$$b_{eff}=\frac{2l_0}{8}=\frac{1}{4}\times0.25(L_1+L_2)=\frac{1}{4}\times0.25\times(6000+6000)=750mm$$

Let us provide 12mmϕ@100mmc/c $\quad A_s=8\left(\frac{\pi\times12^2}{4}\right)=904.78mm^2$

- Location of neutral axis

$$F_a=\frac{A_af_y}{\gamma_s}=\left[\frac{2{,}850\times355}{1.1}\right]\times10^{-3}=919.77KN$$

$$F_s=\frac{A_sf_{sk}}{\gamma_s}=\left[\frac{904.78\times300}{1.1}\right]\times10^{-3}=246.76\,KN$$

$$F_w=\frac{dt_wf_y}{\gamma_s}=\left[\frac{\big(200-(2\times8.5)\big)\ \times5.6\times355}{1.1}\right]\times10^{-3}=330.73KN>F_s$$

$$\Rightarrow PNA\ lies\ in\ web$$

$$The\ tension\ zone\ Z_{cw}=\frac{h}{2}-\frac{F_s}{2t_wf_y/\gamma_a}=\left[\frac{200}{2}-\frac{183}{(2\times5.6\times355)/1.1}\right]=99.95mm$$

Negative Moment of resistance of the section($M_{pl,Rd}$):

$$M_{pl,Rd}=M_{pl,Rd}+F_s\left(\frac{h}{2}+a\right)-\frac{F_s^2}{\frac{4t_wf_y}{\gamma_a}}=65.89+\left[277.46\left(\frac{200}{2}+85\right)-\frac{277.46^2}{4\times5.6\times\frac{355}{1.1}}\right]\times10^{-3}=$$

$117.21KNm>69.7KNm$ …………………ok!

Whereas a = location of reinforcement from top surface of upper flange = 85 mm and concrete cover = 25 mm

Positive Bending Moment

- Effective width of the concrete flange.

$$b_{eff} = \frac{L_o}{4} = \frac{1}{4}(0.8x6000) = 1200\ mm$$

- Location of PNA

$$F_a = \frac{A_a f_y}{\gamma_s} = \left[\frac{2850x355}{1.1}\right] \times 10^{-3} = 919.77KN$$

$$F_c = \frac{0.85 f_{ck} b_{eff} h_c}{\gamma_c} = \left[\frac{0.85x20x1200x250}{1.5}\right] \times 10^{-3} = 3{,}400\ KN\ > F_a$$

PNA lies in concrete component with neutral axis depth determined from:

$$Z_c = \frac{\left(\frac{f_y A_a}{\gamma_a}\right)}{\left(\frac{0.85 f_{ck} b_{eff}}{\gamma_c}\right)} = \left[\frac{\left(\frac{2850x355}{1.1}\right)}{\left(\frac{0.85x20x1200}{1.5}\right)}\right] = 67.63mm$$

Positive moment of the section($M_{pl,Rd}$) is given by:

$$M_{pl,Rd} = \frac{f_y A_a}{\gamma_a}\left(\frac{h}{2} + h_c - \frac{Z_c}{2}\right) = \left[\frac{355x2850}{1.1}\left(\frac{200}{2} + 150 - \frac{67.63}{2}\right)\right] \times 10^{-6}$$

$$= 198.84\ KNm > 66.85KNm$$

- **Check for vertical shear and bending moment and shear force interaction**

Vertical shear force,$V_{pl,Rd} = 198.585KN > V_{sd} = 67.93KN \Rightarrow Hence\ safe$

Bending Moment and vertical shear interaction can be neglected if:

$V_{sd} < 0.5V_{pl,Rd}$; $67.93KNm < 0.5x198.585KNm = 99.3\ KNm$

$\Rightarrow effect\ of\ shear\ on\ moment\ capacity\ has\ not\ to\ be\ considered$.

- **Check for shear buckling**

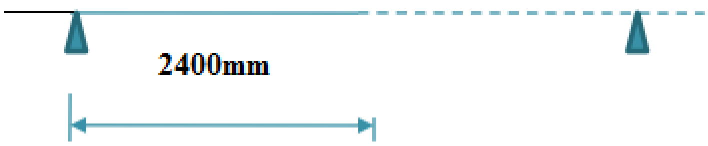
$$\frac{d}{t_w} = \frac{(250-(2\times 8.5))}{5.6} = 41.61 < 69 \times 0.84 = 57.97$$ Hence safe

Step 6: Design of shear connectors

The full shear connection is assumed for this design work.

- Between simple end support and point of maximum positive moment

  Length (shear span) =0.4L = 0.4x6000 = 2,400mm



Longitudinal shear force$(V_l) = F_a = 919.77KN$

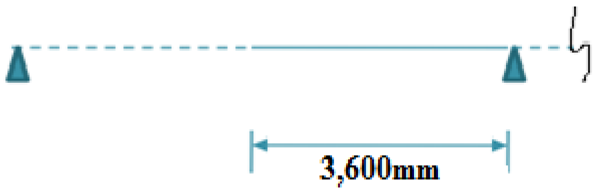
Design resistance of shear connectors: Let us provide 22mm,ϕ100mm high, $P_{,Rd}$=70KN

Assuming full shear connection: No. of shear connectors=$\frac{Vl}{P,Rd}$=$\frac{919.77}{70}$=13

Spacing =2,400mm/13=92mm

- Between simple end support and point of maximum positive moment

  Length (shear span) =6000mm-2400mm



Longitudinal shear force $(V_l) = F_a + F_s = F_a + \frac{f_y A_s}{\gamma_s} = 919.77 + 246.76 = 1166.53KN$

Design resistance of shear connectors: Let us provide 22mm,ϕ100mm high,$P_{,Rd}$=70KN

Assuming full shear connection: No. of shear connectors=$\frac{Vl}{P,Rd}$=$\frac{1,166.53}{70}$=17

Spacing =3600mm/17=210mm

4. Design output and Discussion

4.1. Design output

The summary of necessary outputs obtained from data from structural design at Addis Ababa saving Houses Development Enterprise for reinforced concrete frame and design results for steel-concrete composite frame of model-1X, model-2X, model-3X, model-1Y, model-2Y, model-3Y, model-1Z, model-2Z, model-3Z and model-4Z are presented in Table 2, 3, 4, 5 and Table 6 respectively. To show the whole process of computation involved in the design, sample design are shown in chapter three and the detailed representative design for sample models are attached in Annexes.

4.2. Summary on Outputs

4.2.1. Cost Analysis

The materials volume obtained from the output of the design and current material prices are used for the cost analysis in order to identify the economical span and storey of the steel-concrete composite frames.

The cost of the steel-concrete composite frame will be indirectly proportional to the span length of the beam/column spacing. The cost of substructure will be the cost of foundations, which will not vary.

For cost analysis, current values of construction costs including overhead cost are used here. Table 7 below shows the unit rate for different items.

Table 2 Summary of output for Reinforced concrete Beams

Beams							
Position	Beam ID	No.	Size	Reinforcement Bars			
				Main bar		Shear Reinforcement	No.
				Negative	Positive		
Foundation & Ground floor	GFB-1	1	500x300	$5\varphi_{20}$	$8\varphi_{20}$	φ_{10} c/c 100	2
	GFB-2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10} c/c 100	7
	GFB-3	1	500x300	$17\varphi_{24}$	$27\varphi_{24}$	φ_{10}c/c 130	10
	GFB-4	1	500x300	$18\varphi_{20}$	$28\varphi_{20}$	φ_{8} c/c 100	12
	GFB-5	1	500x300	$5\varphi_{16}$	$17\varphi_{16}$	φ_{8} c/c 100	4
	GFB-A	1	500x300	$6\varphi_{16}$	$10\varphi_{16}$	φ_{8} c/c 160	4
	GFB-B	1	500x300	$8\varphi_{16}$	$10\varphi_{16}$	φ_{10}c/c 166	3
	GFB-C	1	500x300	$11\varphi_{20}$	$16\varphi_{20}$	φ_{10} c/c 130	5
	GFB-D	1	500x300	$15\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	GFB-D'	1	500x300	$6\varphi_{16}$	$6\varphi_{16}$	φ_{8} c/c 130	1
	GFB-E	1	500x300	$11\varphi_{20}$	$20\varphi_{20}$	φ_{10}c/c 130	3
	GFB-F	1	500x300	$8\varphi_{21}$	$10\varphi_{21}$	φ_{10}c/c 130	3
first floor	FFB-1	1	500x300	$5\varphi_{20}$	$12\varphi_{20}$	φ_{10}c/c 100	2
	FFB-2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10}c/c 100	7
	FFB-3	1	500x300	$17\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	FFB-4	1	500x300	$18\varphi_{20}$	$28\varphi_{20}$	φ_{10}c/c 100	12
	FFB-5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_{8}c/c 100	4
	FFB-A	1	500x300	$6\varphi_{16}$	$10\varphi_{16}$	φ_{8}c/c 100	1
	FFB-B	1	500x300	$3\varphi_{20}$	$6\varphi_{16}$	φ_{8}c/c 130	2
	FFB-C	1	500x300	$14\varphi_{16}$	$11\varphi_{16}$	φ_{8}c/c 130	2
	FFB-D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{8}c/c 130	2
	FFB-D'	1	500x300	$4\varphi_{16}$		φ_{8}c/c 130	1
	FFB-E	1	500x300	$8\varphi_{16}$	$13\varphi_{16}$	φ_{10}c/c 130	3
	FFB-F	1	500x300	$10\varphi_{16}$	$12\varphi_{16}$		

Beams							
Position	Beam ID	No.	Size	Reinforcement Bars			
				Main bar		Shear Reinforcement	No.
				Negative	Positive		
second floor	SFB-1	1	500x300	$5\varphi_{20}$	$12\varphi_{20}$	φ_{10} c/c 100	2
	SFB-2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10} c/c 100	7
	SFB-3	1	500x300	$17\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	SFB-4	1	500x300	$18\varphi_{20}$	$9\varphi_{16}$	φ_8 c/c 100	12
	SFB-5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_8 c/c 100	4
	SFB-A	1	500x300		$6\varphi_{16}$	φ_8 c/c 160	4
	SFB-B	1	500x300	$3\varphi_{20}$	$2\varphi_{20}$	φ_{10}c/c 166	3
	SFB-C	1	500x300	$13\varphi_{16}$	$20\varphi_{20}$	φ_{10} c/c 130	5
	SFB-D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	SFB-D'	1	500x300		$4\varphi_{16}$	φ_8 c/c 130	1
	SFB-E	1	500x300	$10\varphi_{20}$	$16\varphi_{16}$	φ_{10}c/c 130	3
	SFB-F	1	500x300	$10\varphi_{16}$	$6\varphi_{16}$	φ_{10}c/c 130	3
3rd floor	TFB-1	2	500x300	$3\varphi_{16}$	$10\varphi_{16}$	φ_{10} c/c 100	2
	TFB-2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10} c/c 100	7
	TFB-3	1	500x300	$17\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	TFB-4	1	500x300	$18\varphi_{20}$	$9\varphi_{16}$	φ_8 c/c 100	12
	TFB-5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_8 c/c 100	4
	TFB-A	1	500x300		$6\varphi_{16}$	φ_8 c/c 160	4
	TFB-B	1	500x300	$3\varphi_{20}$	$2\varphi_{20}$	φ_{10}c/c 166	3
	TFB-C	1	500x300	$13\varphi_{16}$	$20\varphi_{20}$	φ_{10} c/c 130	5
	TFB-D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	TFB-D'	1	500x300		$4\varphi_{16}$	φ_8 c/c 130	1
	TFB-E	1	500x300	$10\varphi_{20}$	$16\varphi_{16}$	φ_{10}c/c 130	3
	TFB-F	1	500x300	$10\varphi_{16}$	$6\varphi_{16}$	φ_{10}c/c 130	3
4th floor	1	1	500x300	$3\varphi_{16}$	$10\varphi_{16}$	φ_{10} c/c 100	2
	2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10} c/c 100	7
	3	1	500x300	$17\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	4	1	500x300	$18\varphi_{20}$	$9\varphi_{16}$	φ_8 c/c 100	12
	5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_8 c/c 100	4
	A	1	500x300		$6\varphi_{16}$	φ_8 c/c 160	4
	B	1	500x300	$3\varphi_{20}$	$2\varphi_{20}$	φ_{10}c/c 166	3
	C	1	500x300	$13\varphi_{16}$	$20\varphi_{20}$	φ_{10} c/c 130	5
	D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	D'	1	500x300		$4\varphi_{16}$	φ_8 c/c 130	1
	E	1	500x300	$10\varphi_{20}$	$16\varphi_{16}$	φ_{10}c/c 130	3
	F	1	500x300	$10\varphi_{16}$	$6\varphi_{16}$	φ_{10}c/c 130	3

Beams							
Position	Beam ID	No.	Size	Reinforcement Bars			
				Main bar		Shear Reinforcement	No.
				Negative	Positive		
5th floor	1	1	500x300	$3\varphi_{16}$	$10\varphi_{16}$	φ_{10} c/c 100	2
	2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10} c/c 100	7
	3	1	500x300	$17\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	4	1	500x300	$18\varphi_{20}$	$9\varphi_{16}$	φ_{8} c/c 100	12
	5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_{8} c/c 100	4
	A	1	500x300		$6\varphi_{16}$	φ_{8} c/c 160	4
	B	1	500x300	$3\varphi_{20}$	$2\varphi_{20}$	φ_{10}c/c 166	3
	C	1	500x300	$13\varphi_{16}$	$20\varphi_{20}$	φ_{10} c/c 130	5
	D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	D'	1	500x300		$4\varphi_{16}$	φ_{8} c/c 130	1
	E	1	500x300	$10\varphi_{20}$	$16\varphi_{16}$	φ_{10}c/c 130	3
	F	1	500x300	$10\varphi_{16}$	$6\varphi_{16}$	φ_{10}c/c 130	3
6th floor	1	1	500x300	$3\varphi_{16}$	$10\varphi_{16}$	φ_{10} c/c 100	2
	2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10} c/c 100	7
	3	1	500x300	$17\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	4	1	500x300	$18\varphi_{20}$	$9\varphi_{16}$	φ_{8} c/c 100	12
	5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_{8} c/c 100	4
	A	1	500x300		$6\varphi_{16}$	φ_{8} c/c 160	4
	B	1	500x300	$3\varphi_{20}$	$2\varphi_{20}$	φ_{10}c/c 166	3
	C	1	500x300	$13\varphi_{16}$	$20\varphi_{20}$	φ_{10} c/c 130	5
	D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	D'	1	500x300		$4\varphi_{16}$	φ_{8} c/c 130	1
	E	1	500x300	$10\varphi_{20}$	$16\varphi_{16}$	φ_{10}c/c 130	3
	F	1	500x300	$10\varphi_{16}$	$6\varphi_{16}$	φ_{10}c/c 130	3

Beams							
Position	Beam ID	No.	Size	Reinforcement Bars			
				Main bar		Shear Reinforcement	No.
				Negative	Positive		
7^{th} floor	1	1	500x300	$3\varphi_{16}$	$10\varphi_{16}$	φ_{10} c/c 100	2
	2	1	500x300	$12\varphi_{20}$	$25\varphi_{20}$	φ_{10} c/c 100	7
	3	1	500x300	$17\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	4	1	500x300	$18\varphi_{20}$	$9\varphi_{16}$	φ_{8} c/c 100	12
	5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_{8} c/c 100	4
	A	1	500x300		$6\varphi_{16}$	φ_{8} c/c 160	4
	B	1	500x300	$3\varphi_{20}$	$2\varphi_{20}$	φ_{10}c/c 166	3
	C	1	500x300	$13\varphi_{16}$	$20\varphi_{20}$	φ_{10} c/c 130	5
	D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	D'	1	500x300		$4\varphi_{16}$	φ_{8} c/c 130	1
	E	1	500x300	$10\varphi_{20}$	$16\varphi_{16}$	φ_{10}c/c 130	3
	F	1	500x300	$10\varphi_{16}$	$6\varphi_{16}$	φ_{10}c/c 130	3
8^{th} floor	1	1	500x300	$3\varphi_{16}$	$10\varphi_{16}$	φ_{10} c/c 100	2
	2	1	500x300	$8\varphi_{20}$	$23\varphi_{20}$	φ_{10} c/c 100	7
	3	1	500x300	$16\varphi_{20}$	$27\varphi_{20}$	φ_{10}c/c 130	10
	4	1	500x300	$14\varphi_{20}$	$24\varphi_{20}$	φ_{8} c/c 100	12
	5	1	500x300	$3\varphi_{16}$	$9\varphi_{16}$	φ_{8} c/c 100	4
	A	1	500x300		$6\varphi_{16}$	φ_{8} c/c 160	4
	B	1	500x300		$6\varphi_{16}$	φ_{10}c/c 166	3
	C	1	500x300	$14\varphi_{16}$	$17\varphi_{16}$	φ_{10} c/c 130	5
	D	1	500x300	$13\varphi_{20}$	$13\varphi_{20}$	φ_{10} c/c 130	2
	D'	1	500x300		$4\varphi_{16}$	φ_{8} c/c 130	1
	E	1	500x300	$4\varphi_{16}$	$4\varphi_{16}$	φ_{10}c/c 130	3
	F	1	500x300	$3\varphi_{16}$	$4\varphi_{16}$	φ_{10}c/c 130	3

Table 3 Summary of output for Reinforced concrete column

Columns			
Position	Size	Reinforcement Bar	
		Longitudinal Bar	Shear
Foundation & Ground floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	800X800	$14\varphi_{24}$	Φ10c/c100
	600X600	$14\varphi_{20}$	Φ12c/c120
	500X500	$12\varphi_{20}$	Φ12c/c120
	400X400	$8\varphi_{20}$	Φ10c/c130
	400X400	$8\varphi_{16}$	Φ10c/c100
first floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	600x600	$12\varphi_{20}$	Φ10c/c100
	800x800	$14\varphi_{24}$	Φ10c/c130
	500x500	$14\varphi_{20}$	Φ12c/c120
	450x450	$8\varphi_{20}$	Φ12c/c120
second floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	500x500	$8\varphi_{20}$	Φ12c/c120
	500x500	$12\varphi_{20}$	Φ10c/c100
	600x600	$12\varphi_{20}$	Φ10c/c130
	450x450	$8\varphi_{20}$	Φ12c/c120
3rd floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	450x450	$12\varphi_{20}$	Φ10c/c100
	450x450	$8\varphi_{20}$	Φ12c/c120
	600x600	$12\varphi_{20}$	Φ10c/c130
	500x500	$8\varphi_{20}$	Φ10c/c130
	450x450	$8\varphi_{20}$	Φ12c/c120
4th floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	450x450	$12\varphi_{20}$	Φ10c/c100
	450x450	$8\varphi_{20}$	Φ12c/c120
	500x500	$12\varphi_{20}$	Φ10c/c130
	450x450	$8\varphi_{20}$	Φ10c/c130
	450x450	$8\varphi_{20}$	Φ12c/c120

Columns			
Position	Size	Reinforcement Bar	
		Longitudinal Bar	Shear
5^{th} floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	400x400	$8\varphi_{20}$	Φ10c/c100
	450x450	$12\varphi_{20}$	Φ10c/c130
	450x450	$8\varphi_{20}$	Φ10c/c130
$6\&7^{th}$ floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	400x400	$8\varphi_{20}$	Φ10c/c100
	450x450	$8\varphi_{20}$	Φ10c/c130
8^{th} floor	700X350	$8\varphi_{20}$	Φ12c/c120
	450X450	$12\varphi_{20}$	Φ12c/c120
	400x400	$8\varphi_{20}$	Φ10c/c100
	450x450	$8\varphi_{20}$	Φ10c/c130

Table 4 Summary of Output for model-1X, 1Y and 2Z

Columns					Beams
		Reinforcement Bar			
Position	**Size**	**Longitudinal Bar**	**Shear**	**Structural Steel**	**Type**
Foundation & Ground floor	400x350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 200
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 200
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 200
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 200
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 200
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 200
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 200
first floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE270	IPE 200
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 200
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 200
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 200
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE200	IPE 200
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 200
second floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
3rd floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
4th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
5th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300
6&7th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300
8th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 200
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 200
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 200

Table 5 Summary of Output for model-2X

Columns					Beams
Position	Size	Reinforcement Bar			Type
		Longitudinal Bar	Shear	Structural Steel	
Foundation & Ground floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE120	IPE 300
	450x450	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 300
	350x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 300
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300
First floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
	450x450	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
Second floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
	350x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
3rd floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	350x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
4th floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
5th floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 360
6 &7th floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 360
8th floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 300
	200x200	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300

Table 6 Summary of Output for model-3X

Column					Beams
Position	Size	Reinforcement Bar			Type
		Longitudinal Bar	Shear	Structural Steel	
Foundation & Ground floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 300
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 300
	500x450	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 300
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 300
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 300
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 300
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 300
first floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 400
	500x450	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
second floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 400
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
3rd floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 400
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 400
4th floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 400
5th floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 400
6&7th floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 400
8th floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 300
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 300
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 300

Table 7 Summary of Output for model-2Y

Columns					Beams
		Reinforcement Bar			
Position	**Size**	**Longitudinal Bar**	**Shear**	**Structural Steel**	**Type**
Foundation & Ground floor	400x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 220
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 220
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 220
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 220
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 220
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 220
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 220
first floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE300	IPE 220
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 220
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 220
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 220
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE220	IPE 220
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 220
second floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 330
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 330
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 330
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 330
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 330
3rd floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 330
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 330
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 330
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 330
4th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 330
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 330
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 220	IPE 330
5th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 330
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 330
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 330
6&7th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 330
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 330
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 330
8th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 220
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 220
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 220

Table 8 Summary of Output for model-3Y

Columns					Beams
		Reinforcement Bar			
Position	**Size**	**Longitudinal Bar**	**Shear**	**Structural Steel**	**Type**
Foundation & Ground floor	400x350	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 240
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 240
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 240
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 240
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 240
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 240
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 240
first floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE330	IPE 240
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 240
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 240
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 240
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE240	IPE 240
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 240
second floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 360
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 360
3rd floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 360
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 360
4th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 360
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 240	IPE 360
5th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 360
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 360
6&7th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 360
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 360
8th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 330	IPE 240
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 180	IPE 240
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 240

Table 9 Summary of Output for model-1Z

Columns					Beams
		Reinforcement Bar			
Position	**Size**	**Longitudinal Bar**	**Shear**	**Structural Steel**	**Type**
Foundation & Ground floor	400x350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 200
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 200
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 200
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 200
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 200
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 200
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 200
first floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE270	IPE 200
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 200
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 200
	400x400	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 200
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE200	IPE 200
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 200
second floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
3rd floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
4th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	300x250	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
5th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300
6th floor	400X350	$4\varphi_{12}$	Φ8c/c200	IPE 270	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 140	IPE 300
	150x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300

Table 10 Summary of Output for model-3Z

Columns					**Beams**
Position	Size	Reinforcement Bar			Type
		Longitudinal Bar	Shear	Structural Steel	
Foundation & Ground floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE120	IPE 300
	450x450	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 300
	350x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 300
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 300
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300
First floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
	450x450	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
Second floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
	350x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
3rd floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	350x300	$4\varphi_{12}$	Φ8c/c200	IPE 200	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
4th floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	300x300	$4\varphi_{12}$	Φ8c/c200	IPE 160	IPE 360
5th floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 360
6 &7th floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 360
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 360
	200x150	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 360
8th -11TH floor	450x350	$4\varphi_{12}$	Φ8c/c200	IPE 300	IPE 300
	250x200	$4\varphi_{12}$	Φ8c/c200	IPE 120	IPE 300
	200x200	$4\varphi_{12}$	Φ8c/c200	IPE 100	IPE 300

Table 11 Summary of Output for model-4Z

Column					Beams
Position	Size	Reinforcement Bar			Type
		Longitudinal Bar	Shear	Structural Steel	
Foundation & Ground floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 300
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 300
	500x450	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 300
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 300
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 300
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 300
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 300
first floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 400
	500x450	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
second floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 400
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
3rd floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	400x300	$4\phi_{12}$	Φ8c/c200	IPE 300	IPE 400
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 400
4th floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	350x300	$4\phi_{12}$	Φ8c/c200	IPE 200	IPE 400
5th floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 400
6&7th floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 400
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 400
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 400
8th -14TH floor	500x350	$4\phi_{12}$	Φ8c/c200	IPE 360	IPE 300
	300x250	$4\phi_{12}$	Φ8c/c200	IPE 160	IPE 300
	250x250	$4\phi_{12}$	Φ8c/c200	IPE 140	IPE 300

Table 12 Unit price for materials

N<u>o</u>	Material	Unit	Unit Price(Birr)
1	Reinforcing Bars	Kg	30
2	Concrete	m^3	3,100
3	Formwork	m^2	280
4	Structural Steel	Kg	57.8

An output of the results of the two types of building frames for reinforced concrete frame with column span of 6m and that of steel-concrete composite frames of beam span ranging from 6m to 10 m along x-axis and their total associated costs are summarized and tabulated in Table 13 below.

Table 13 Total costs for different span lengths along x-direction.

No.	Column spacing(in m)	Designation	Total Cost(Birr)	
			RC frame	Composite
1	6(Original Model)	Model-1X	5,431,174.26	6,227,169.10
2	7.5	Model-2X	-	6,195,569.18
3	10	Model-3X	-	5,151,052.61

Figure 11, shows a relationship between span length and total cost of a superstructure frame for beam span length of 6m for RC frame and 6m, 7.5m and 10m for steel-concrete composite frame along x-direction.

An output of the results of the two types of building frames for reinforced concrete frame with column span of 5m and that of steel-concrete composite frames of beam span ranging from 5m to 10 m along y-axis and their total associated costs are summarized and tabulated in Table 14 below.

Table 14 Total costs for different span lengths along Y-direction.

No.	Column spacing(in m)	Designation	Total Cost(Birr)	
			RC frame	Composite
1	5(Original Model)	Model-1Y	5,431,174.26	6,227,169.10
2	7.5	Model-2Y	-	6,009,328.29
3	10	Model-3Y	-	5,151,052.61

Figure 12, shows a relationship between span length and total cost of a superstructure frame for beam span length of 5m for RC frame and 5m, 7.5m and 10m for steel-concrete composite frame along Y-direction.

An output of the results of the two types of building frames for reinforced concrete frame with number of storey of 9 and that of steel-concrete composite frames of number of storey ranging from 7 to 15 and their total associated costs are summarized and tabulated in Table 15 below.

Table 15 Total costs for different number of storeys.

No.	No of storey	Designation	Total Cost(Birr)	
			RC frame	Composite
1	7	Model-1Z	5,431,174.26	6,227,169.10
2	9	Model-2Z	-	6,100,897.69
3	12	Model-3Z	-	5,151,052.61
4	15	Model-4Z	-	4,938,897.36

Figure 13, shows a relationship between number of storey and total cost of a superstructure frame for number of storey of 9 for RC frame and number of storey of 7, 9,12 and 15 for steel-concrete composite frame.

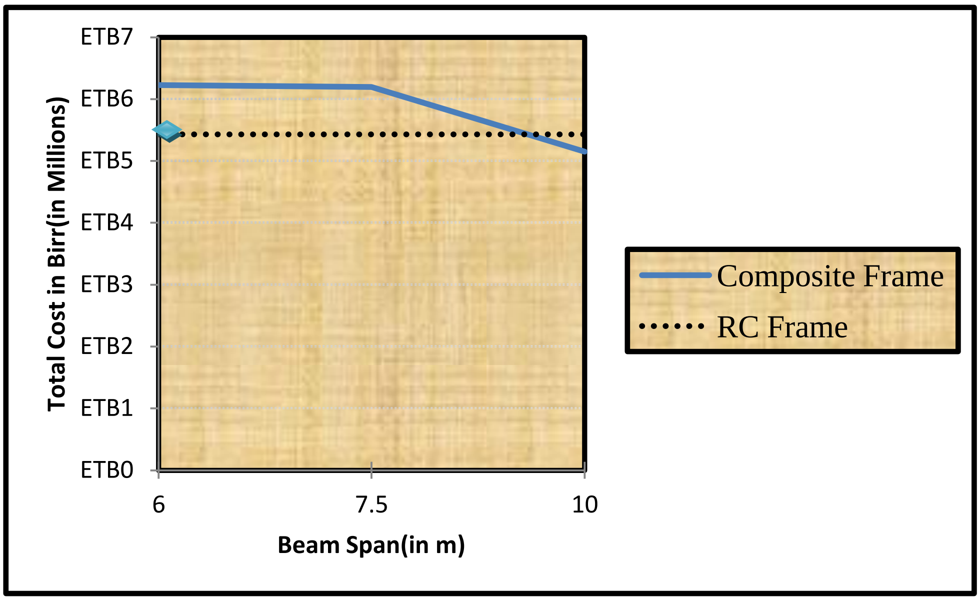


Fig. 11: Beam Span Length/Column Spacing Vs Cost graph along x-direction

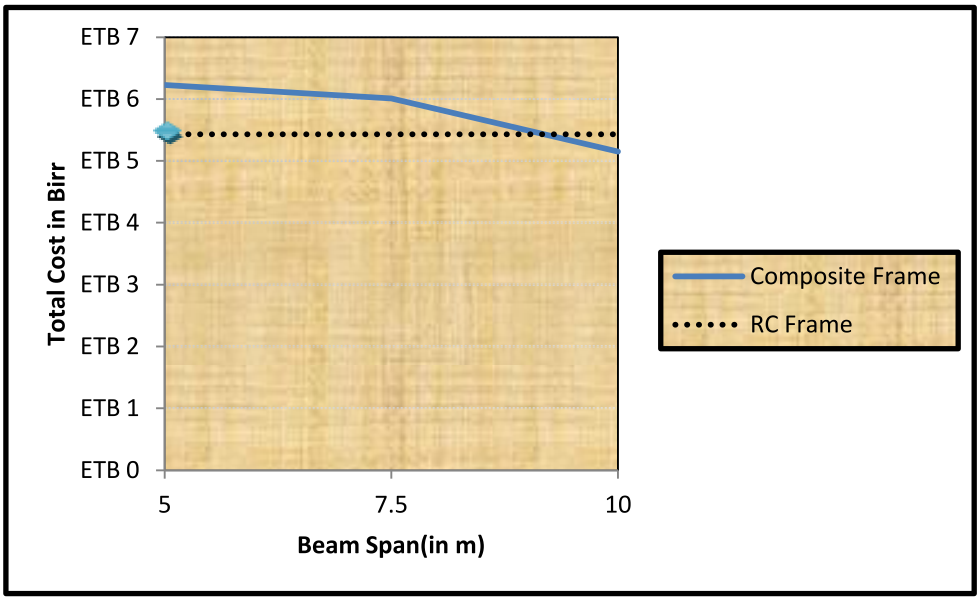


Fig. 12: Beam Span Length/Column Spacing Vs Cost graph along Y-direction

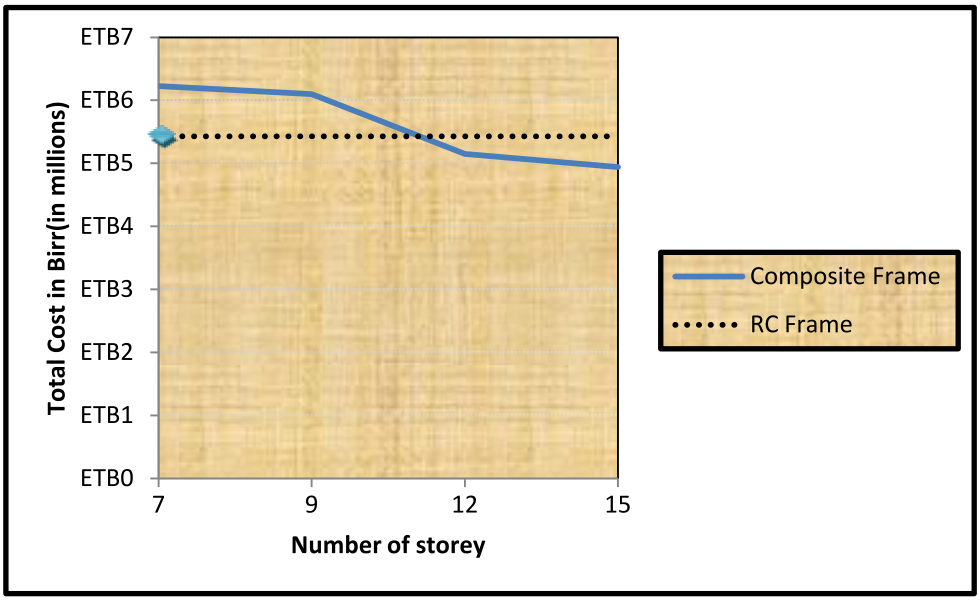


Fig. 13: Number of storeys Vs Cost graph

4.3. Discussions

The total cost of the superstructure of a building frame is considered as a criterion for the comparative study of the building frame.

Form the above tables and graph it can be pointed out that, the cost for Reinforced Concrete frame structure is assumed to be constant. And the estimated cost for steel-concrete composite frame structure has been done by varying the frequently occurred maximum column spacing along both axes that is 6m which is along x-axis and 5m which is along y-axis.

At 6m column spacing along x-axis, the cost for the two frame structures has been estimated.

At 7.5m column spacing along this axis, the cost for steel-concrete composite frame structure has been estimated.

Again for column spacing of 10m along this axis, the cost for steel-concrete composite frame structure has been estimated and tabulated in table 13.

The cost versa column spacing has been plotted and shown in figure 11.

At 5m column spacing along y-axis, the cost for the two frame structures has been estimated.

At 7.5m column spacing along this axis, the cost for steel-concrete composite frame structure has been estimated.

Again for column spacing of 10m along this axis, the cost for steel-concrete composite frame structure has been estimated and tabulated in table 14.

The cost versa column spacing has been plotted and shown in figure 12.

At 9 storeys, the cost for the two frame structures has been estimated.

At 7 storeys, the cost for steel-concrete composite frame structure has been estimated.

At 12 storeys, the cost for steel-concrete composite frame structure has been estimated

Again at 15 storeys, the cost for steel-concrete composite frame structure has been estimated and tabulated in table 15.

The cost versa number of storeys has been plotted and shown in figure 13.

The cost for reinforced concrete frame at 6m and the cost for steel-concrete composite frame at column spacing between 9 to 10m are the same along x-axis.

Again, the cost for reinforced concrete frame at 5m and the cost for steel-concrete composite frame at column spacing between 9 to 10m are the same along y-axis.

The cost for reinforced concrete frame at 9 and the cost for steel-concrete composite frame at storeys ranging 9 to 12 are the same.

If the cost of concrete increases at alarming rate, the above cost effective column spacing will be decreased and vise versa.

If the cost of concrete increases at alarming rate, the above cost effective number of storeys will be decreased and vise versa.

If the cost of structural steel increases at alarming rate, the cost effective column spacing and number of storeys will be increased and vise versa.

5. Conclusion and Recommendation

5.1. Conclusion

In this study, the comparative study of Reinforced concrete frame structure and steel-concrete Composite frame structure for multistoried condominium building frame (G+7) typology is presented. Parameters considered are cost of frames for reinforced concrete frame and steel-concrete composite frames as well as column spacing and number of storeys and from that result conclusions can be drawn-out are as follows:-

- At frequently occurred maximum column spacing i.e 6m which is along X-axis and 5m which is along y-axis, the cost for steel-concrete composite frame is much higher than that of the reinforced concrete frame or steel-concrete composite is uneconomical in our country at this column spacing.
- At 7.5m of frequently occurred maximum column spacing along both axes, again the steel-concrete composite frame is uneconomical in our country.
- Finally, it is found and concluded that when the column spacing is between 9 to 10m ,the frames structure have same cost and above this column spacing the steel-concrete composite frame structure is cost effective than ordinary reinforced concrete frame and when the number of storey is 9 to 12, the frames structure have same cost and below this number of storeys, RC frame is cost effective and vise versa.
- As reviewed in different literatures, the optimum column spacing and number of storeys for steel-concrete composite frame structures is 6 to 12 meters and 10 storeys respectively and the results are within this interval.

5.2. Recommendation

The following recommendations can be drawn starting from results and conclusion:

- Since the prices of materials are fluctuating/changing dynamically, this obtained result will also change accordingly. Therefore; this result works only for limited period of time of nearly stabile price.

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Annexes:

1. Design data and Assumptions

Material data used

Concrete

$f_{ck} = 20\ N/mm^2$; $E_{cm} = 29\ KN/mm^2$

Reinforcing steel

$S - 300$, $f_{yk} = 300\ N/mm^2$, $E_s = 210\ K\,N/mm^2$,

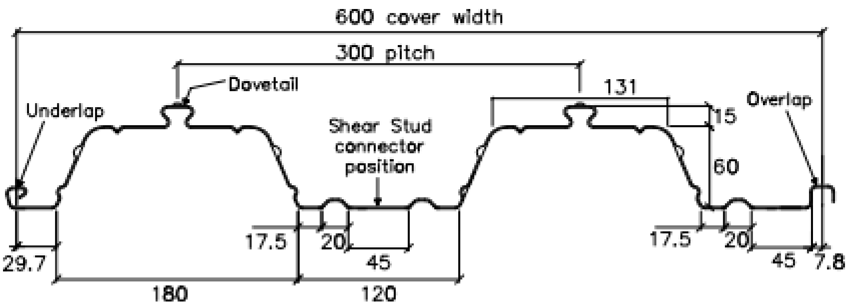
Structural steel

Steel grade Fe 510, characteristic yield strength; $f_y = 355\ N/mm^2 (t \leq 40mm)$

Modulus of elasticity $E_a = 210\ KN/mm^2$

Profile sheeting

ComFlor 60 deck profile is selected for modeling of profile sheeting.



Partial safety factor

Structural steel $\gamma_a = 1.10$

Concrete $\gamma_c = 1.50$

Reinforcing steel $\gamma_s = 1.15$

Design assumption

Full shear connection is assumed in this design works.

Imposed/live Load=2.0 kN/m^2 (for General Loading case)

Partition Load =1.5 kN/m^2 Beam spacing: b=2m

Floor finish Load =0.5 kN/m^2 Construction: Unsupported

Construction Load =0.5 kN/m^2