JIMMA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
JIMMA INSTITUTE OF TECHNOLOGY
FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING
STRUCTURAL ENGINEERING STREAM

Comparative Study of Reinforced Concrete, Structural Steel and Steel Concrete Composite Buildings

A Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Structural Engineering

By
Masresha Mamo

January, 2020
Jimma, Ethiopia

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OMPARATIVE STUDY OF REINFORCED CONCRETE, STRUCTURAL STEEL AND STEEL CONCRETE COMPOSITE BUILDINGS

MASRESHA MAMO

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DECLARATION

I hereby declare that this thesis is the original work of mine and has not been presented elsewhere yet.

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ABSTRACT

The majority of structural building structures are designed and constructed in reinforced concrete which are mainly depends upon availability of the constituent materials and the level of skill required in construction, as well as the practicality of design codes.

Reinforced concrete building used high material usage because of their increased dead load, hazardous formwork. The use of steel in construction industry is very low in Ethiopia compared to many developing countries. There is a great potential for increasing the volume of structural steel in construction, especially in the current development. In Ethiopia, structural steel is not using as an alternative construction material and not using it where it is economical in a heavy loss for the country. Steel-concrete composite construction has gained large acceptance all over the world as a substitute for pure steel and pure concrete construction. In Ethiopia, many consulting engineers are reluctant to accept the use of composite steel-concrete structure because of its unfamiliarity and complexity in its analysis and design. However, this approach is a new concept for construction industry.

This research discusses comparative study of reinforced concrete, structural steel and steel-concrete composite building structures by analysis of G+5, G+8 and G+11 modeling. Modal response spectrum analysis for seismic zone III was carried out based on ES EN-8 using packaged software ETABS 2016. The span length, interstorey height and loading systems was kept constant. The main parameter discussed on this study are the sory response such as storey shear, storey drift, story displacement, modal frequency and time period; construction costs such as direct cost; and self-weight of the buildings.

As per the comparisons made, the steel-concrete composite building showed a reduction in base shear, story drift and story displacement for G+5, G+8 and G+11 as compared to the reinforced concrete and structural steel building. And also the steel-concrete composite building showed a reduction in modal period compared to the reinforced concrete and structural steel building for G+5; increased compared to the reinforced concrete and reduced compared to structural steel building for G+8 and G+11. The reinforced concrete building showed a reduction in modal frequency for G+5, G+8 and G+11 compared to the steel-concrete composite and structural steel building. The structural steel building showed a lighter in self-weight for G+5, G+8 and G+11 as compared to reinforced concrete and steel-concrete composite building whereas steel-concrete composite building is lighter compared to reinforced concrete building at G+5, G+8 and G+11. The steel-concrete composite building showed a reduction in direct cost as compared to the reinforced concrete and structural steel building for G+8 and G+11, but it showed an increased in direct cost compared to reinforced concrete building for G+8 whereas reinforced concrete building is cheaper compared to structural steel building at G+5, G+8 and G+11.

Key Words: *Reinforced concrete, structural steel, composite, Modal analysis and ETABS*

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ACRONYM

CCFT	Circular Concrete Filled Tubes
SCFT	Steel Concrete Filled Tubes
CQC	Complete Quadratic Combination
DCH	High Ductility Class
DCM	Medium Ductility Class
EBCS	Ethiopian Building Code Standard
ETABS	Engineering Three-D Analysis Building System
ES EN	Ethiopian Standard Europe Norm
RCFT	Rectangular Concrete Filled Tubes
RS	Response Spectrum
RCC	Reinforced Cement Concrete
SLS	Serviceability Limit States
ULS	Ultimate Limit States

DEFINITION OF TERMS

Base Shear: the maximum expected lateral force that occur due to seismic ground motion at the base of a structure.

Beam: structural element subjected mainly to transverse loads and to a normalized axial force.

Column: structural element, supporting gravity loads by axial compression or subjected to a normalized design axial force.

Dual System: structural system in which support for the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed to in part by the frame system and in part by structural walls, coupled or uncoupled.

Frame-Equivalent Dual System: Dual system in which the shear resistance of the frame system at the building base is greater than 50% of the total shear resistance of the whole structural system.

Frame System: structural system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system.

Mass Structures: are solid structures which resist the loads acting on them by their own weight.

Self-weight: is the total weight of the skeletal part of the frame structure itself.

Shell Structures: are structures which transmit loads mainly due to their geometry. The geometry that enables them to sustain loads is their curvature.

Story Displacement: is total displacement of i^{th} story with respect to ground or base and there is maximum permissible limit prescribed in specified codes for buildings.

Story Drift: is defined as ratio of displacement of two consecutive floors to height of that floor. It is usually interpreted as inter-storey drift of one level to another level above or below.

Wall-Equivalent Dual System: dual system in which the shear resistance of the walls at the building base is higher than 50% of the total seismic resistance of the whole structural system.

Time Period: the time required to complete one complete cycle of vibration.

Structure: refers to a system of connected parts used to support loads and to transmit loads from the point of application to the point of support.

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

In the past, structural engineers had the choice of masonry building in order to meet different needs such as housing, commerce sites, temples and others. Over the centuries the structures have evolved considerably. If we go back to the previous time, we can find great and famous structures that are still standing up in different parts of the world. The problem with these structures is that in the vast majority of cases were oversized, resulting in higher than necessary resistance with its associated cost. Recently, reinforced concrete has been the most used material in the world for all type of constructions. For this reason, it can be considered as a classical solution. That is due to its advantageous characteristics: relative ease of construction, high compression resistance, good seismic and vibration behavior, material availability in the nature (Cantons, 2016).

Today, the trend of going towards composite structure and steel framed structure has started and growing in us. But the failure of many these buildings due to earthquake have forced engineer to look for the alternative method of construction. Use of composite or hybrid material remains the particular interest, due to its significant potential in improving the overall performance through modest changes in manufacturing and constructional technologies. In Ethiopia, many consulting engineers are reluctant to accept the use of steel-concrete composite structure because of its unfamiliarity and complexity in analysis and design. For these reasons, it is interesting to compare this classical solution to other solutions than can be perfectly carried out in the same project, with the same boundary conditions and with the same shape. In this paper, besides reinforced concrete, structural steel and steel concrete composite will be taken into account to compare their behavior.

Steel structures are chosen in structural design due to its high resistance per weight unit, which allows light constructions and, in consequence, more open spaces with less number of supports and smaller dimensions on the structural elements. Furthermore, steel structures show high ductility, which is very important to achieve high deformation without reaching the failure point. Focusing in the construction process, these structures can be built in less time than reinforced concrete ones, which generates a direct impact both in manpower needs as in time and cost reduction (Singh, 2017).

A member is said composite, when a concrete member and steel component are used together in such a way that they experience transfer of forces and moments in them, in order to take full advantages of steel in tension and concrete in compression are utilized together to get best capabilities of both of these. Steel-concrete composite construction means steel section encased in concrete for columns and the concrete slab or profiled deck slab is connected to the steel beam with the help of mechanical shear connectors so that they act as a single unit. It can also be defined as the structures in which composite sections made up of two different types of materials such as steel and concrete are used for beams, and columns. the structural design of office buildings has been carried out taking into account the three different structural materials mentioned above.

In the present work, comparative study of reinforced concrete, structural steel and steel-concrete composite building is made. Comparative study includes story shear, storey displacements, storey drifts, modal period, modal frequency, self-weight and cost of structural material.

1.2 Statement of the Problem

Reinforced concrete building used high material usage because of their increased dead load, hazardous formwork. The use of steel in construction industry is very low in Ethiopia compared to many developing countries. There is a great potential for increasing the volume of structural steel in construction, especially in the current development. In Ethiopia, structural steel is not using as an alternative construction material and not using it where it is economical in a heavy loss for the country. Steel-concrete composite building has gained large acceptance all over the world as a substitute for structural steel and reinforced concrete building. In Ethiopia, many consulting engineers are reluctant to accept the use of composite steel-concrete structure because of its unfamiliarity and complexity in its analysis and design. However, this approach is a new concept for construction industry (Cunningham, 2013).

It is being practical to choose types of structural buildings for different criteria after assuring structural safety. Due to this cause, safety and serviceability of the building is analyzing to make it stable and sustainable throughout its design life. A structural design is executed in such a way that the building is remain fit with appropriate degrees of reliability. Therefore, structural designer focus not only on structural safety and serviceability with durability but also to optimize the cost expended in building structure.

In Ethiopia, the reinforced concrete structure is common to construct compared to other structural systems such as structural steel and composite structures. But, there is no comparative analysis available to convince designers to adopt other option. This research is therefore aimed to provide such information.

1.3 Objective of the Study

1.3.1 General Objective

- The aim of this thesis is comparative study of reinforced concrete, structural steel and steel-concrete composite buildings at different number of storey.

1.3.2 Specific Objective

- To verify and compare the structural behaviors of reinforced concrete, structural steel and steel-concrete composite buildings such as story shear, story drift, story displacement, time period and modal frequency.
- To compare the self-weight of reinforced concrete, structural steel and steel-concrete composite buildings.
- To determine the direct cost analysis of reinforced concrete, structural steel and steel-concrete composite buildings.

1.4 Significance of the Study

The result of this study is expected to provide data and information that helps for stake holders in construction industry such as designer, architects, contractors and students to know the most feasible and economical structural material easily for building structure on the behaviours of structural building.

Additionally, it can be useful to address other sectors such as the public sector, public authorities and research institutes for magnification the position of consideration to sustainability and using sustainable building materials as an important way to be considered in building construction.

The issue of this study is important to the vast majority of construction clients. The study identified that the designer’s priorities in relation to safety, serviceability and economy constraints are key factors in forming an effective brief. The appointment of the design team is shown to be a key decision in the process of developing this brief and determining the nature, and hence the behavior of the structure.

1.5 Scope and Limitation of the Study

The scope of this study is focus on the comparative study of reinforced concrete, structural steel and steel-concrete composite buildings. To do this, a symmetrical plan layout of 4-bay by 3-bay for a purpose of office building having G+5, G+8 and G+11 with a total storey height of 21m, 30m and 39m, respectively is prepared. The interstorey height including bottom storey hight is 3.0 meter and column spacing is 6 meter in X and 5 meter in Y-direction. Following the plan, reinforced concrete building frame system with solid slab is formed. Using the same plan, structural steel and steel-concrete composite moment resisting frame system with steel girder beam and composite floor system topping reinforced concrete slab on corrugated steel deck is formed.

Considering the more rigorous detailing requirements associated with high ductility class DCH that are more difficult to implement, medium ductility class DCM is chosen for the seismic design of the structure. Dynamic analysis such that modal response spectrum analysis is performed. Horizontal displacement due to earthquake was taken into account. For structural steel building columns and beams are steel I-section. In case of steel-concrete composite building filled tube composite column are considered whereas steel I-section are considered for beam. Unions are considered as rigid joints to be able to create the frame system. Span length, interstorey height and loading of the building are constant. Structural behaviors such as story shear, story drift, story displacement and direct cost analysis and self-weight of the structure are observed to evaluate the better structural system for the selected office building. Construction costs such as transport cost, foundation cost, finishing cost, electrician and sanitary cost are not considered in the cost analysis.

CHAPTER TWO

RELATED LITERATURE REVIEW

2.1. Structural Buildings

2.1.1 Reinforced Concrete Building

Reinforced concrete is one of the main building materials utilized in structural design. As a complex material, it consists of steel reinforcing bars fixed in concrete. As clear as it is, the three fundamental elements of simple structural concrete are cement, water and a big volume of moveable aggregate (sand and gravel). Concrete is somehow considered a complex material, and its use ties with many concerns, such as finishing, shaping, curing and reinforcing of the cast material. Normal cementations concrete has some features, most importantly its resistance to wetness, insects, fire, rot and wear and also its low majority cost. Being shapeless in its original mixed situation, it can be made kind of forms (Arya, 2009).

In most structural applications, tension stresses of considerable magnitude have to be accommodated. For this purpose, steel reinforcements (rods, bars or wires) are embedded in the concrete at the time of casting, so forming the composite material known as reinforced concrete. The reinforcements being steel have a high tensile strength and by judicious design they can be so disposed in the concrete as to be available to take all tensile stresses wherever these occur, whether as a result of direct tension forces or bending, shear or torsion. In this way full advantage is taken of the strength of the concrete in the compression zones of the structure, and the reinforcements provide the tensile strength which unreinforced concrete lacks. Reinforcements suitably disposed; can also serve to increase the strength of concrete members in compression, as when as control the effects of shrinkage and temperature changes (AZAD, 2012).

Figure 2. 1: Reinforced Concrete Structure with Beams and Columns (Karimizadeh,2015)

2.1.2 Structural Steel Building

Steel is used in a variety of forms in nearly every building. It is also one of the strongest, generally the most reliable in its quality control. Steel necessitates the mining of limestone, magnesium, iron ore, coal and other trace essentials. The usage of steel as one of the main structural building material in the constructions in the late nineteenth century for the reason that low-cost approaches used for construction is on a huge scale were industrialized. Steel is a member of metals family, which has design flexibility, and sensible cost with compare to reinforced concrete (Karimizadeh, 2015).

However, Steel is the most recyclable material in the world. It can be recycled over and over again without losing its properties, saving natural resources and reducing construction waste in landfills, thus minimizing two major problems faced by the construction sector. And also, steel construction is classified in the dry construction method; therefore, it can be reduced pollution in during its construction period.

Figure 2. 2: The Steel Building Structure with Beams and Columns (Karimizadeh, 2015)

2.1.3 Steel-concrete composite Building

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 SRCS) 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 (BASSA, 2016).

2.2. Advantage of Structural Buildings

2.2.1 Advantages of Reinforced Concrete Building

The list below provides some of the main advantages of reinforced concrete as a structural building material (BASSA, 2016).

- Reinforced concrete has a high compressive strength compared to other building materials.
- Due to the provided reinforcement, reinforced concrete can also withstand a good amount tensile stress.
- Fire and weather resistance of reinforced concrete is fair.
- The reinforced concrete building system is more durable than any other building system.
- Reinforced concrete, as a fluid material in the beginning, can be economically molded into a nearly limitless range of shapes.
- The maintenance cost of reinforced concrete is very low.
- In structure like footings, dams, piers etc. reinforced concrete is the most economical construction material.
- It acts like a rigid member with minimum deflection.
- Compared to the use of steel in structure, reinforced concrete requires less skilled labor for the erection of structure.

2.2.2 Advantages of Structural Steel Building

The list below provides some of the main advantages of steel as a structural building material (Karimizadeh, 2015).

- Steel structures are quite quick to be constructed which usually outcomes in faster economic payment.
- Steel has a big strength/weight ratio. Thus, the weight of steel structure is reasonably low.
- The properties of steel can be predicted quite confidently.
- There are minimum construction concerns and worker mistakes in the steel construction.

- The prefabrication of steel structural material delivers a safer, reduces the pollution and cleaner working on the building construction site.
- They are easy to repair and there is easy access to damaged parts to repair them if necessary. These structures can be repaired easily and speedily.
- Adaptation of manufactured. Steel is more flexible in context of architecture approach in comparison with other materials. Prefabricated and mass production of steel is extremely suitable.
- Steel is a greatly recyclable building material. Likewise, it is reusable after being taken down from a previous building structure.

2.2.3 Advantage of Steel-concrete Composite Building

BASSA (2016) 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.
- 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 com0posite 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.

2.3 Floor Systems of Structural Buildings

An appropriate floor system is an important factor in the overall economy of the building. Some of the factors that influence the choice of floor system are architectural. Other factors affecting the choice of floor system are related to its intended structural performance, such as whether it is to participate in the lateral load-resisting system, and to its construction, for example, whether there is urgency in the speed of erection (SADAT, 2014).

2.3.1 Reinforced Concrete Floor Systems

Different types of RC floor systems are being used for building construction. Some typically used floor systems are described below in brief.

The slab shown in Figure 2.3 spans two ways between orthogonal sets of beams that transfer the load to the columns or walls. The two-way system allows a thinner slab and is economical in concrete and reinforcement. It is also compatible with a lateral load resisting rigid-frame structure. The maximum length-to-width ratio for a slab to be effective in two directions is approximately (SADAT, 2014).

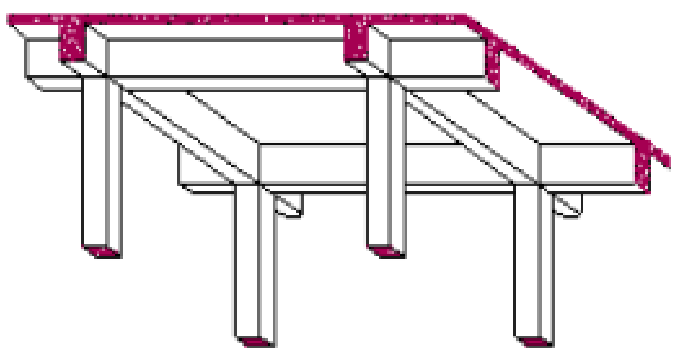


Figure 2. 3: Two-Way Solid Slab and Beam (SADAT, 2014)

2.3.2 Composite Floor Systems of Steel Framing

The use of steel members to support a concrete floor slab offers the possibility of composite construction in which the steel members are joined to the slab by shear connectors so that the slab serves as a compression flange. In one simple and constructional convenient slab system, steel decking, which in often used to act merely as rapidly erected permanent framework for a bar-reinforced slab, serves also as the reinforcement as the concrete slab in a composite role, using thicker wall sections with indentations or protrusions for shear connectors. Slabs may also be designed to act compositely with the supporting beams by the more usual forms of stud, angle, or channel shear connectors, so that the slab alone spans the short distance between the beams while the compositely acting slab and beam provide the supporting system. The further combination of a concrete slab on metal decking with shear connectors welded through to the supporting beam or truss is an efficient floor system (SADAT, 2014).

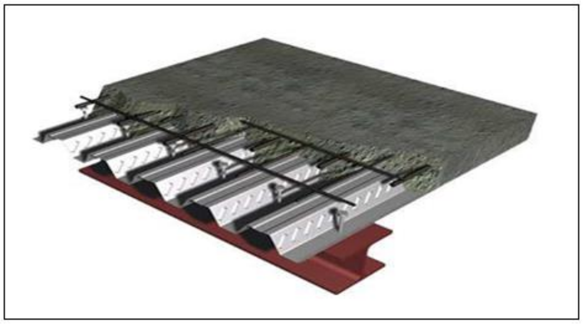


Figure 2. 4: Cross-Section of Composite Floor Systems of Steel Framing (SADAT, 2014)

2.4 Structural Behaviors and Comparison

2.4.1 Base Shear

A G+5 story building in seismic zone IV is analyzed by (Ganwani & Jamkar, 2016) and seismic performances of both reinforced concrete and composite materials are compared. It is found that the base shear is more in reinforced concrete as compared to the composite frame due to the more seismic weight of reinforced concrete frame as represented by Figure 2.5.

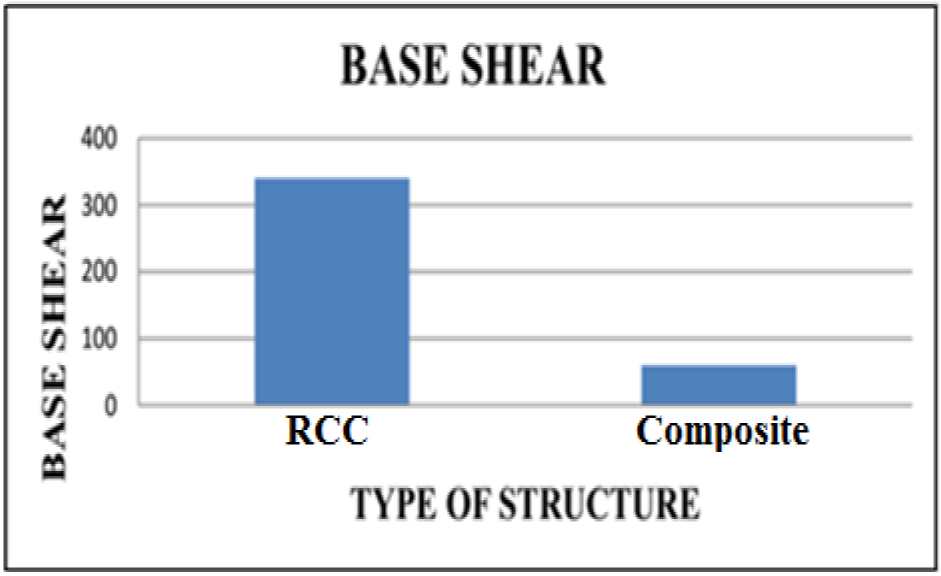


Figure 2. 5: Base Shear vs. Building Storey *(Ganwani & Jamkar, 2016)*

Mahajan and Kalurkar discussed on behavior of RCC and composite structure under seismic loads as the base shear is the horizontal reaction to the earthquake forces and horizontal forces results from the storey weight. Storey weight includes the self-weight of the structure also; hence in the reinforced cement concrete model the self-weight is seems to be the more and hence maximizing the earthquake forces which results in the maximum base shear. As we have the static formula for base shear and base shear is the direct function of the seismic weight therefore naturally base shear is more in the case of RCC structure. The analysis is carried out as per code IS:1893-2002 and the results of base reactions directly shows that base shear in longitudinal and in transverse direction is less in composite structure than RCC structure. The base shear is the basic parameter for deciding the earthquake resistant structure. To make the structure safe, the base shear should be kept as low as possible. The base shear in Composite structure is reduced by 34.46% in X Direction and 46.6% in Y direction.

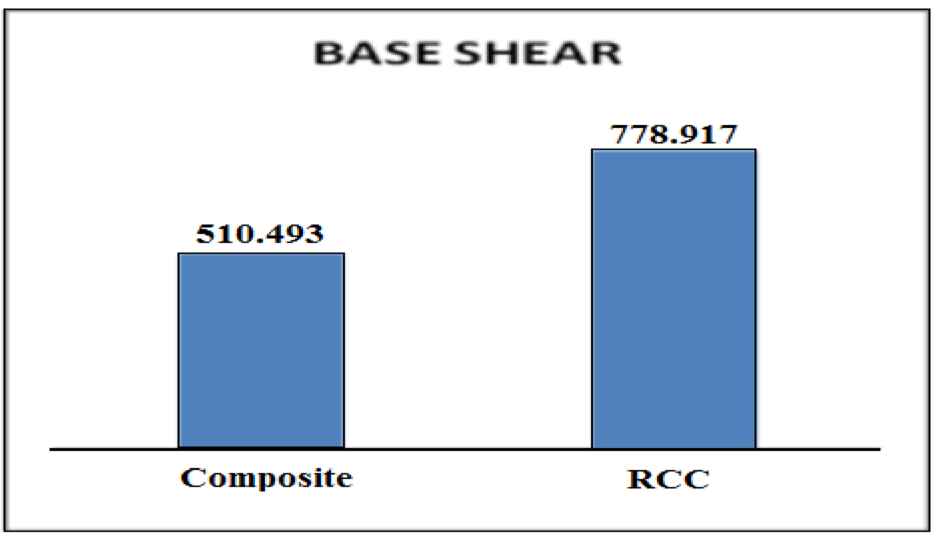


Figure 2. 6: Base Shear Comparison *(Mahajan & Kalurkar, 2016)*

Warade and Salunke (2013) concluded that the base shear is maximum in case of RCC and minimum in case of steel. The composite found to have more value of base shear than steel but very much less than RCC. Multi-level car parking structure is analyzed using seismic co-efficient method. Total 15 models were modeled with RCC, composite and steel, 5 for each material i.e. (G+6, G+7, G+8, G+9, G+10). Reason being as the weight increases, base shear values are also boosted as represented by Figure 2.7.

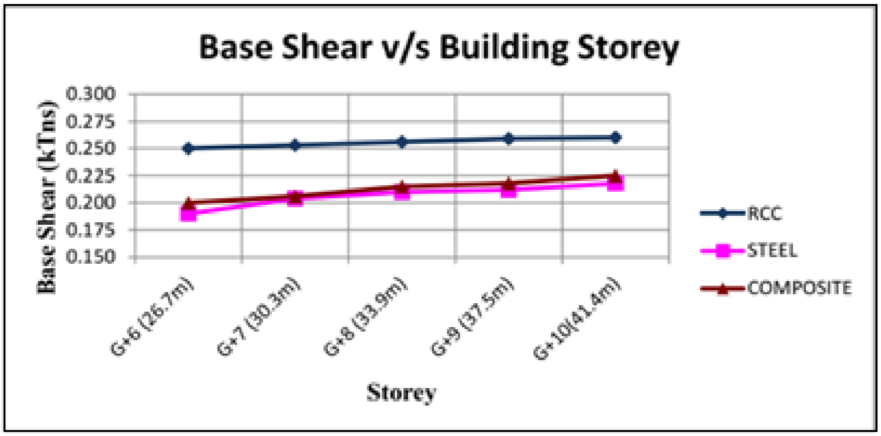


Figure 2. 7: Base Shear vs. Type of Structure *(Warade & Salunke , 2013)*

2.4.2 Storey Drift

Kolhe et al. (2015) published storey drift results concluding that the composite frames have lowest storey drift values as compared to the steel frames and the only justification for that is the stiffness of composite frame. The variability in storey drift values in X and Y directions is due to the column orientation which leads to the different moments of inertia (Kolhe , et al., 2015).

In a comparative study of Steel and RCC for G+6 and G+10 storey structures (Sangave , et al., 2015) have analyzed for seismic zone V in ETABS. The work concludes that within permissible limits, RCC structures have less values of storey drift in comparison with steel structures. So stiffness is playing the lead role in storey drift factor.

Mohite et al. (2015) have analyzed of B+G+11 storey commercial building in their research. The building is located at Kolhapur which comes under seismic zone III. The RCC as well as steel-concrete composite frame is considered for equivalent static analysis of building. The storey drift in composite structure resulted in less values in comparison to the RCC structure. So due to increase in stiffness values of the structure the storey drift values goes down and results in good seismic performance of the structure (Mohite , et al., 2015).

2.4.3 Story Displacement

Patel and Thakkar have done on comparison of ten storey CFT, RCC and Steel building. load intensity in all three types of buildings is kept nearly same for comparison of various parameters and behavior of CFT, RCC and Steel building. When graph of mode shape v/s Time period for a ten storey CFT structure was plotted, first three mode shapes were found in Y direction, X-direction and XY direction. The time period for 1st mode in Y-direction was 1.332 second while for X-direction was 1.316 second. First mode in Y-direction had maximum displacement and was governing. As shown in Table.3, percentage reduction in time period of CFT building is 44.1% and 17.4% with compared to RCC and Steel building respectively. Also base shear due to earth quake load and load carrying capacity of CFT building is found to be higher than RCC by 15.2%, while for steel by 6.8%.

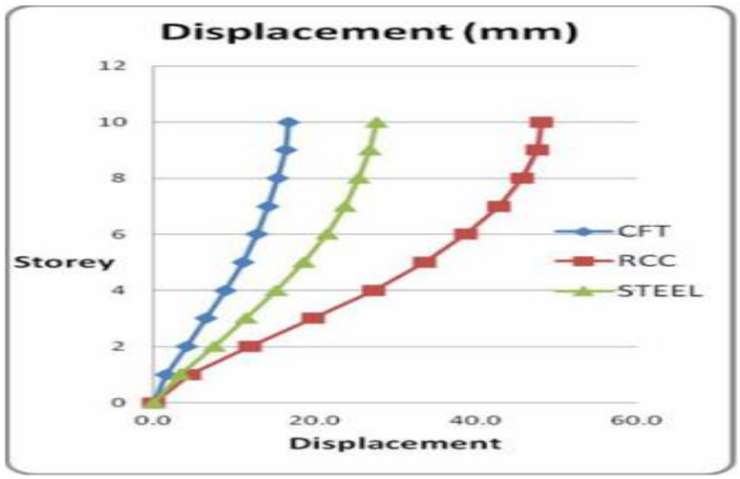


Figure 2. 8: Storey Displacement (Patel & Thakkar, 2013)

Abrol et al. (2017) studied on seismic analysis of reinforced concrete and composite structures maximum lateral displacements values of maximum displacements along X and Y for reinforced concrete and composite structures it is clear that the nodal displacements in a composite structure, by both the methods of seismic analysis, compared to a reinforced concrete structure in all the three global directions are less which is due to the higher stiffness of members in a composite structure compared to an reinforced concrete structure. Fig 2.9 show the variation of displacements in reinforced concrete and composite structures. Percentage difference between the both is 20%.

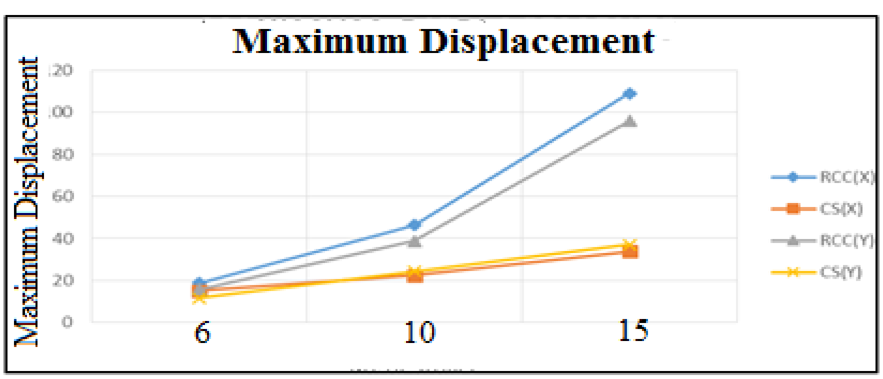


Figure 2. 9: Maximum Displacements along X and Y directions (Rakesh, et al., 2017)

2.4.4 Time Period

Kumar and Rao, (2016) considered on seismic analysis of composite structures and its comparison with reinforced concrete structures. The time period of the structure is reduced from composite to RCC as shown. The time period is reduced from 1.214 s to 0.852 s in low rise (5 story) structure,1.954 s to 1.242 s in medium rise (10 story) structure, and 2.537 s to 1.882 s in high rise (15 story) structure. They conclude that through E-TABS values of time period of the structures are extracted. The maximum time period is for composite structures, it means it is more flexible to oscillate back and forth when lateral force act on the building and RCC structures has least time period which says that it is less flexible.

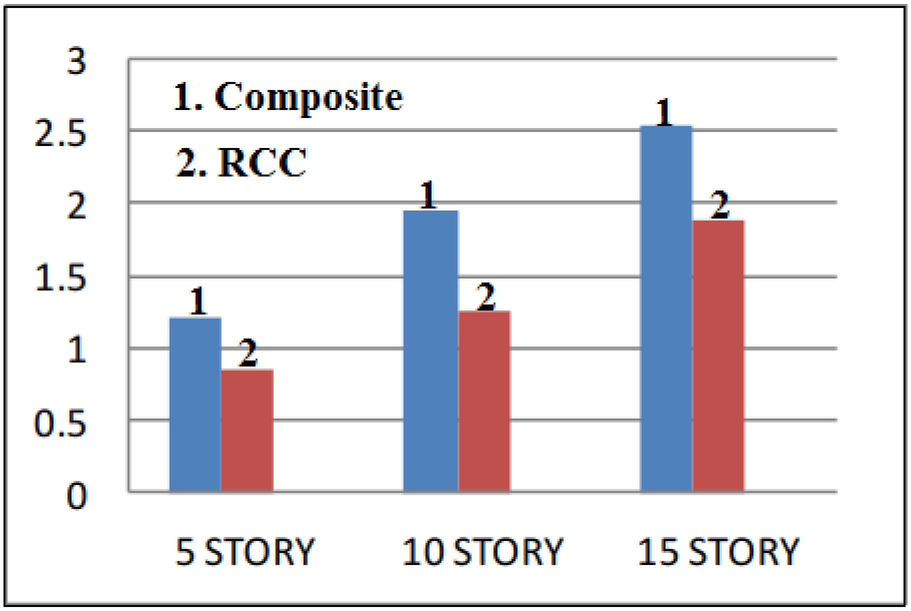


Figure 2. 10: Maximum Time Period (kumar & Rao, 2016)

Mahajan and Kalurkar discussed on behavior of RCC and composite structure under seismic loads it is observed that for both the structures time period continuously decreases and correspondingly the frequency increases from 1st node to 12th node. The time period of composite structure is more than RCC structure and at the same time frequency is more in RCC structure than composite structure. The time period of composite structure is increased by 19% to 25% and on the other hand frequency is decreased by 22% to 24%. The reduction in stiffness of composite structure results in increase of time period and decrease in frequency.

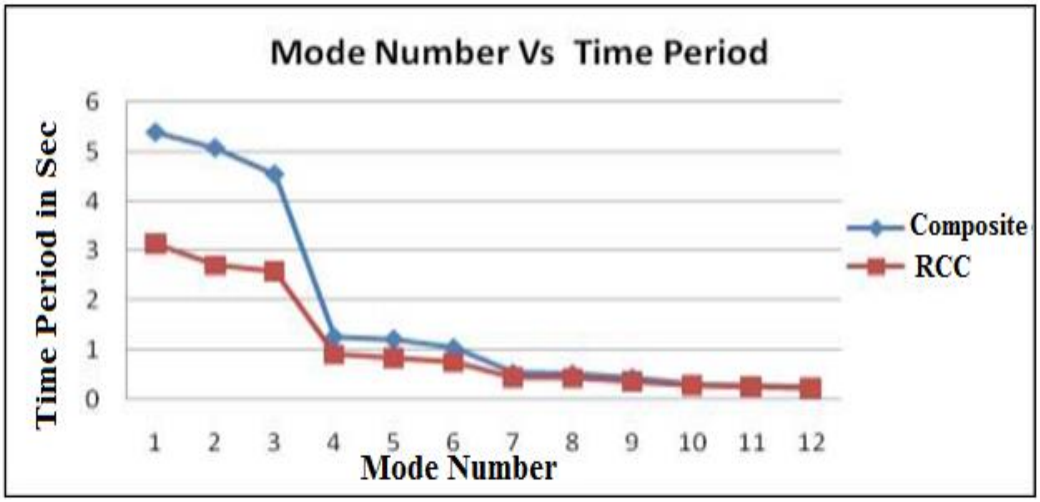


Figure 2. 11: Time Period (Mahajan & Kalurkar, 2016)

2.4.5 Self-weight

Kumar and Rao, (2016) conclude that the dead weight of the structure is reduced from RCC to composite as shown. The dead weight of the structure is reduced from 9588 KN to 6840 KN in low rise structure, 25155.06 KN to 14208.07 KN in medium rise structure and 36535.493 KN to 21921.34 KN in high rise structure. Due to the light weight of the structure the composite structures are less susceptible against seismic forces acting on the structure.

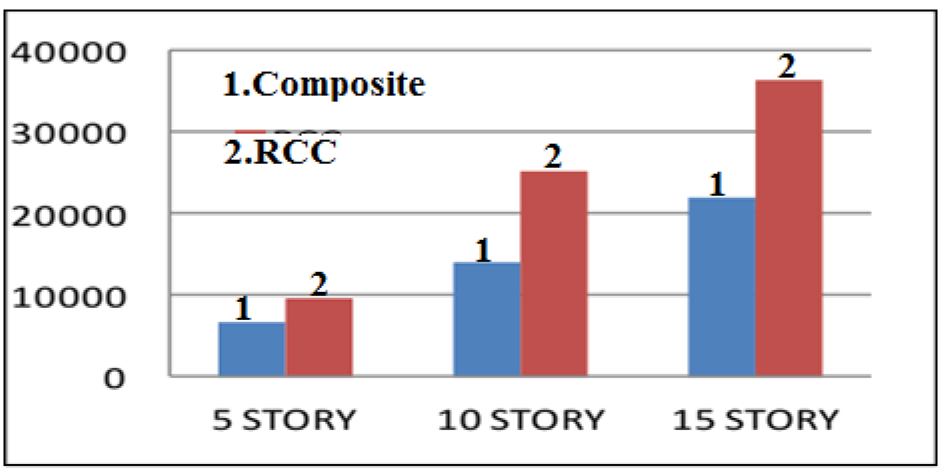


Figure 2. 12: Dead weight of structure *(kumar & Rao, 2016)*

Zaveri et al. (2016) considered a review on the comparative study of steel, RCC and composite building dead load of composite is less than RCC and more than steel. As the results, show the composite option is better than RCC because composite option for high rise building is best suited. Weight of composite structure is quite low as compared to RCC structure. The reduction in the total weight of the Composite framed structure for 6 storey, 10 storey and 15 storey are 22.64%, 24.19% and 28.95% with respect to RCC frame Structure. As the dead weight of a composite structure is less compared to an RCC structure, it is subjected to less amount of forces induced due to the earthquake (Zaveri , et al., 2016).

Mahajan and Kalurkar discussed on behavior of RCC and composite structure under seismic loads weight of any structure is depends upon its components and material used in construction. Weight should be kept as low as possible to reduce the earthquake effect. In order to find out dead weight and make it a lighter structure we have studied the weight of all structural members in composite steel-concrete and RCC building. From the following figure it is seen that composite structure is having less weight by 35.05 % comparing to RCC. The dead weight of composite structure is found to be 30 % to 35% less than RCC structure and hence the seismic forces are reduced by 30% to 35%. As the weight of the structure reduces it attract comparatively less earthquake forces than RCC structure. This will add to further reduction in axial forces, shear forces and bending moment as compared to RCC structure.

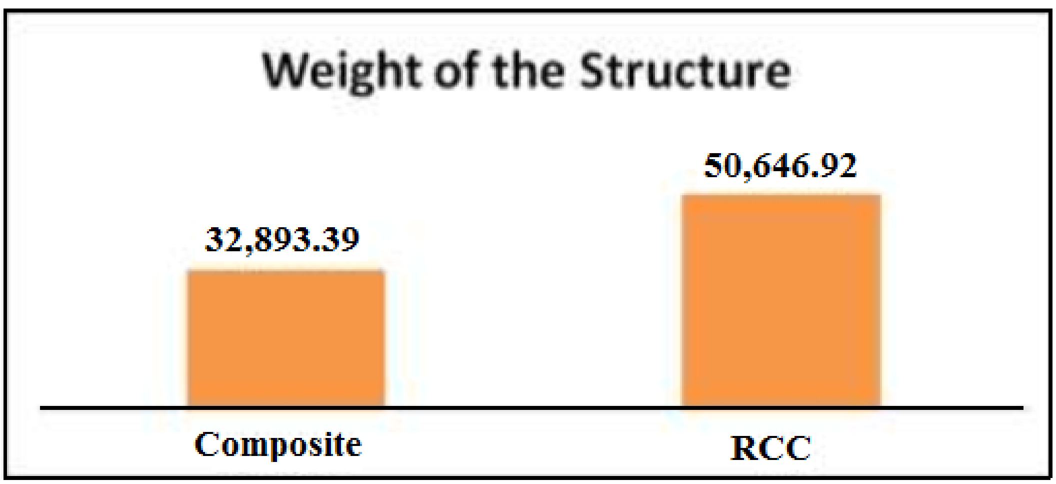


Figure 2. 13: Dead weight of the structure *(Mahajan & Kalurkar, 2016)*

Charantimath et al. (2014) has analyzed three buildings of 10, 20 and 30 storey having dimensions of 30m X 24m. The weight comparison between RCC and composite building is represented in graphical form as shown in Figure 3.3 which shows that composite building is lighter than RCC building. As the no. of stories increases, the difference in weight is also increases.

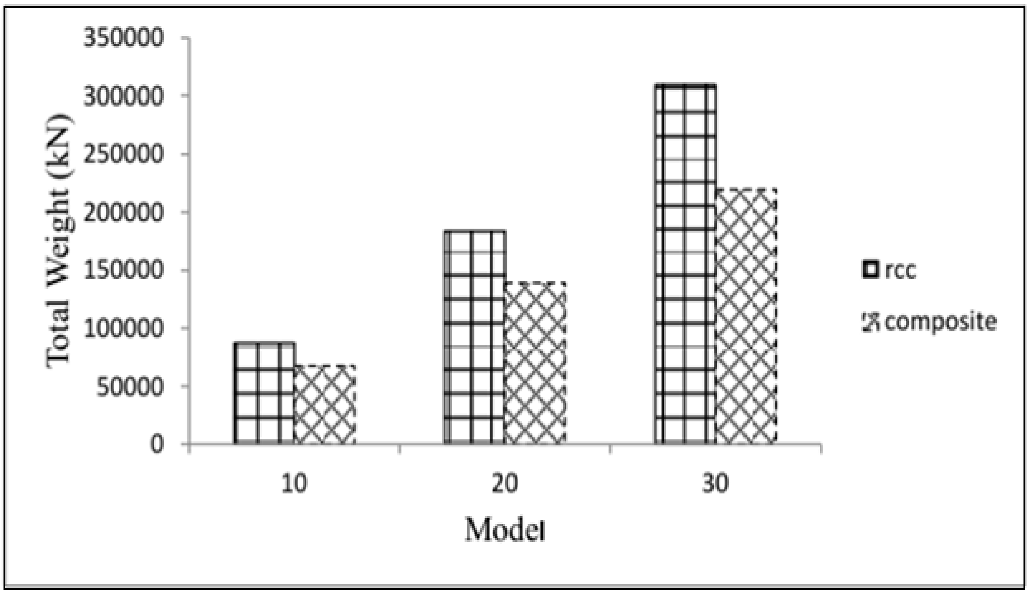


Figure 2. 14: Comparison of Total wt. v/s RCC and Composite *(Charantimath , et al., 2014)*

Warade and Salunke (2013) has compared different buildings, their work can be represented as the table of self-weights (in kTns) shown in Table-2.1.

Table 2. 1: Comparison for Self-weight of the Structure of the Building *(Warade & Salunke , 2013)*

STOREY	G+6(26.7m)	G+7(30.3m)	G+8(33.9m)	G+9(37.5m)	G+10(41.4m)
RCC	10.53	11.86	13.19	14.52	15.86
STEEL	7.74	8.73	9.72	10.69	11.68
COMPOSITE	8.04	9.06	10.07	11.09	12.11

Panchal et al. (2011) has compared self-weight for the RCC, Steel and composite G+30 storey commercial building. RCC structure found to have more weight than other two and steel having least weight. Their work can be represented in table 2.2.

Table 2. 2: Self-weight of building with respect to material *(D. R Panchal & Marathe, 2011)*

Building Type	RCC	STEEL	COMPOSITE
Self -weight (kN)	368168	248397	256354

2.4.6 Cost

Sattainathan et al. (2015) consider on comparative study of cost and time evaluation in RCC, steel and composite high rise building and the result shows the use of concrete filled steel tube columns has been consistently applied in the design of tall buildings as they provide considerable economy in comparison with conventional steel building. Also performance wise result good compared to RCC and steel building.

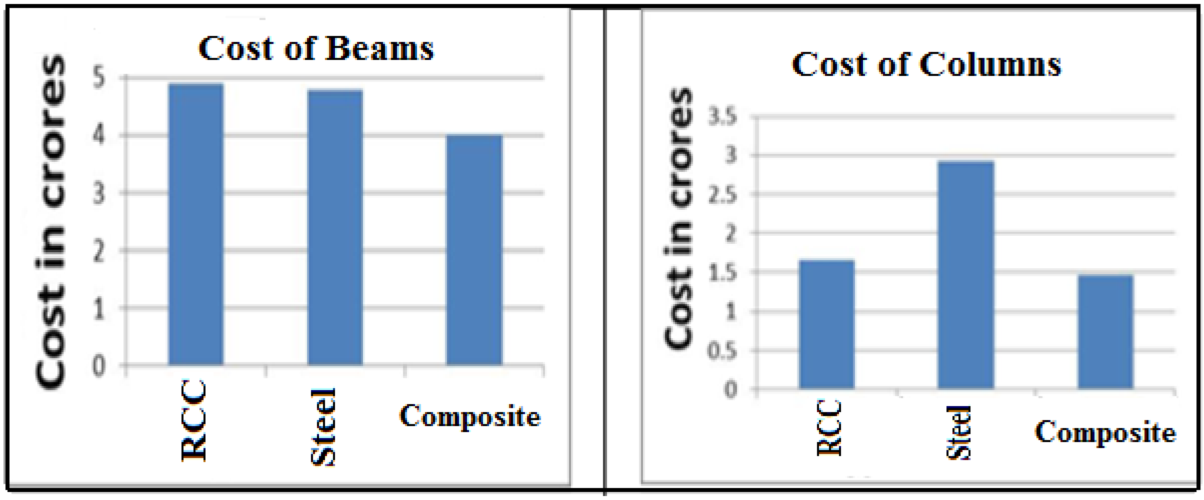


Figure 2. 15: Cost comparison in beams and columns *(Sattainathan, et al., 2015)*

Begum (2013) provides on cost analysis of steel concrete composite structures provides 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, he 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 and 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 and so on. Steel-concrete composite frame system can be an economically viable solution for high-rise buildings in Bangladesh (Begum, 2013).

They took four various multistoried commercial buildings such that G+12, G+16, G+20, G+24 and analyzed 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 (FY) 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 RC 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 (Shweta & Wagh , 2014). He pointed out that the main economy in using profiled deck is achieved due to speed in construction. He notified that normally 2.5 to 4.0 m 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 costlier, reduction in direct costs of steel composite structure resulting from speedy erection will make Steel-concrete composite structure economically viable. Finally, he showed that above 15 stories, steel concrete composite structure is cost effective as shown in figure-2.16

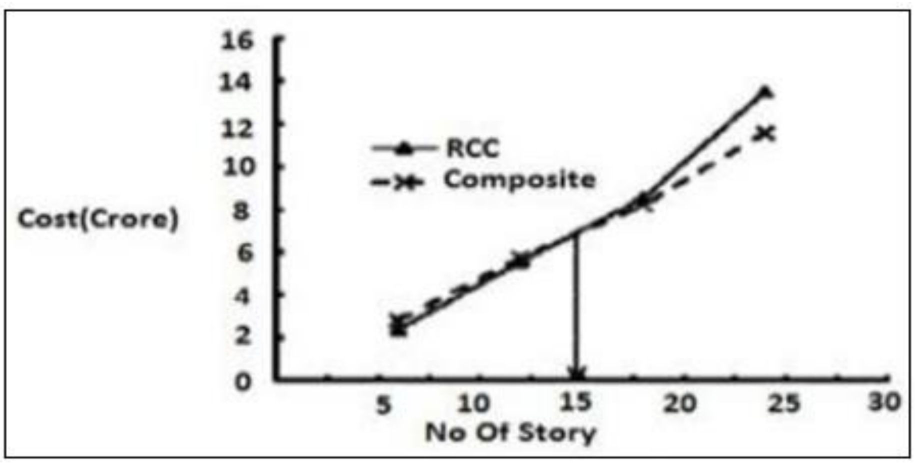
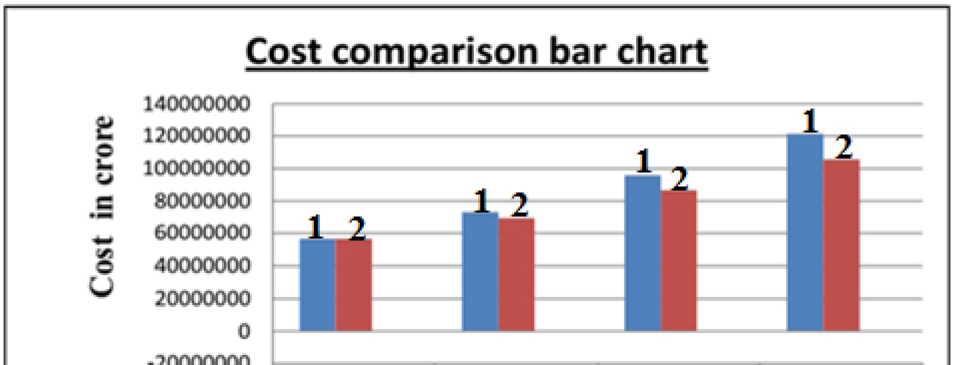


Figure 2. 16: Cost vs Number of Story for Composite and RCC Building (Aniket Sijaria , 2014).

Wagh et al. (2014) have done the comparative study of RCC and steel-concrete composite structures. The four multistoried buildings are considered. The cost estimation is done using the MS-Excel software and results are compared. The result shows that the composite structure is economical and lighter as compared to RCC construction. The results are shown as bar chart in Figure 2.17.



	G+12	G+16	G+20	G+24
1. R.C.C	56720409	73029883	95776019	121352652
2. Composite	56657375	69397893	86720187	105740009
%age Difference	-0.112	-5.23	-10.44	-14.77

Figure 2. 17: Cost Comparison Bar Chart *(Wagh , et al., 2014)*

Begum et al. (2013) have compared cost of RCC and steel-concrete composite structure and found that cost of steel-concrete composite structure is more in case of low rise buildings up to 15th storey but high for medium rise buildings to high rise buildings the cost of steel-concrete structure is less as compared to RCC structure. The final results show that for the buildings having no. of storey more than 15, steel-concrete composite construction is very economical as compared to the RCC construction. The cost comparison graph is as Figure 2.18.

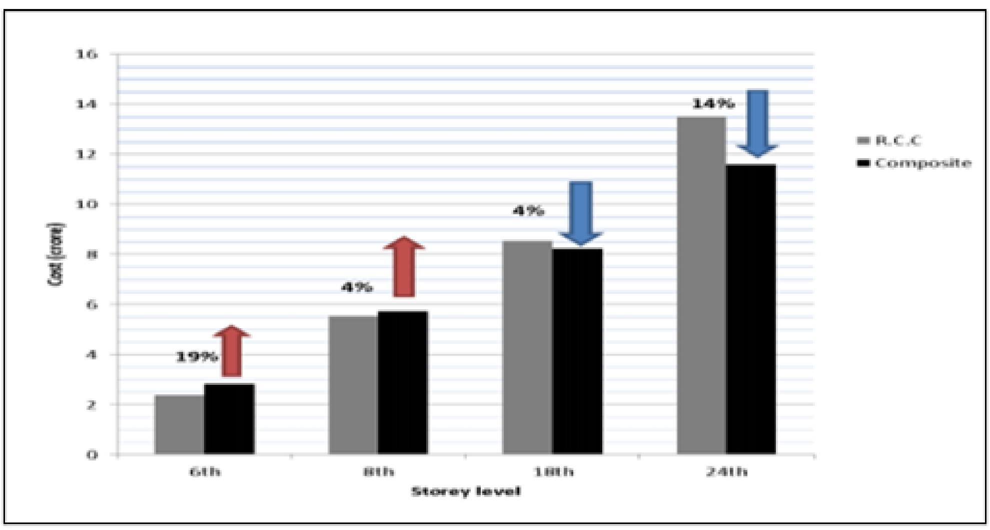


Figure 2. 18: Cost Comparison Bar Chart *(Begum , et al., 2013)*

CHAPTER THREE

RESEARCH METHODOLOGY

In this section, the approaches and techniques of the researcher is presents and describes used to collect data and investigate the research problem. It must be remembered that for analysis and design part the new version of Ethiopian Standard or new European Code has been the reference standards in all cases. Finally, the structural behaviours and the total material usage are recorded to evaluate better structural system in each case.

3.1 Study Area

In this case, the characteristics of the dominant ground type in the city of Addis Ababa, Ethiopia is taken into account. Therefore, the exact location of the building is the type of ground found more frequency in the Gulele sub-city is considered. The type of ground type considered on this zonal area is ground type C which is a profile of deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters. Gulele sub-city is a seismic zonal area located in zone III and situated in North West of Addis Ababa, its geographical coordinates are 9° 3' 46.8" North and 38° 44' 36.96" East.

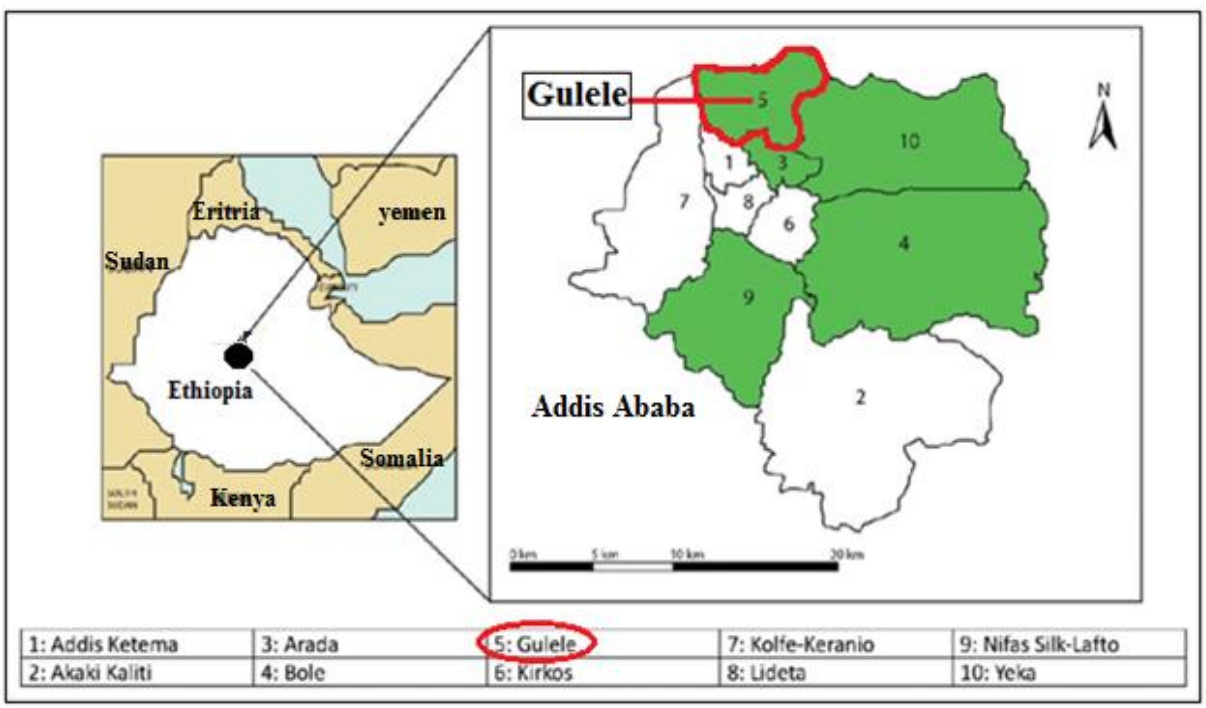


Figure 3. 1: Geographical Map of Addis Ababa, Ethiopia (Google Information, 2019)

3.2 Environmental and Economic Conditions

3.2.1 Environmental Conditions

Environmental conditions playing significant roles to achieve sustainability in structural building materials. Ethiopia has four major seasons: Summer 'Kiremet' (June – August); Autumn 'Tibe' (September – November); Winter 'Bega' (December – February) and Spring 'Belg' (March – May). The coldest month is not always in Winter 'Bega' and the hottest month is not always in Summer 'Kiremet'. Ethiopia lies near the equator where maximum heat from the sun is received. The length of days and nights are almost the same in most regions. The rainfall decreases from the southwest to the northeast. The rainfall is largely concentrated during the summer months of June, July and August. Since, the climate is moderate, except in the lowlands of the Danakil Desert and the Ogaden, which are hot all the year. The highlands are temperate with night frost in the mountains. Therefore, climate condition of Addis Ababa is moderate through a year.

3.2.2 Economic and Social Conditions

Economic and Social Conditions also playing significant roles to achieve sustainability in structural building materials. The level of Ethiopian economy is on growing which requires a great proportion of imports for growing stability towards the development of the economy. Furthermore, Ethiopia owns great sources such as natural sources, and energy. Hence, energy problems can be a start point for growth of the investment and operating cost. Costs for building construction are also moderately high in Ethiopia considering the economic conditions and political sanctions. The key goals of the economic development strategy of Ethiopia can be reaching the highest possible rate of development on economic stability, more sensible supply of local income, and also increasing the life standards by making improvements to the social structure and economic issues. The building construction sector is playing a significant role in the economic and social progresses of the Ethiopia. Furthermore, the building construction sector produces one of the maximum multiplier effects through its wide backward and forward connections with other parts of the economy. Also fortunately, the relative between building construction sector and environment has become a striking part for the current researches in this region.

3.3 Preliminary Structural Consideration

Normally, the structural design of building is a result of certain initial and/or boundary conditions, location of the building, the climate, loads and so on. After the assessment of all those conditions and the combination of these, the designer made an appropriate design of the various structural elements of the building. The same applies to the geometry of the building. In addition, it is necessary to choose the suitable materials to withstand such loads.

3.3.1 Building Description

The building considered here is an office building having G+5, G+8 and G+11 with a total storey height of 21m, 30m and 39m, respectively located in seismic zone III and for earthquake loading, the provisions of the ES EN-8 is considered. The plan of building is shown in fig. 5. The plan dimension of the buildings is 15x24 m, which supposes a useful area of 360 m^2 per story for all structural alternative. Typical interstory height for all structural alternative is 3m whereas bottom story height is 2.5m. The resistant structural scheme is based on stiff frames formed by beams and columns in both directions. This bidirectional configuration creates spans of 5 meters in X-direction and 6 meters in Y-direction. That means there are four frames in X-direction and five frames in Y-direction. The study is carried out on the same building plan for reinforced concrete, structural steel and composite building. The basic loading on all structural alternatives are kept same.

3.3.2 Structural Materials

The materials are the key factor in the structural behavior. The choice of the right materials is vital to ensure the different aspects mentioned above. Each material has different properties that make it interesting when designing a structure. The problem is that often a single material is not enough to meet all the resistant needs of the building. For this reason, today the majority of materials used in construction are composite materials. Thus a composite material that combines all the advantages of each material is achieved that way and allows a good structural behavior.

The most significant case of composite material is reinforced concrete. The concrete itself is already a composite material, which thanks to its different components achieves a high compressive strength and durability. The problem lies in its tensile strength, which is very low about 10% of its compressive strength. On the other hand, steel is a material with very good tensile strength, even with reduced sections, making it ideal for combining with the concrete material. The three materials considered in this paper will be: reinforced concrete, structural steel and steel-concrete composite materials.

3.3.2.1 Reinforced Concrete Materials

Concrete is an artificial material obtained from the mixture of determined quantities of cement, aggregates and water. Cement and water create a paste that surrounds the aggregates, constituting a heterogeneous material. Sometimes, substances called admixtures and additions are added to modify some properties of the concrete.

There are many types of concrete available, created by varying the proportions of the main ingredients. In this way or by substitution for the cementitious and aggregate phases, the finished product can be tailored to its application with varying strength, density, or chemical and thermal resistance properties.

Mass concrete (without reinforcement) has a good compressive strength but is weak against tensile strength. This fact can be considered as a limiting factor in some structural applications. To provide concrete with greater tensile strength steel rods are used as reinforcement. The steel reinforcement is responsible of handling tensile strengths, providing concrete better properties as structural material. Reinforcement is also used to increase compressive resistance, as to reduce the cracking in concrete and deflections and to achieve major ductility on concrete. The combination of concrete and steel rods constitutes the reinforced concrete.

a) Cement

Portland cement is the most common type of cement in general usage. It is a basic ingredient of concrete, mortar and many plasters. It consists of a mixture of calcium silicates (alite, belite), aluminates and ferrites - compounds which combine calcium, silicon, aluminum and iron in forms which will react with water. Portland cement and similar materials are made by heating limestone which is a source of calcium with clay and/or shale (a source of silicon, aluminum and iron) and grinding this product called clinker with a source of sulfate.

b) Water

Combining water with a cementitious material forms a cement paste by the process of hydration. The cement paste glues the aggregate together, fills voids within it, and makes it flow more freely. A lower water-to-cement ratio yields a stronger, more durable concrete, whereas more water gives a freer-flowing concrete with a higher slump. Impure water used to make concrete can cause problems when setting or in causing premature failure of the structure. Hydration involves many different reactions, often occurring at the same time. As the reactions proceed, the products of the cement hydration process gradually bond together the individual sand and gravel particles and other components of the concrete to form a solid mass.

c) Aggregates

Fine and coarse aggregates make up the bulk of a concrete mixture. Sand, natural gravel and crushed stone are used mainly for this purpose. Recycled aggregates such as from construction, demolition, and excavation waste are increasingly used as partial replacements for natural aggregates, while a number of manufactured aggregates, including air-cooled blast furnace slag and bottom ash are also permitted.

The presence of aggregate greatly increases the durability of concrete above that of cement, which is a brittle material in its pure state, and also reduces cost and controls cracking caused by temperature changes. Thus concrete is a true composite material. Redistribution of aggregates after compaction often creates inhomogeneity due to the influence of vibration. This can lead to strength gradients.

d) Admixtures

Admixtures shall be understood to mean those substances or products which, once incorporated into concrete prior to or during mixing or additional mixing in individual proportions not exceeding 5% of the weight of the cement, ensure the desired alteration, in the fresh or hardened state, in any of the concrete's characteristics, usual properties or performance.

e) Additions

Additions are those inorganic or pozzolanic materials, or materials with latent hydraulicity, which, when finely divided can be added to concrete in order to improve one of its characteristics or to endow it with special properties. ES EN-2 only covers fly ash and silica fumes added to concrete at the time of casting. Fly ash is the solid residue collected by electrostatic precipitation or mechanical trapping of the dust accompanying the combustion gases of pulverized coal-fed thermoelectric plant burners. Silica fumes are a by-product obtained during the reduction of high-purity quartz, with carbon in electric arc furnaces for the production of silicon and ferrosilicon.

Additions may be used as concrete constituents provided that evidence can be provided of their suitability for use, and that the desired effect can be achieved without negatively impact on the concrete's characteristics or posing a risk to the concrete's durability or the corrosion-resistance of its reinforcements.

f) Reinforcement Bar

As mentioned before, steel provides concrete with tensile strength. According to ES EN-2, passive reinforcement is achieved by using mainly two types of bars: ribbed wieldable steel bars and ribbed wieldable steel supplied in coils. The possible nominal diameters of ribbed bars shall be as defined in the following series: 6 - 8 - 10 - 12 - 14 - 16 - 20 - 32 and 40 mm. Apart from in the case of electro-welded mesh fabrics or basic lattice reinforcements, diameters of less than 6 mm shall be avoided wherever any welding technique, either resistant or non-resistant, is used in the making or installation of passive reinforcements. The properties of reinforcement suitable for use with this Ethiopian Code are described in ES EN-2 section C.1(1) Table C.1 and Table C.2.

For the reinforced concrete alternative, the two materials considered when designing the building are concrete and rebar steel. The properties for each material used during modelling are represented in Table 3.1.

Table 3. 1: Material Properties for the Reinforced Concrete Alternative

Material Properties	Concrete		Rebar Steel
	Grade C-25	**Grade C-30**	
Specific weight (γ_c) (kg/m^3)	2400	2400	7850
Characteristic strength/yield stress (f_{ck}/f_{yk}) (N/mm^2)	25	30	500
Elastic modulus (E) (N/mm^2)	30000	31000	210000
Poisson coefficient (v)	0.2	0.2	0.3

3.3.2.2 Structural Steel Materials

Structural steel is a category of steel used as a construction material for making structural steel shapes. A structural steel shape is a profile, formed with a specific cross section and following certain standards for chemical composition and mechanical properties. Structural steel shapes, sizes, composition, strengths, storage practices and so on are regulated by standards. Structural steel is an industrial production material which ensures that has adequate quality control. This material is characterized by high strength, rigidity and ductility, making it a material widely used for the projection of earthquake-resistant structures.

There are many types of structural steel depending on their yield strength or their welding capability under certain conditions. ES EN-3 takes into account the structural steel types and its nominal thickness as shown in Table 3.2 and Figure 3.2:

Table 3. 2: Minimum yield Strength and Ultimate Yield Strength (N/mm2)

Type	Nominal thickness t (mm)			
	$t \leq 40$		$40 < t \leq 80$	
	f_y	f_u	f_y	f_u
S 235	235	$360<f_u<510$	215	$360<f_u<510$
S 275	275	$430<f_u<580$	255	$410<f_u<560$
S 355	355	$490<f_u<680$	335	$470<f_u<630$

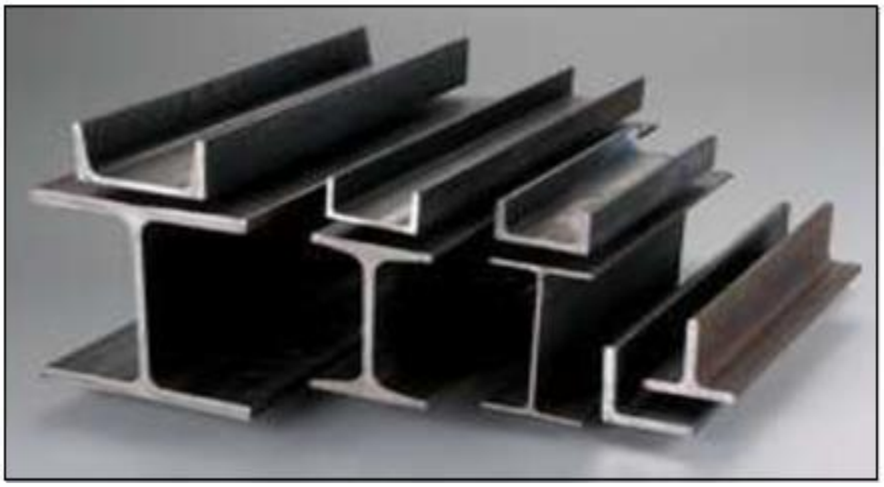


Figure 3. 2: Types of Structural Steel Profile

For the structural steel alternative, the materials considered when designing the building are concrete, profiled steel and rebar steel. Concrete properties for the structural steel alternative is the same as reinforced concrete alternative. The properties for structural steel material used during modelling are represented in Table 3.3.

Table 3. 3: Profiled and Rebar Steel Properties for the Structural Steel Alternative

Properties	**Rebar Steel**	**Structural Steel**
Specific weight (γ_c) (kg/m^3)	7850	7850
Characteristic yield stress (f_{sk}/f_{yk}) (N/mm^2)	355	500
Elastic modulus (E) (N/mm^2)	210000	200000
Poisson coefficient (v)	0.3	0.3

3.3.2.3 Steel-concrete Composite Materials

The materials for this alternative is the combination of reinforced concrete and structural steel described in Section 3.3.2.1 and Section 3.3.2.2 of this paper.

3.3.3 Actions

To make an adequate structural design or analysis, it is necessary to know all the loads acting on the structure and their value. The value of the actions may be known or unknown. In the latter case we must appeal to the rules for estimating the value of such actions and to carry out structural analysis.

A building or generally a structure has to be designed considering two types of loads: vertical or gravitational loads and lateral loads. Gravitational loads correspond to the structure self-weight and the summation of all the loads contained in the building shape. On the other hand, lateral loads correspond to wind action and seismic effects. Actions can appear for different reasons and may have different origins, but consider it is always necessary to define the problem.

3.3.3.1 Gravitational Loads

a) Self-weight Load

This load corresponds to the weight of the structural element itself and may vary depending on the material, shape and volume. In this thesis, the main materials considered are concrete, reinforcement bar and structural steel, which satisfies the three structural patterns considered such as reinforced concrete, structural steel and steel concrete composite.

Table 3. 4: Self-Weight of the Structural Materials

Type of material	**Self-weight (kg/m^3)**
Reinforced concrete	2400
Structural steel and reinforcement bar	7850

b) Super Dead Loads

These loads are considered as permanent loads. Their magnitude can be constant along time or can vary at one point. In this analysis, only constant value dead loads had been considered. In this way, the elements considered as dead loads are non-structural walls, impervious isolation layers in floors, tilling elements and its corresponding mortar layer and all the equipment needed to satisfy the function of the building such that Heating and cooling systems, electric equipment, pipes and ducts and so on. When partition walls are indicated on the plans, their weight shall be considered as dead load acting as distributed/line loads and uniform loads in their actual positions on the beams and floors, respectively. The loads due to anticipated partition walls, which are not indicated on the plans, shall be treated as live loads. Weight of fixed service equipment and other permanent machinery, such as electrical feeders and other machinery, heating, ventilating and airconditioning systems, lifts and escalators, plumbing stakes and so on. shall be included as dead load whenever such equipment is supported by structural members. The value of these actions has been taken from the Ethiopian standard-1 part 1-1, and is represented in table 3.5.

Table 3. 5: Supper Dead Loads that Considered on the Buildings

Element	Load	Unit
Supper dead load on beams (due to wall)	7	kN/m
Supper dead load on slabs (due to finishing, wall and fixed service equipment)	4	kN/m^2

c) Live Loads

Live load is the load superimposed by the use or occupancy of the building. These loads are the consequence of the usage of the building and their origin may be very different. The different live load categories are used in the model as per the architectural plan and the requirement of the ES EN-1. The categories of live load used and their value is shown on table 3.6.

Table 3. 6: Live Load Categories and Values

Category	Specific Use	Value (kN/m^2)
Category A	Domestic	2
Category B	Office	3
Category C	Area where people congregate	3
Category D	Shopping	4
Category E	Store	7.5
Category F	Parking (Light vehicle)	2
Category G	Parking (Heavy vehicle)	5
Category H	Roof not accessible	0.5

3.3.3.2 Lateral Loads

There are many types of lateral loads which can act on buildings, from which earthquake and wind are the major ones in the analysis of high rise buildings. The resistance of tall buildings to earthquake as well as to wind is the main determinant in the formulation of new structural systems that evolve by the continuous efforts of structural engineers to increase building height while keeping the requirement within acceptable limits and minimizing the amount of materials. In this case, from section 3.1 of this paper the exact location of the building is seismic zone area then only seismic actions are considered in the analysis.

a) Seismic Actions

The exact location of the building is categorized as a seismic zone-III. For building the design elastic response spectrum reduced by the behavior factor q is used. Determination of behavior factor q, which depends on the type of structural system, regularity in elevation and plan, and ductility class, is described later in this section.

The seismic action of the design of building is represented by the elastic response spectrum, Type 1 (Assuming $M_s > 5.5$ for soil class C) as per ES EN-8 section 3.2.2.2 (2). Reference peak ground acceleration is 0.1g for Addis Ababa, Ethiopia and since the building is categorized as Importance class II with importance factor $\gamma_I = 1.0$, the design ground acceleration is ag = 1.0 x 0.1g = 0.1g. With the design ground acceleration a_g = 1.0g, the provisions of low seismicity do not apply and the building is designed to meet the requirements of either of the two ductility class. Considering the more rigorous detailing requirements associated with ductility class high DCH that are more difficult to implement, then medium ductility class DCM is chosen for seismic design of the building. The elastic response spectrum is defined for 5% damping.

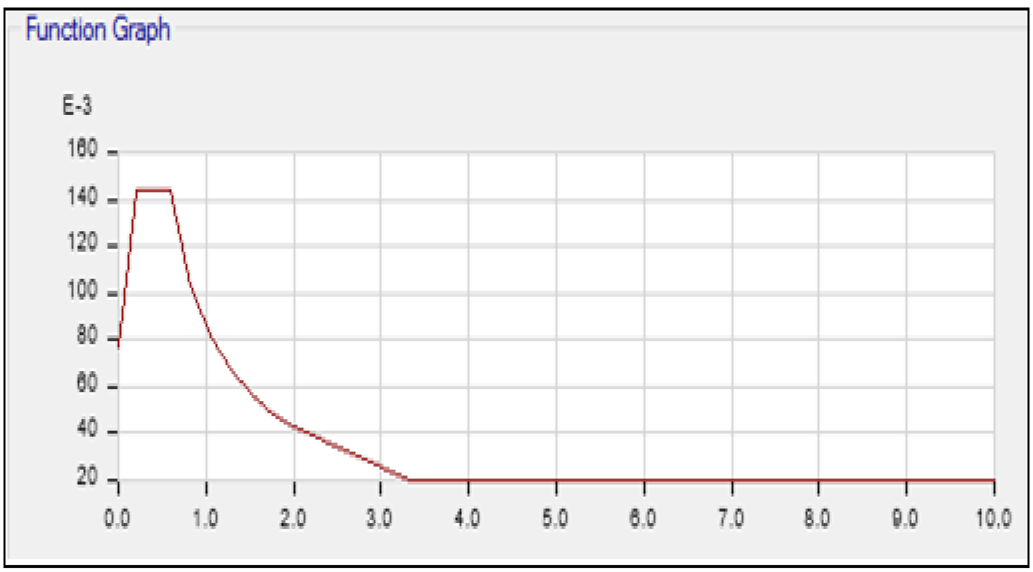


Figure 3. 3: Design Response Spectrum Function Graph (ETABS 2016 v 16.2.0)

According to ES EN-8, there are five typical ground types such as A, B, C, D, E and 2 special ground types such as S1 and S2 that may be used to account for the influence of local.

The average shear wave velocity $V_{s,30}$ in the top 30m from the surface is computed according with the equation given in ES EN-8 section 3.1.2 (3): $v_{s,30} = \frac{30}{\sum_{i=1,N}\frac{h_i}{v_i}}$.

Where, hi and vi denote the thickness (in meters) and the shear wave velocity (at a shear strain level of 10^{-5} or less) for the i^{th} formation or layer, in a total of N. The site should be classified according to the value of the average shear wave velocity, $v_{s,30}$, if this is available. Otherwise the value of N_{SPT} should be used ES EN-8 3.1.2 (2).

If this number is not available either, the undrained cohesion "Cu" can be used. The description of each ground type, and the definition of parameters is presents in table 3.7.

Table 3. 7: Seismic parameters for the different ground types (ES EN-8, Table 3.1)

Ground Types	**Description of stratigraphic profile**	**Parameters**		
		$V_{s,30}$ (m/s)	**N_{SPT} (blows/30cm)**	**Cu (kpa)**
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters' in thickness, characterized by a gradual increase of mechanical properties of depth.	360-800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with vs > 800 m/s.			
S1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	<100 (indicative)	–	10-20
S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A–E or S1			

Ground type that best suited to the geology of Gulele Sub-city is considered as soil C; so this type of ground is taken into account when modeling the structures. The elastic response spectrum shape is defined in figure-3.4.

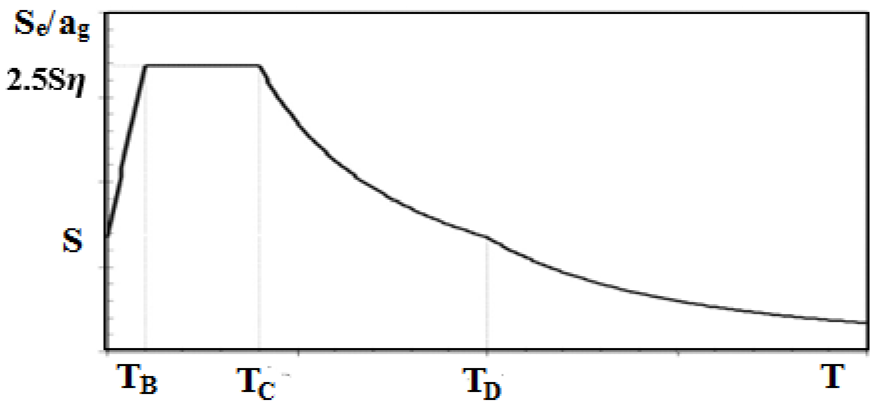


Figure 3. 4: Shape of the Elastic Response Spectrum (ES EN-8, Figure 3.1)

Where, T_B is the lower limit of the period of the constant spectral acceleration branch; T_C is the upper limit of the period of the constant spectral acceleration branch; T_D is the value defining the beginning of the constant displacement response range of the spectrum and S is the soil factor.

The parameters that define the shape of the spectrum depend on the ground type and can be obtained in table 3.8 and table 3.9 below.

Table 3. 8: Parameters of the Elastic Response Spectrum Type 1 (ES EN-8, Table 3.2)

Ground Type	**S**	**T_B(s)**	**T_C(s)**	**T_D(s)**
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

Table 3. 9: Parameters of the Elastic Response Spectrum Type 2 (ES EN-8, Table 3.3)

Ground Type	**S**	**T_B(s)**	**T_C(s)**	**T_D(s)**
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

According to the parameters stated in table 3.7 and 3.8, figure 3.4 and figure 3.5 present the elastic response spectrums defined by Ethiopian Standard-8 for each ground type.

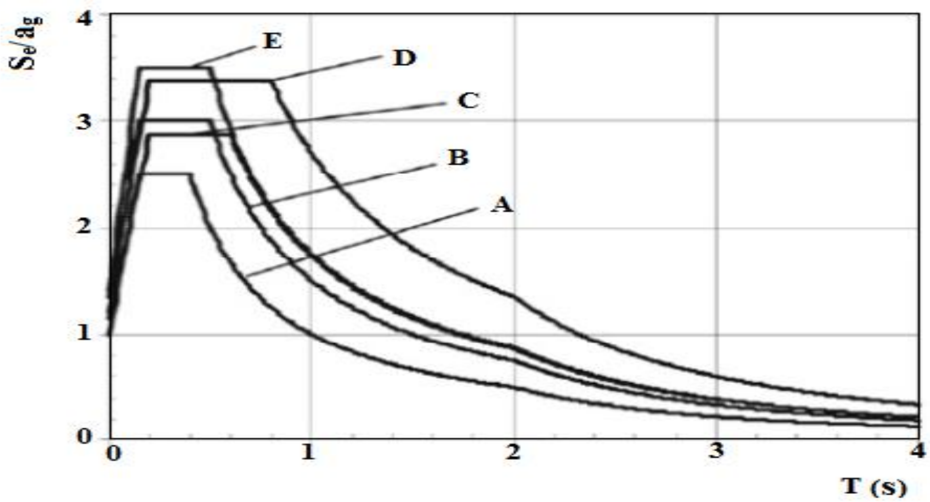


Figure 3. 5: Elastic Response Spectrum Type 1 for Damping 5% (ES EN-8, Figure 3.2)

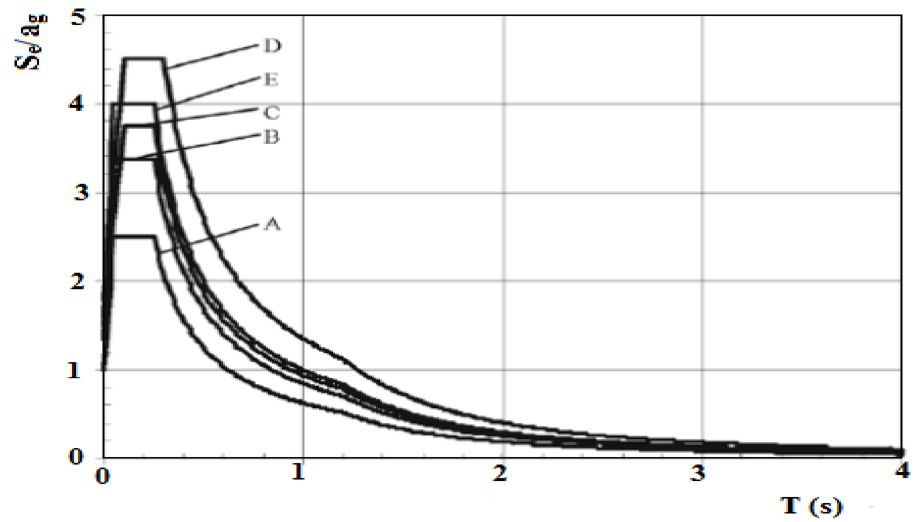


Figure 3. 6: Elastic Response Spectrum Type 2 for Damping 5% (ES EN-8, Figure 3.3)

In the calculations, only the horizontal displacement due to earthquake was taken into account.

3.3.4 Combination of Actions

A combination of actions shall consist of a set of compatible actions which shall be considered as acting simultaneously for a specific check. Each combination will usually comprise permanent actions, one determinant variable action and one or more concomitant variable actions. Any of the variable actions may be the determinant action. The combinations are depending on the limit states. For Ultimate limit states (ULS) and serviceability limit states (SLS) the combinations may vary by introducing different coefficients. The representative value of an action is the value used to check its limit states. One action may have one or more representative values, depending on its type. Some basic loads assigned in ETABS models are:

1. DL = dead loads
2. LL = live loads
3. EQXT = earthquake loads on the top side of X-direction.
4. EQXB = earthquake loads on the bottom side of X-direction.
5. EQYL = earthquake loads on the left side of Y-direction.
6. EQYR = earthquake loads on the right side of Y-direction.

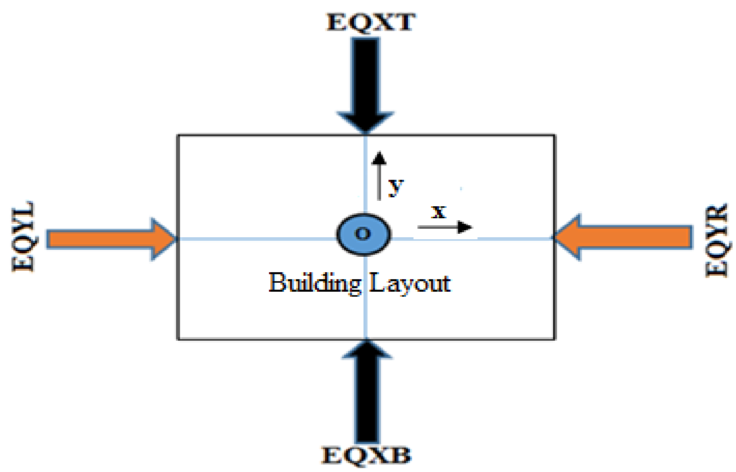


Figure 3. 7: Location of Seismic Loads on the Building

a) Ultimate limit states

The combinations of actions for persistent or temporary situations is given by the expressions:

Gravity = $1.35 \times G_K + 1.5 \times Q_K$ and for seismic action situations: $\sum G_{K,j} + \sum \psi_{E,i} \times Q_{K,i} + A_{Ed}$

Where, $G_{k,j}$ is Characteristic value of permanent actions; $Q_{k,i}$ Characteristic value of the determinant variable action; $\psi_{E,i} Q_{k,i}$ is Representative value of a combination of variable actions acting at the same time as the determinant variable action; A_{Ed} is represents the accidental action which is Response Spectrum (RS).

b) Serviceability limit states

In this case, only persistent and temporary design situations are considered for serviceability limit states. Therefore, combinations of actions shall be defined as following expressions in ES EN-2. Combination of action for both seismic and response spectrum is discussed in Appendix-A.

3.4 Modeling and Analysis of Structural Buildings

3.4.1 Modelling and Design Software

Today, computer-aided design is common in all areas and especially in the world of civil engineering. Being able to carry out complex simulations and calculations in a short time has allowed expedite the planning phase of projects. On the other hand, it implies a great saving of money over traditional methods of design and testing structures.

These software is useful tools to get an idea of the behavior that will have a structure in reality when subjected to certain actions. In this case, the software ETABS 2016 version 16.2.1 is the mainly used software for analysis and design of the structural buildings. This program allows modeling the entire building and analyzing their structural behavior. By using this softwares, it is possible to entirely describe the building structure and simulate its behavior under the different loads. The structural model fulfils all the requirements of ES EN-8.

3.4.2 Modelling Description

The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered ES EN-8 section 4.3.1(1).

- All beams and columns are modeled as line elements. But floors are modeled as areal elements. All element modeling follows centerline analysis.
- Effective widths of beams are not calculated; therefore, all beams are modeled as rectangular sections. Infills are not considered in the model.
- The cracked elements are considered as per ES EN-8 section 4.3.1(6). The elastic flexural and shear stiffness properties are taken to be equal to one-half of the corresponding stiffness of the un-cracked elements, such that the moment of inertia and shear area of the un-cracked section were multiplied by factor 0.5.
- The torsional stiffness of the elements was reduced as per ES EN-8 section 4.3.1(7). Torsional stiffness of the cracked section was set equal to 10% of the torsional stiffness of the un-cracked section. The accidental torsional effects are taken into account by means of accidental eccentricities in both direction according to ES EN-8 section 4.3.2(1).
- The center of mass and center of stiffness of each floors are determined from the ETABS software.

3.4.2.1 Modelling of Reinforced Concrete Buildings

a) Modeling of reinforced concrete columns

The columns are modeled and designed using ETABS element type "column". Rectangular columns are considered in the design. According to the bidirectional configuration considered, these columns will withstand bending moments in the two main directions and high compression on their Z-axis acting simultaneously to consider the worst loading case scenario. Moreover, columns have to be able to handle the shear forces applied on the two main directions due to lateral loads acting in X and Y-axis.

b) Modeling of reinforced concrete beams

Beams are modeled and design using ETABS element type “Beam”. Rectangular beams are taken into account during modelling to look for the better and best fitting solution. Beams will be subjected to the principal bending moment acting in their longitudinal plane and shear forces acting predominantly in their extremes. These stresses will determine the necessary rebar for each beam on different floors. On the other hand, it’s necessary to ensure the correct interaction beam-column to satisfy the proper performance of the frame system. To ensure it the model has to fulfil the strong column-weak beam principle. This can be check in ETABS through the beam column capacity ratio obtained after the analysis.

c) Modeling of reinforced concrete slabs

Slabs are modeled using ETABS element type “shell”. As told before, solid slab is used on the model. To consider the slab as a monolithic unit capable of resisting lateral forces, it is assigned to each slab a diaphragm that simulates that behavior. Slabs are adequately mesh to obtain satisfactory results. The art of creating area element models includes determining what constitutes an adequate mesh.

3.4.2.2 Modelling of Structural Steel Buildings

a) Modeling of structural steel columns

The columns are modeled and designed using ETABS element type "column". According to the bidirectional configuration considered, these columns will withstand bending moments in the two main directions and high compression on their Z-axis acting simultaneously to consider the worst loading case scenario. Moreover, columns have to be able to handle the shear forces applied on the two main directions due to lateral loads acting in X and Y-axis. Built-up steel sections are used to model for steel columns.

b) Modeling of steel girder and secondary beams

Beams are modeled and design using ETABS element type “Beam”. Built-up steel sections are taken into account during modelling to look for the better and best fitting solution. Beams is subjected to the principal bending moment acting in their longitudinal plane and shear forces acting predominantly in their extremes. These stresses will determine the necessary profile for each beam on different floors. Similar to reinforced concrete alternative, it’s necessary to ensure the correct interaction beam-column to satisfy the proper performance of the frame system. Joists is assigned using ETABS element type “secondary beam”.

c) Modeling of composite deck slabs

Composite slabs are modeled using ETABS element type “deck”. Since, this alternative considers structural steel and concrete in the modeling of slab. Then, the solution for the slabs that combines steel and concrete and that performs the best composite slab.

3.4.2.3 Modelling of Steel-concrete Composite Building

a) Modeling of composite columns

The columns are modeled and designed using ETABS element type "column". Rectangular composite columns are considered in the design due to their ease of construction; Column dimensions is vary depending on its position or floor, so all columns have not the same section.

According to the bidirectional configuration considered, these columns will withstand bending moments in the two main directions and high compression on their Z-axis acting simultaneously to consider the worst loading case scenario. Columns have to be able to handle the shear forces applied on the two main directions due to lateral loads acting in X and Y-axis.

b) Modeling of steel girder and composite beams

For this case modeling of beam are done following exactly the same procedures stated for the structural steel beam alternative in section 3.4.2.2 of this paper.

c) Modeling of composite deck slabs

Again modeling of composite slabs are done following exactly the same procedures stated for the structural steel composite alternative in section 3.4.2.2 of this paper.

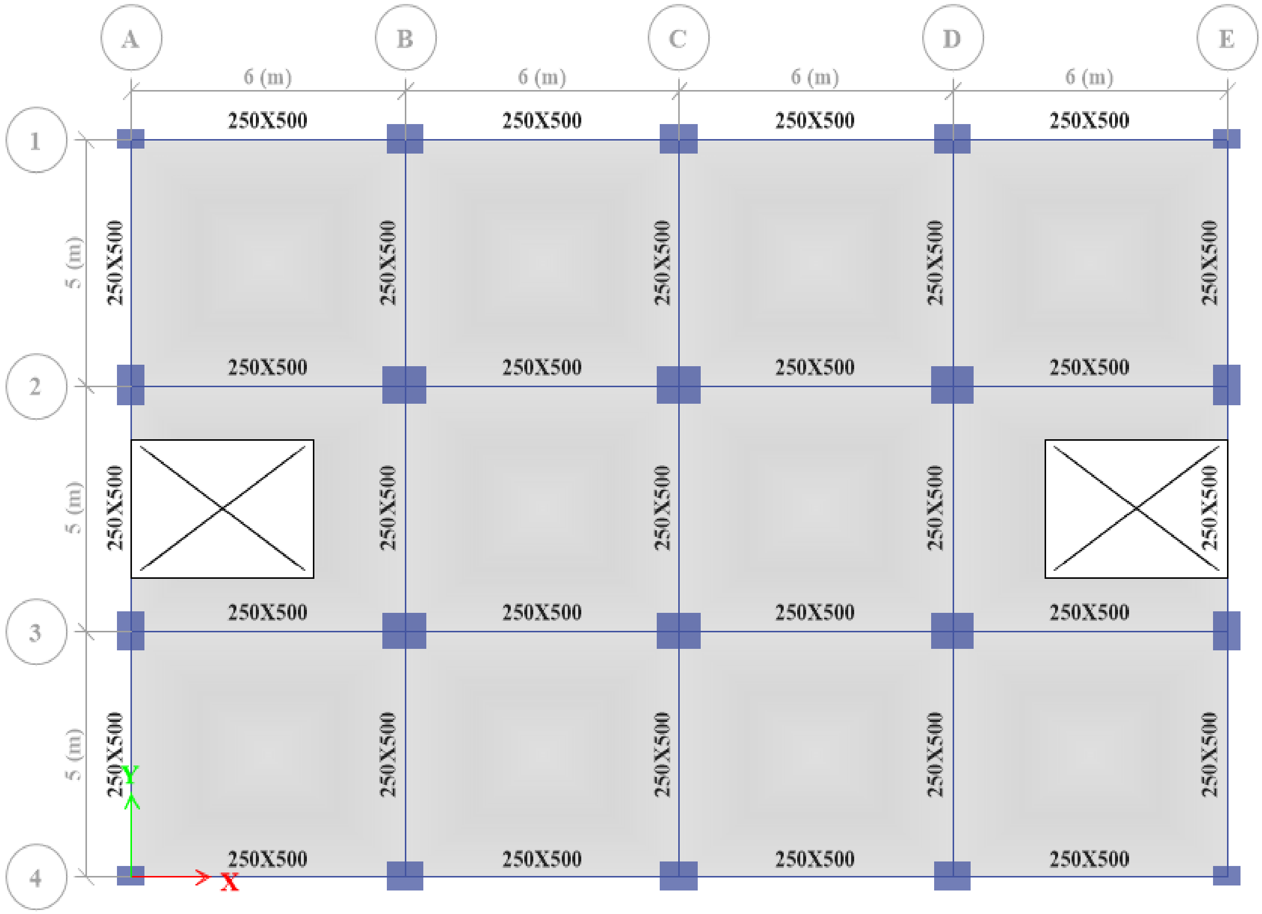


Figure 3. 8: 2D Plan Model of Reinforced Concrete Alternative @ 3m

Figure 3. 9: 2D Plan Model of Structural Steel Alternative @ 3m

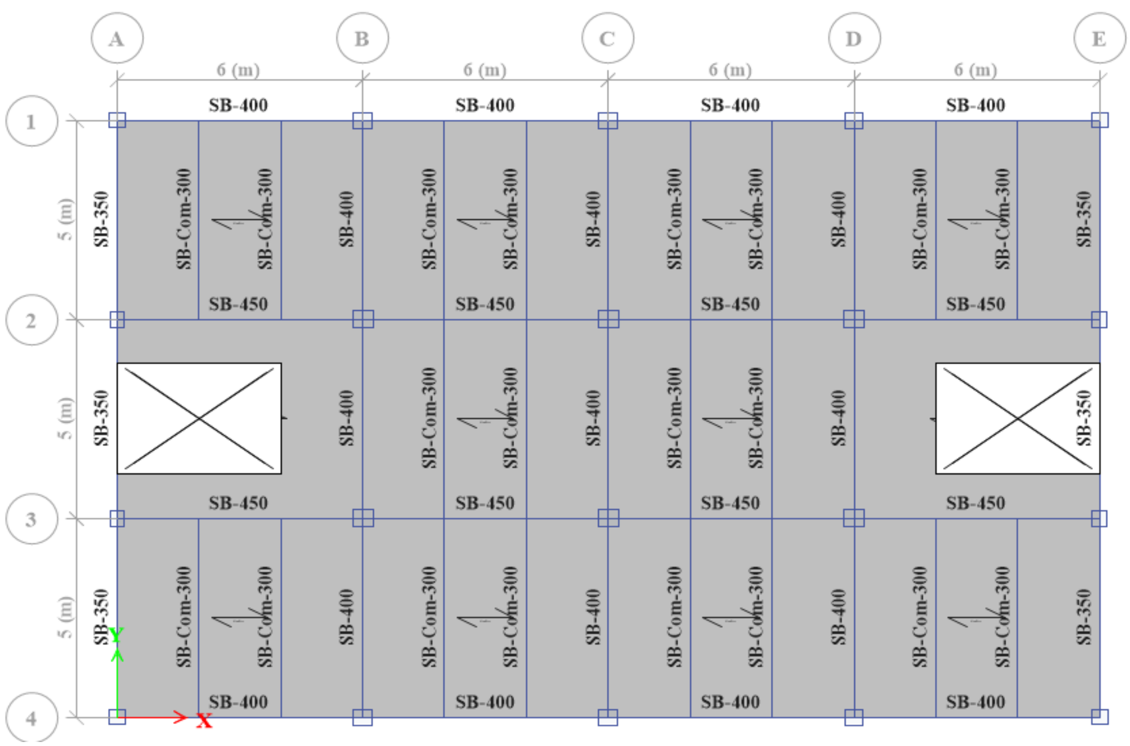


Figure 3. 10: 2D Plan Model of Steel-concrete Composite Alternative @ 3m

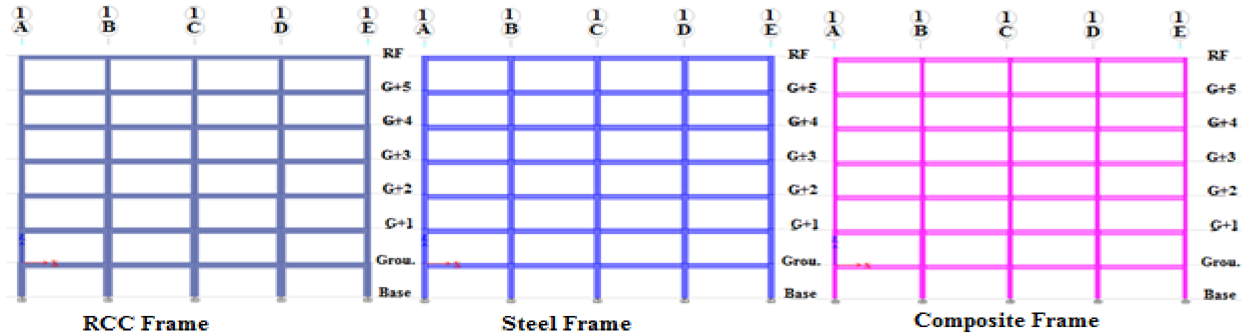


Figure 3. 11: Elevation Model of RCC, Steel and Composite Frame for G+5 Buildings

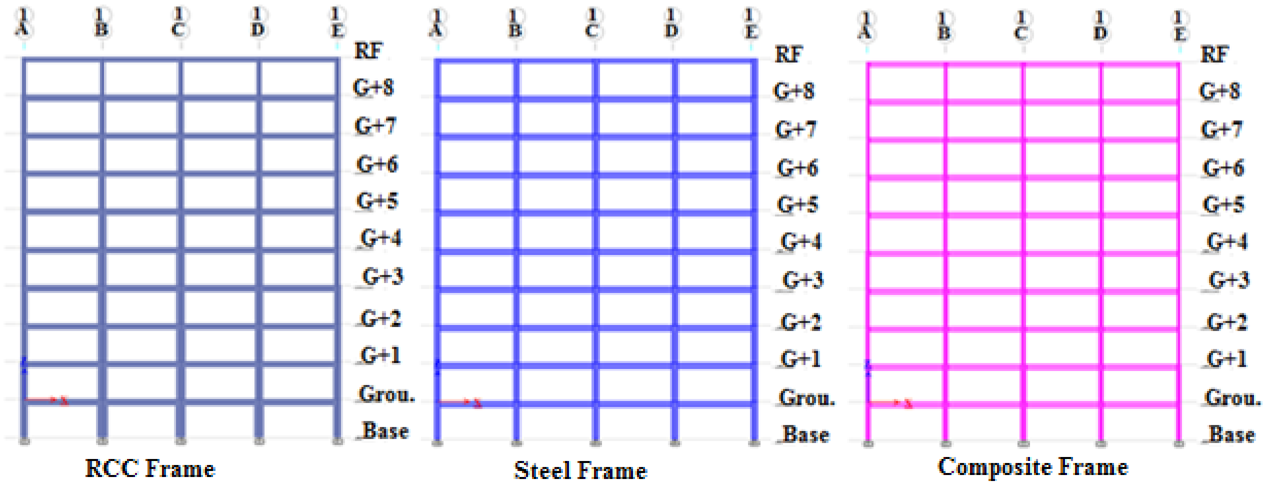


Figure 3. 12: Elevation Model of RCC, Steel and Composite Frame for G+8 Alternatives

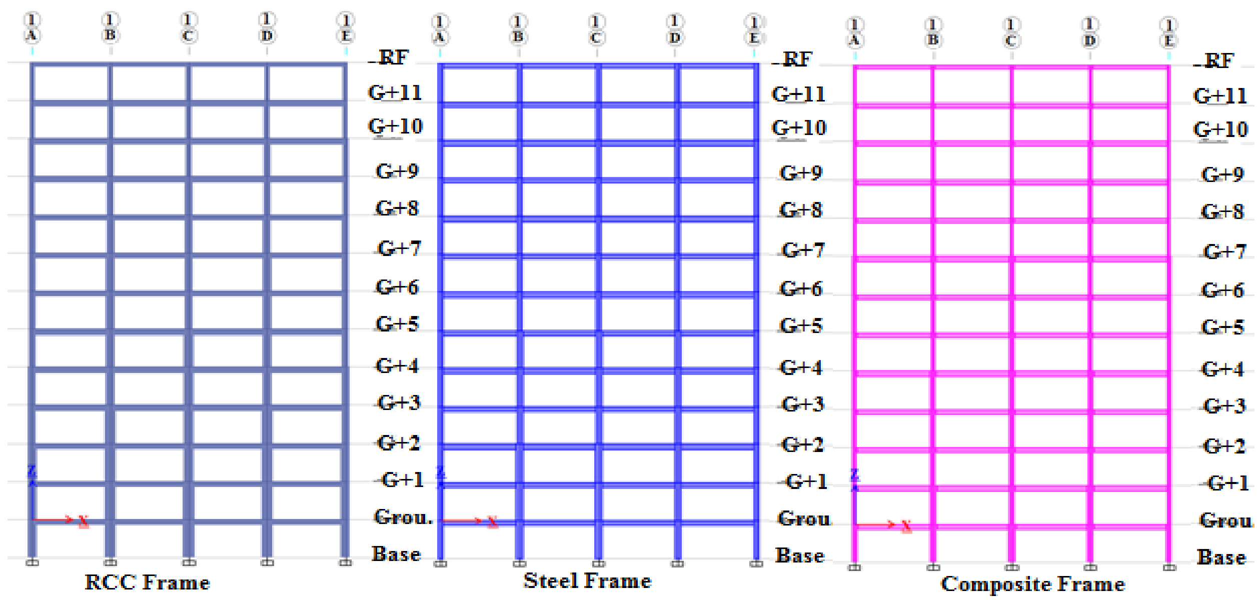


Figure 3. 13: Elevation Model of RCC, Steel and Composite Frame for G+11 Alternatives

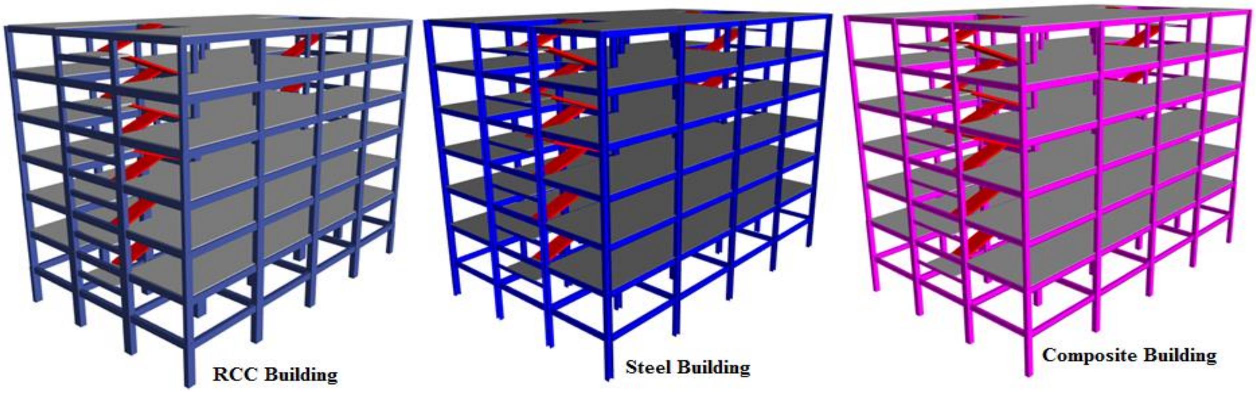


Figure 3. 14: 3D Model of RCC, Steel and Composite Building for G+5 Alternatives

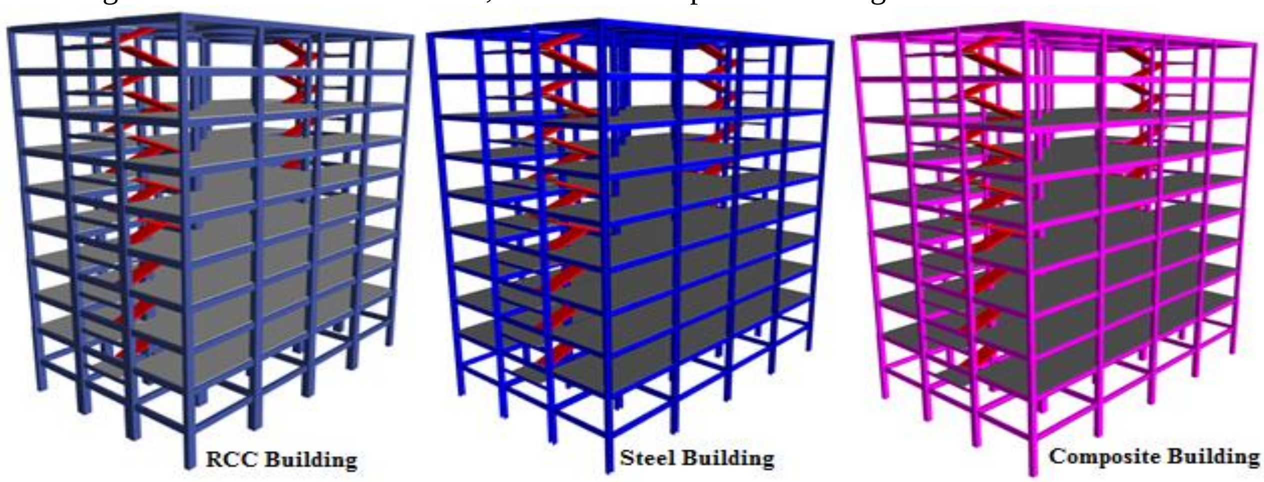


Figure 3. 15: 3D Model of RCC, Steel and Composite Building for G+8 Alternatives

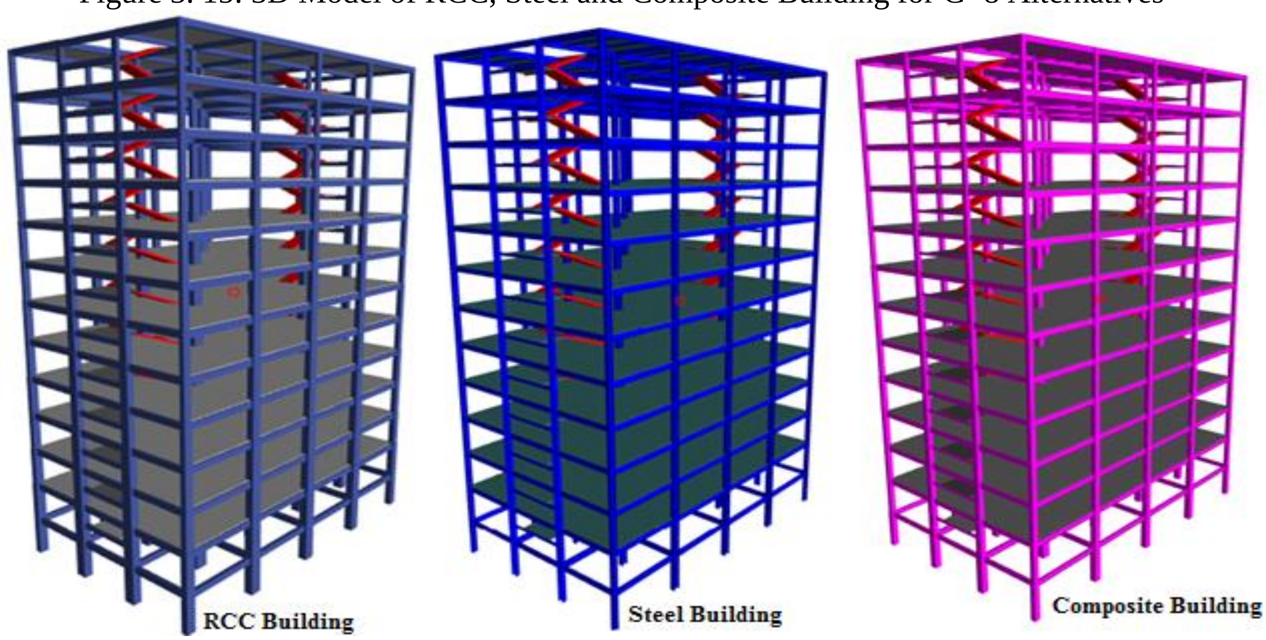


Figure 3. 16: 3D Model of RCC, Steel and Composite Building for G+11 Alternatives

3.4.3 Structural Regularity

Regularity of the structure in elevation and in plan influences the required structural model, method of analysis and value of the behavior factor q. Criteria describing regularity in plan and in elevation are given as per ES EN-8 in section 4.2.3.2 and 4.2.3.3, respectively. The structure can be categorized as being regular or irregular in elevation and in plan. A lot of work has to be done to check the criteria for regularity, see Appendix-E at the end of this paper.

3.4.3.1 Criteria for Regularity in Plan

In general, the regularity in plan can be checked when the structural model is defined. The criteria for regularity in plan are described in ES EN-8 Section 4.2.3.2. Irregularity in plan may influence the magnitude of the seismic action over strength factor. The reference value of the basic behavior factor q_o can be used according to ES EN-8 Table 4.1. For the building to be regular in plan the following requirements should be fulfilled.

- The slenderness of the building λ shall be not higher than 4, such that $\lambda = {}^{L_{max}}/_{L_{min}}$
- The structural eccentricity shall be smaller than 30% of the torsional radius, which is calculated using: $e_{ox} \leq 0.30r_x$ and $e_{oy} \leq 0.30r_y$.
- The torsional radius shall be larger than the radius of the gyration of the floor mass in plan $r_x \geq l_s$ and $r_y \geq l_s$.

Based on the above conditions, the criteria for regularity in plan for all structural alternatives are discussed in Appendix-E, see table E.1 to table E.10 and the overall buildings are considered as regular in plan.

3.4.3.2 Criteria for Regularity in Elevation

For the building to be regular in elevation the following requirements should be fulfilled.

- All lateral force resisting system should run from their foundation to the top without interruption. Except the basement wall all walls and columns runs to the top.
- Both lateral stiffness and mass of individual of stories shall remain constant or reduce gradually.

Similar to plan regularity, the criteria for regularity in elevation for all structural alternatives are discussed in Appendix-E, see table E.11 to table E.22 and the overall buildings are considered as irregular in elevation. Therefore, the test result is summarized in table 3.10.

Table 3. 10: Consequences of Structural Regularity on Seismic Analysis and Design

Regularity		**Allowed Simplification**		**Behaviour Factor**
Plan	Elevation	Model	Linear-elastic Analysis	(For linear Analysis)
Yes	Yes	Planar	Lateral force[a]	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial[b]	Lateral force[a]	Reference value
No	No	Spatial	Modal	Decreased value

3.4.4 Methods of Analysis

According to ES EN-8 section 4.3.3, depending on the structural characteristics of the building one of the following two types of linear-elastic analysis may be used:

a) the "lateral force method of analysis"

b) for buildings meeting the conditions given in 4.3.3.2;

b) The "modal response spectrum analysis", which is applicable to all types of buildings (see 4.3.3.3).

In this case, modal response spectrum analysis is performed independently for the ground excitation in two horizontal directions. According to section 4.3.3.3.1(3) the sum of the effective modal masses for the modes taken into account to at least 90% of the total mass of the structure. The accidental torsional effects are taken into account by means of accidental eccentricity which is $e_i = \pm 0.05 \times L_i$ in both directions ES EN-8 section 4.3.2(1). The Complete Quadratic Combination rule for the combination of different modes is used as per ES EN-8 section 4.3.3.3.2(3). The modal mass participation ratio is discussed in Appendix-D.

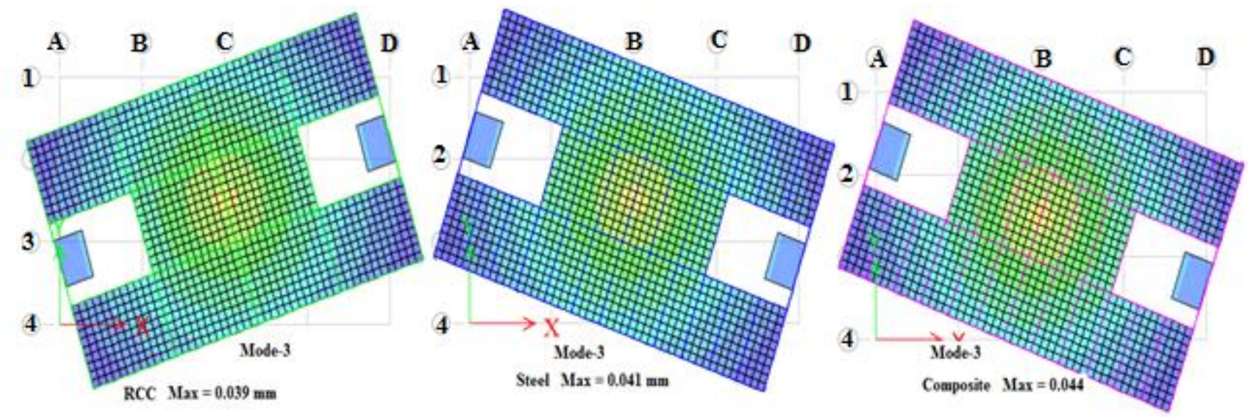


Figure 3. 17: Plan Mode Shape of RCC, Steel and Composite Building for G+5 Alternatives

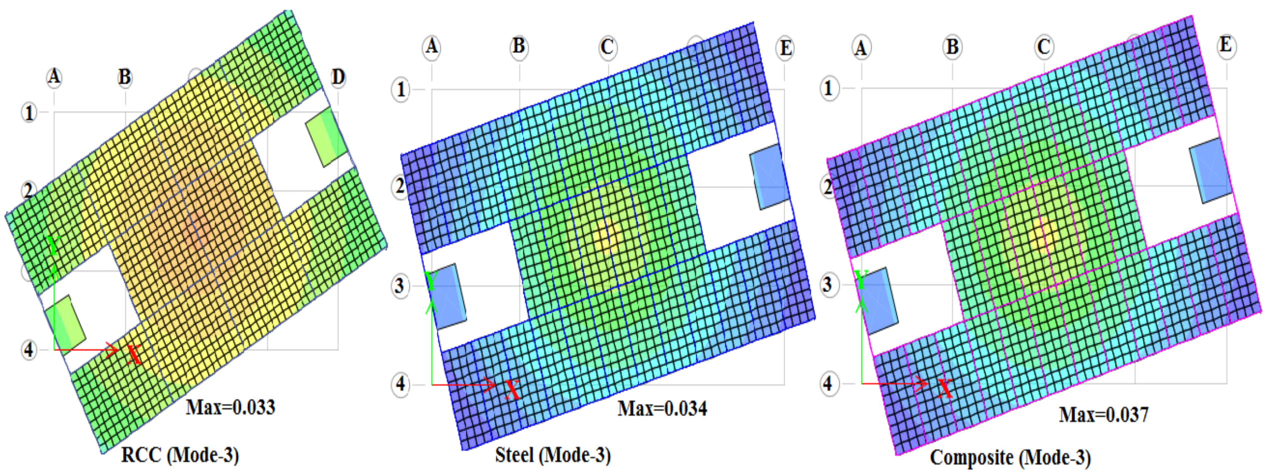


Figure 3. 18: Plan Mode Shape of RCC, Steel and Composite Building for G+11 Alternatives

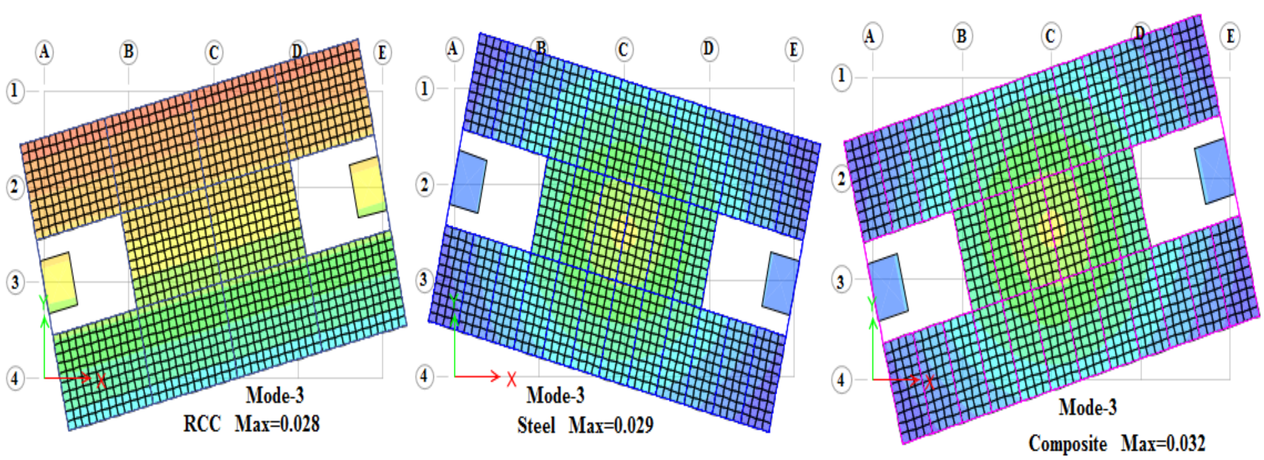


Figure 3. 19: Plan Mode Shape of RCC, Steel and Composite Building for G+11 Alternatives

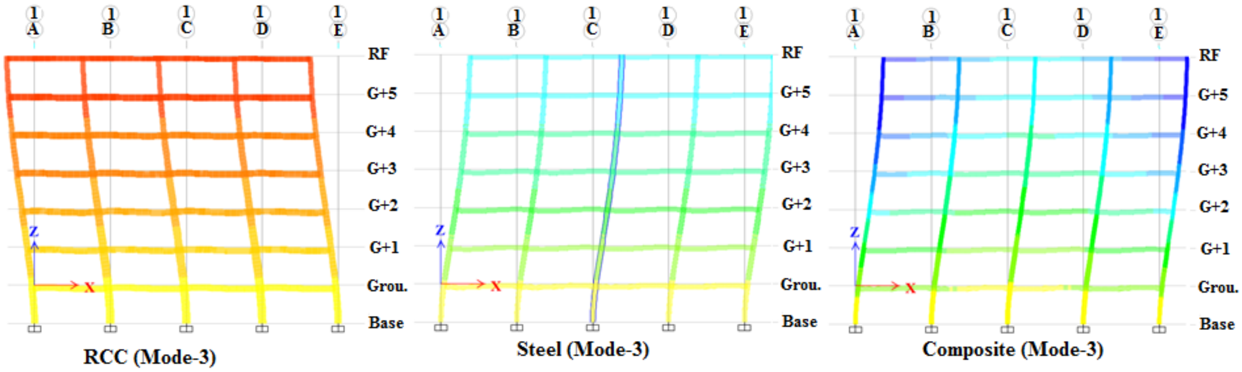


Figure 3. 20: Frame Mode Shape of RCC, Steel and Composite Building for G+5 Alternatives

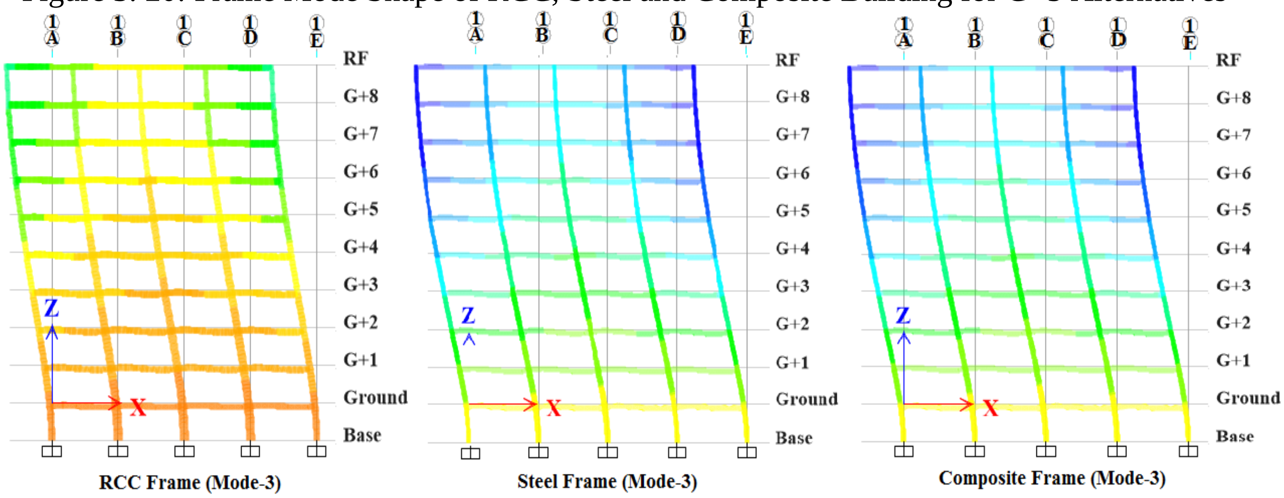


Figure 3. 21: Frame Mode Shape of RCC, Steel and Composite Building for G+8 Alternatives

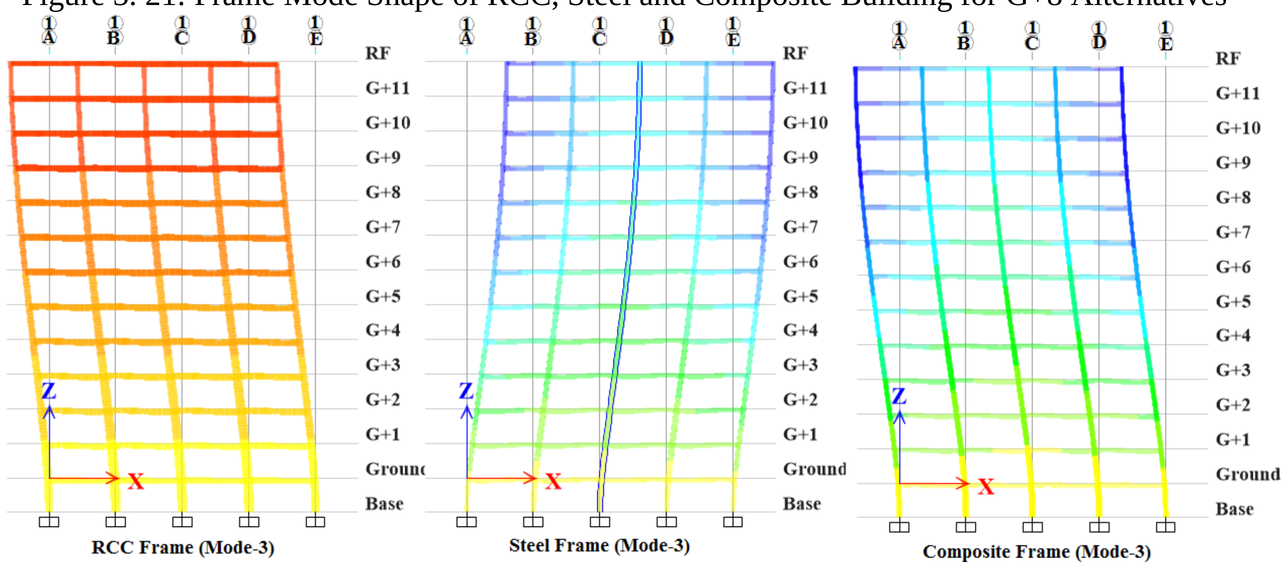


Figure 3. 22: Frame Mode Shape of RCC, Steel and Composite Building for G+11 Alternatives

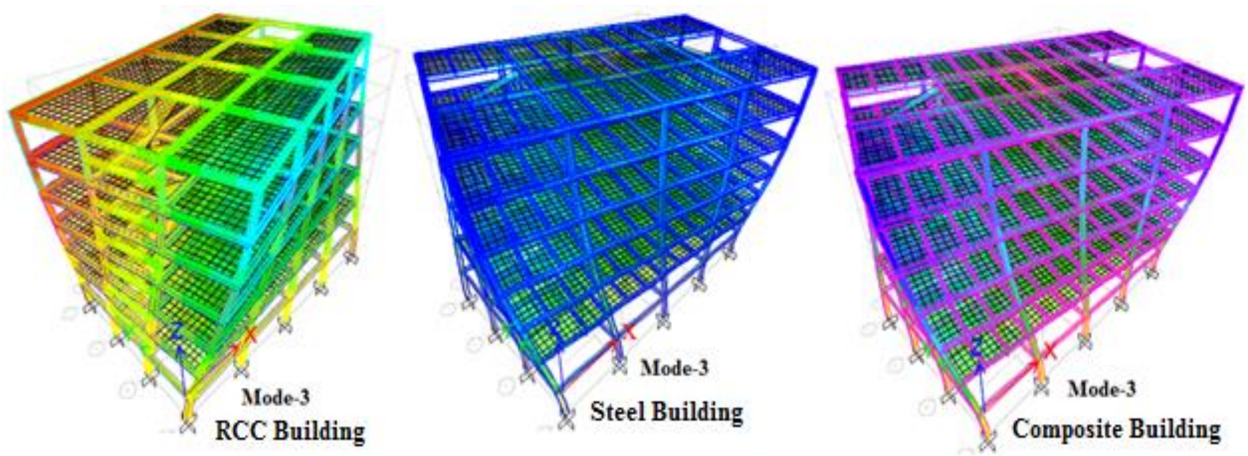


Figure 3. 23: 3D Mode Shape of RCC, Steel and Composite Building for G+5 Alternatives

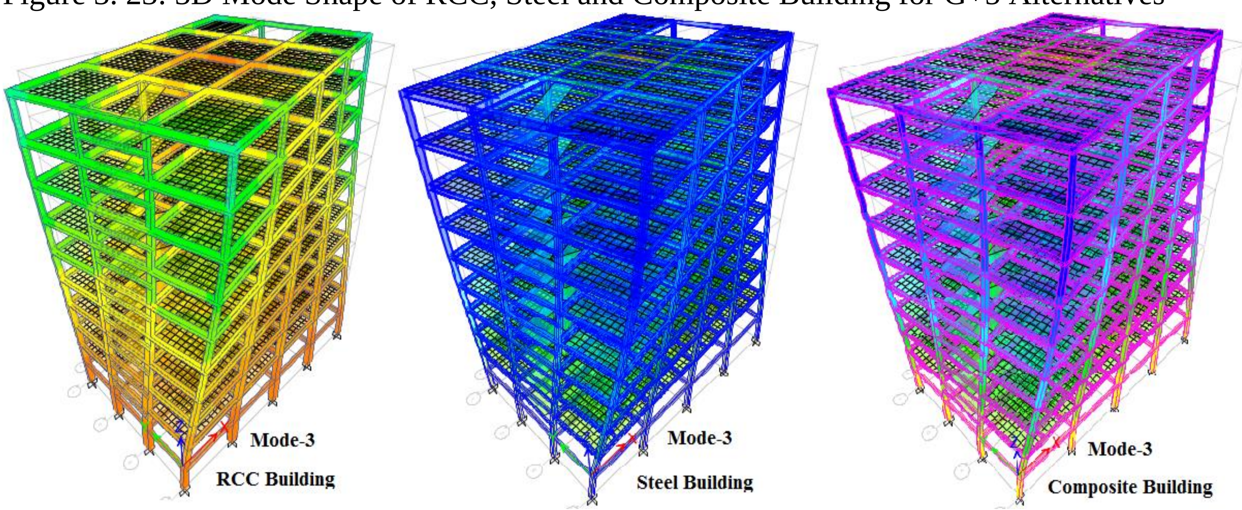


Figure 3. 24: 3D Mode Shape of RCC, Steel and Composite Building for G+8 Alternatives

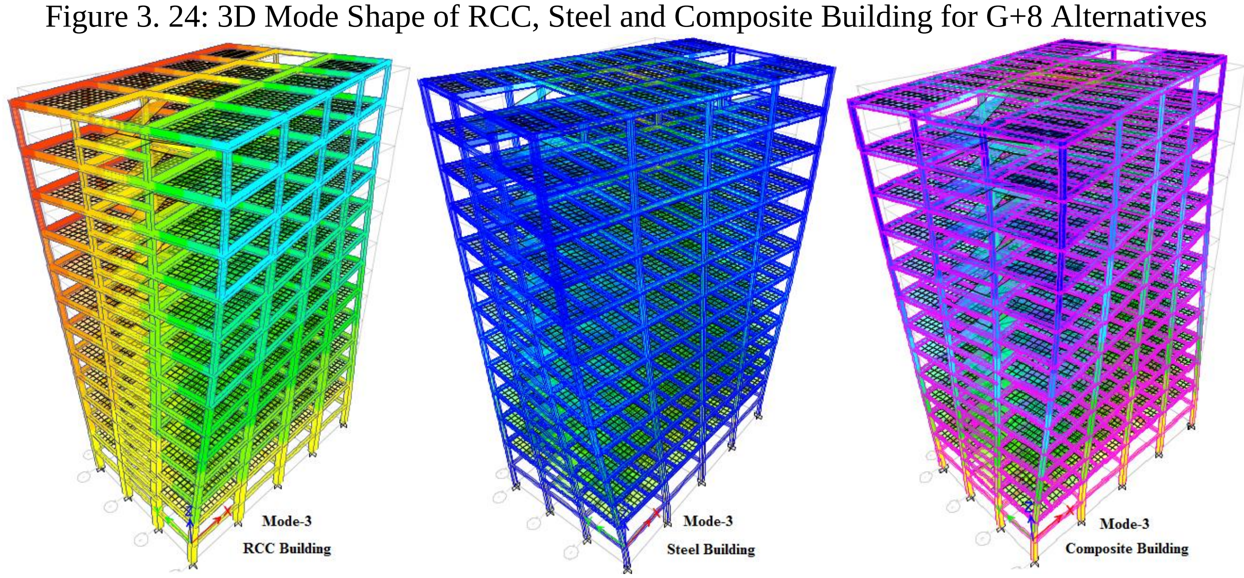


Figure 3. 25: 3D Mode Shape of RCC, Steel and Composite Building for G+11 Alternatives

3.4.5 Structural Types and Behavior Factor

According to ES EN-8 section 3.2.2.5(3), the behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various parts of ES EN 8. The value of the behaviour factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions.

3.4.5.1 Reinforced Concrete Building Alternatives

As per ES EN-8 section 5.2.2.2(1), The upper limit value of the behaviour factor q, introduced in section 3.2.2.5(3) of ES EN-8 to account for energy dissipation capacity, shall be derived for each design direction as: $q = q_o \times K_w \geq 1.5$.

Where; q_o - is the basic value of the behaviour factor, dependent on the type of the structural system and on its regularity in elevation see Table 5.1 ES EN-8 and k_w - is the factor reflecting the prevailing failure mode in structural systems with walls which is taken as for frame systems 1.0 as per ES EN-8 section 5.2.2.2(11).

Table 3. 11: Value of Behaviour Factor, q_o for Systems Regular in Elevation (ES EN-8 Table 5.1)

Structural Type	**DCM**	**DCH**
Frame system, dual system, coupled wall system	$3.0\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$
Uncoupled wall system	3.0	$4.0\alpha_u/\alpha_1$
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

In this case, the reinforced concrete building is classified as frame system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system. Since, for structural type of frame system as per ES EN-8 section 5.2.2.2(5) the multiplication factor α_u/α_1 has not been evaluated through an explicit calculation, for buildings which are regular in plan the values of α_u/α_1 is taken as 1.3, see figure-6.1 of ES EN-8 and q_o = 3*1.3 = 3.9.

According to section 4.2.3.1(7) of ES EN-8, for non-regular in elevation buildings the decreased values of the behaviour factor are given by the reference values multiplied by 0.8. Since, the building is irregular in elevation q = 0.8*3.9 = 3.12.

3.4.5.2 Structural Steel Building Alternatives

The structural type this steel building alternative is assigned to moment resisting frames in which the horizontal forces are mainly resisted by members acting in an essentially flexural manner as shown in figure 3.13.

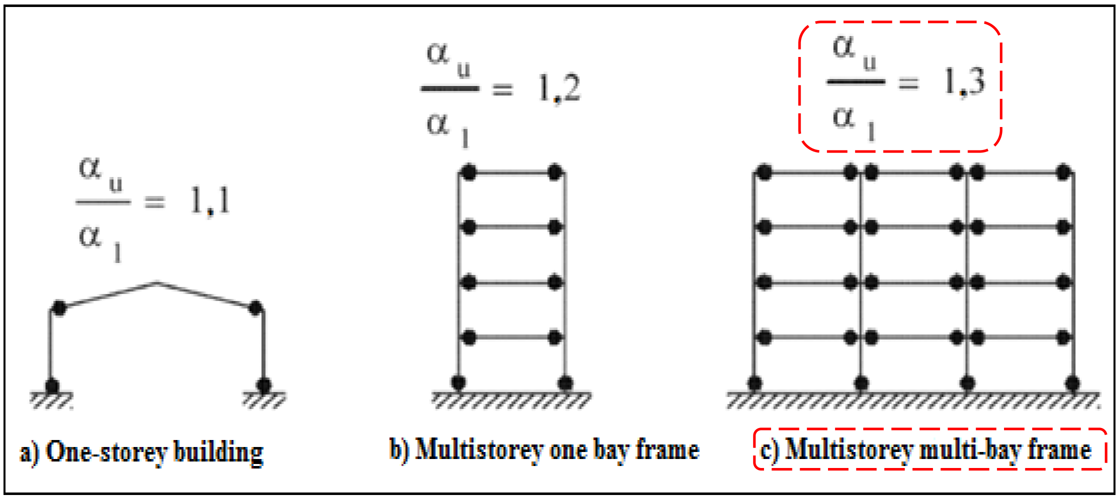


Figure 3. 26: Structural types of Moment Resisting Frames (ES EN-8 Figure 6.1)

As per ES EN-8 section 6.3.2(1), "The behaviour factor q, introduced in 3.2.2.5, accounts for the energy dissipation capacity of the structure. For regular structural systems, the behaviour factor q should be taken with upper limits to the reference values which are given in Table 6.2".

Table 3. 12: Values of Behavior Factors for Systems Regular in Elevation (ES EN-8 Table 6.2)

Structural Type	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5.0\alpha_u/\alpha_1$
b) Frame with concentric bracings Diagonal bracings V-bracings V-bracings	4 2	4 2.5
Table is continued for other structural types……….See ES EN 8 Table 6.2		

For structural type of moment resisting frames and medium ductility class as per ES EN-8 section 6.3.2(1) Table 6.2, the behaviour factor q_o should be taken with upper limits to the reference values of q_o= 4. As told before, the building is irregular in elevation, since q = 0.8*4 = 3.2.

3.4.5.3 Steel-concrete Composite Building Alternatives

According to ES EN-8 section 7.3.1(1) composite steel-concrete structures of this thesis shall be assigned to composite moment resisting frames with the same definition and limitations as in structural steel alternatives, but in which columns are composite steel-concrete, see Figure 6.1 in ES EN-8.

As per ES EN-8 section 7.3.1(1), the behaviour factor q, introduced in 3.2.2.5, accounts for the energy dissipation capacity of the structure and for regular structural systems, the behaviour factor should be taken with upper limits to the reference values which is given in Table 6.2 of ES EN 8. Since, the behaviour factor q is the same as the structural steel building alternatives q = 3.2.

3.4.6 Safety Verifications

According to ES EN-8 section 4.4.1(1), “For the safety verifications the relevant limit states in section 4.4.2 and 4.4.3, and specific measures in section 2.2.4 shall be considered”.

3.4.6.1 Resistance Condition

In structures important for civil protection the structural system shall be verified to ensure that it has sufficient resistance and stiffness to maintain the function of the vital services in the facilities for a seismic event associated with an appropriate return period, ES EN-8 section 2.2.3(2).

The criteria for taking into account the second order effect is based on the intersorey drift sensitivity coefficient θ, which is defined with equation of ES EN-8 section 4.4.2.2(2): $\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h}$.

where; θ is the interstorey drift sensitivity coefficient: P_{tot} is the total gravity load at and above the storey considered in the seismic design situation: d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements ds at the top and bottom of the storey under consideration and calculated in accordance with section 4.3.4 of ES EN-8, V_{tot} is the total seismic storey shear; and h is the interstorey height.

As per ES EN-8 section 4.4.2.2(2), second-order effects or P-Δ effects need not be taken into account if $\theta \leq 0.10$.

The consideration of P-Δ effects based on the consequence value of the interstorey drift sensitivity coefficient θ is summarized in Table 3.13.

Table 3. 13: Consideration of P-Δ effects Analysis Based on Values of θ

$\theta \leq 0.1$	No need to consider P-Δ effects
$0.1 \leq \theta \leq 0.2$	P-Δ effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$
$0.2 \leq \theta \leq 0.3$	P-Δ effects must be accounted for by analysis including second order effects explicity
$\theta \geq 0.3$	Not permitted

☞ For this thesis, the P-Δ effects for all structural alternatives are discussed in Appendix-C from Table C.10 to Table C.27 and there is no need to consider P-Δ effects for all cases.

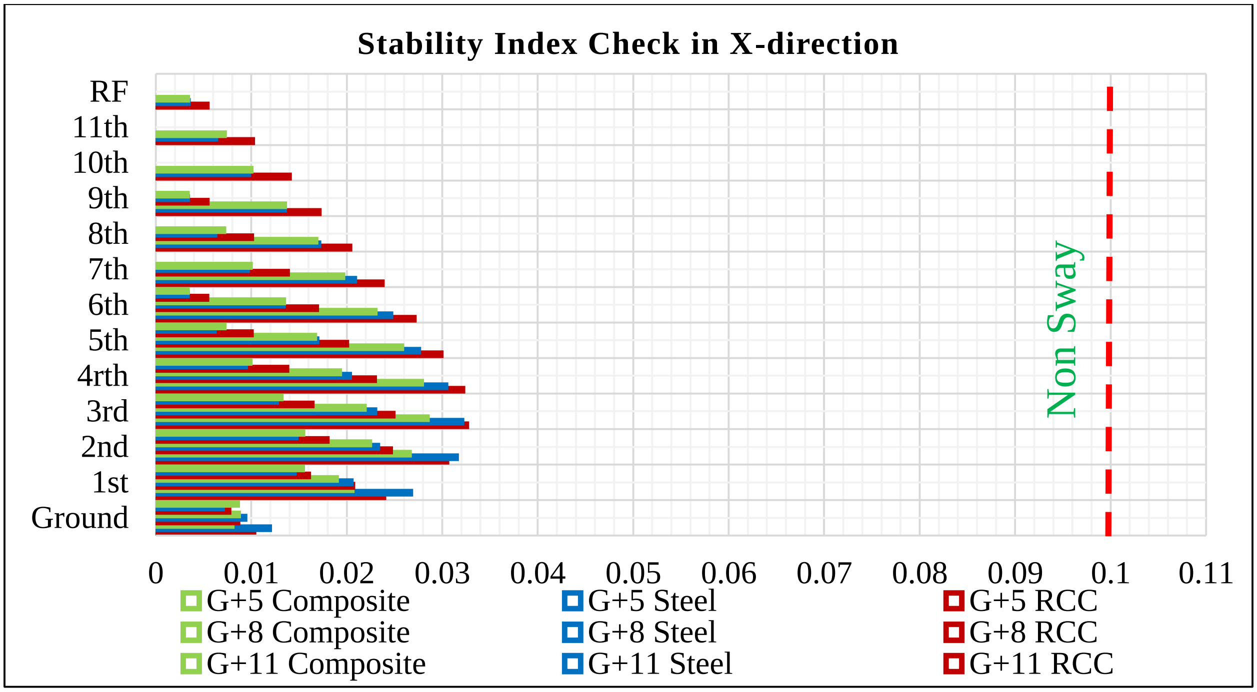


Figure 3. 27: Stability Index Check in X-direction for all Alternatives

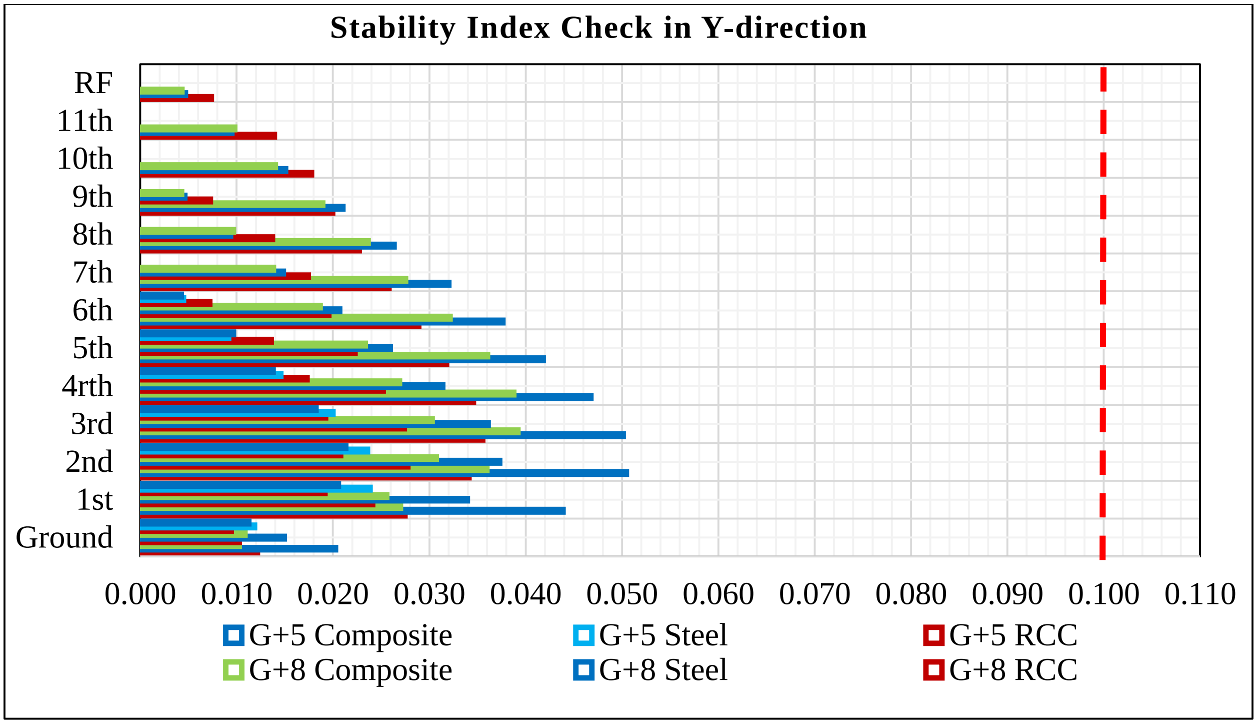


Figure 3. 28: Stability Index Check in Y-direction for all Alternatives

3.4.6.2 Damage Limitation

According to ES EN-8 section 4.4.3.1(1), the "damage limitation requirement" is considered to have been satisfied if under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the "no-collapse requirement" in accordance with section 2.1(1) and 3.2.1(3), the interstorey drifts are limited in accordance with section 4.4.3.2.

As per ES EN-8 section 4.4.3.2(1a), limitation of interstorey drift for buildings having non-structural elements of brittle materials attached to the structure is given by: $d_r v \leq 0.005h$.

Where: dr is the design interstorey drift as defined in 4.4.2.2(2); h is the storey height; and v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.

- ☞ The drift requirement checks for all structural alternatives is discussed briefly in Appendix-C from Table C.1 to Table C.9 and all structural alternatives are satisfied the damage limitation in both directions.

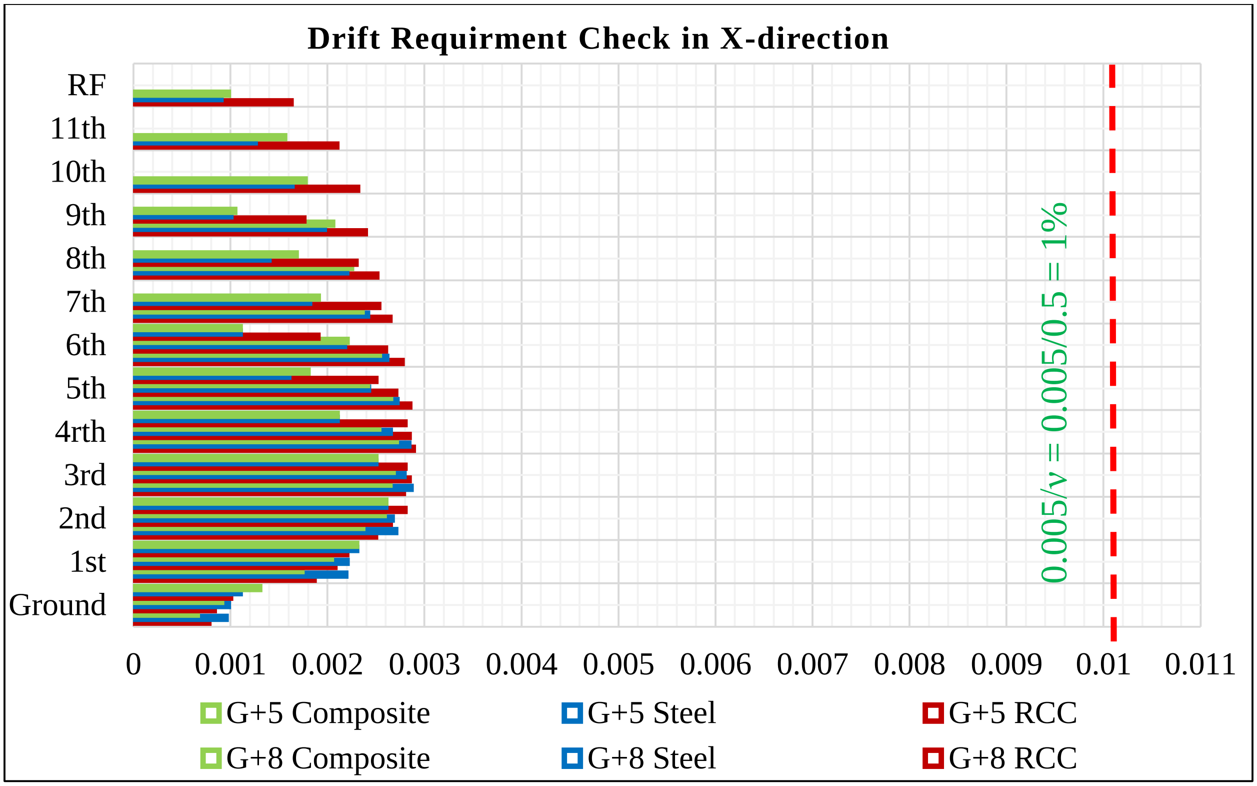


Figure 3. 29: Drift Requirement Check in X-direction for all Alternatives

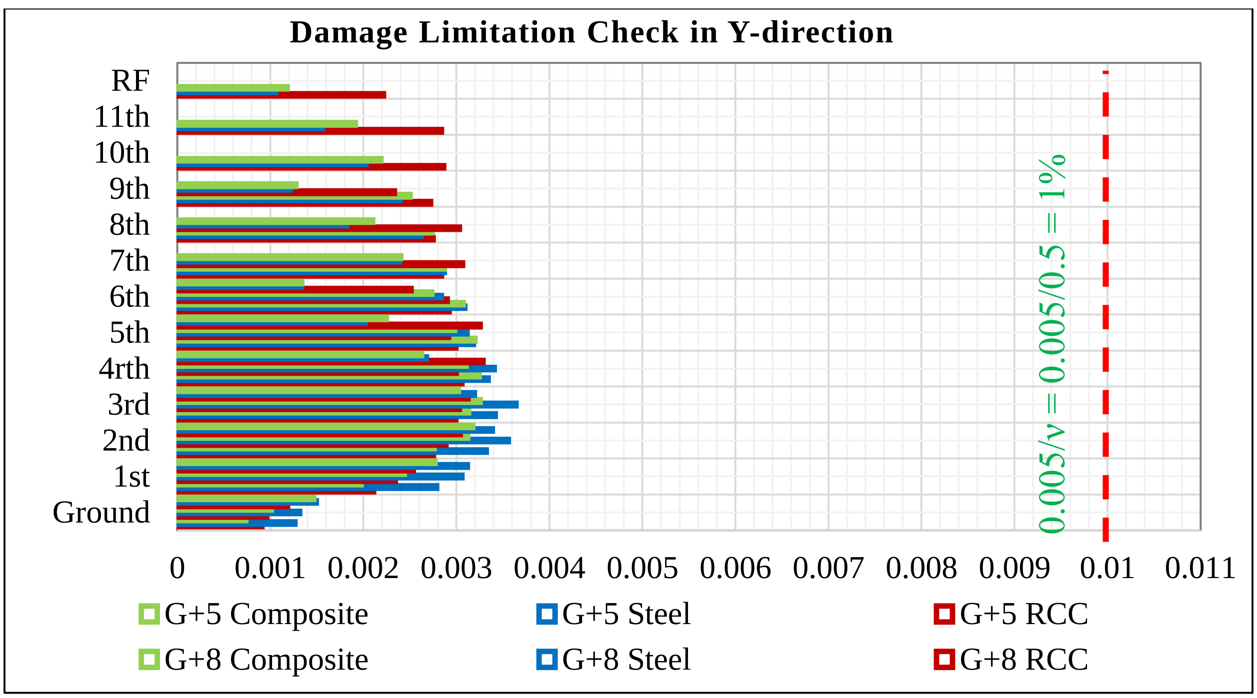


Figure 3. 30: Drift Requirement Check in Y-direction for all Alternatives

3.5 Design Formulation Based on Reference Codes

First, it is necessary to determine the internal forces acting on the structure, such as bending moments and shear forces. To do this one must define the different elements, assigning parameters such as geometry, material used and armoring preferences such as rebar diameter and covering.

Since the forces acting depend on the applied loads, and the weight of the structure varies depending on the assigned geometry, the process go through the analysis using several different configurations until the best fitting solution is found. Once the forces are completely defined is possible to proceed with the final design of the structural element and the general design of the building.

3.5.1 Design of Reinforced Concrete Alternative

The solution adopted for this structural alternative is based on stiff frames. Stiff frames are a good solution due to its solidity and durability. It is a statically indeterminate structure and hence the performance and efficiency of a rigid frame depends on the relative stiffness of beams and columns. For the system to work properly must ensure the proper functioning of knots, which must be sufficiently rigid and capable of transmitting bending moments. The fact that the armoring and concreting is carried out in-situ is an additional guarantee that the connections between beams and columns are carried out correctly. As height increases, the dimension of the structural elements also increases to support the extra load and maintain the correct behavior.

Concrete cover: The concrete cover over the reinforcement bars is held according with the statements done in ES EN-2 section 4.4.1 taking into consideration the durability of the elements. To properly design the correct cover, it is necessary to identify the environmental conditions for each case and the corresponding exposure class appealing to ES EN-2 in Table 4.1.

Spacing of bars: “The spacing of bars shall be such that the concrete can be placed and compacted satisfactorily for the development of adequate bond.” The recommendations for bars spacing and distribution are extracted from ES EN-2 section 8.2 (1).

3.5.1.1 Design of Reinforced Concrete Column

Columns is subjected to biaxial bending moments and axial load. Then, a proper design has to be done to withstand those loads. Since, shear forces will be present due to the interaction of the different elements such as beams and columns and it has to be taken into account as well.

a) Bending

Bending is considered taking into consideration the recommendations proposed in ES EN-2 section 6.1. As per ES EN-2 section 6.1 (4), for cross-sections loaded by the compression force it is necessary to assume the minimum eccentricity, e_o = h/30 but not less than 20 mm where h is the depth of the section. Since, biaxial moment is acting in the vertical element the above consideration has to be extended to both axes. Then, ES EN-2 section 5.8.9 takes into account that behavior defining the eccentricities from where the design is carried.

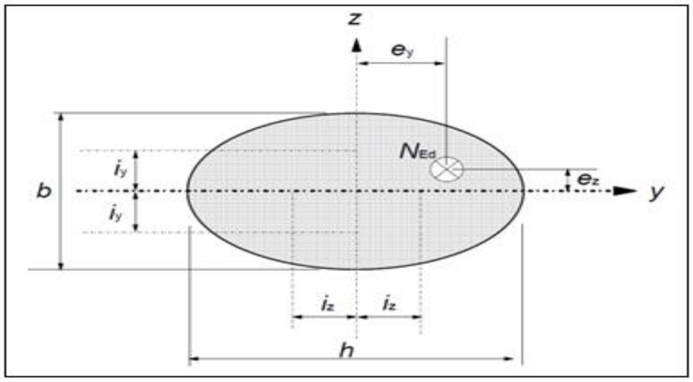


Figure 3. 31: Definition of eccentricities e_y and e_z (ES EN-2, Figure 5.8)

The design reinforcement to cope bending moments is design according to the recommendations in ES EN-2 section 9.5.2(2): "The total amount of longitudinal reinforcement should not be less than $A_{s,min}$". $A_{s,min} = \max\begin{cases}\frac{0.10N_{ED}}{f_{yd}} \\ 0.002A_c\end{cases}$. As per ES EN-2 section 9.5.2 (3), the area of longitudinal reinforcement should not exceed $A_{s,max}$. According to ES EN-2 section 9.5.2 (3), the recommended value is 0.04 Ac outside lap locations unless it can be shown that the integrity of concrete is not affected and that the full strength is achieved at ULS. This limit should be increased to 0.08 Ac at laps.

b) Shear

The shear reinforcement is available using fences and stirrups so as to cope traversal actions, following the recommendations of bent and disposition set out in sections 8.3 and 8.7 respectively on ES EN-2. The general design and verification for the shear forces are done according to ES EN-2 6.2.1, where the shear resistance of the element is defined as follows: $V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td}$

Where: V_{Rd} is the design shear resistance of the member without shear reinforcement; $V_{Rd,s}$ is the design value of the shear force which can be sustained by the yielding shear reinforcement; $V_{Rd,max}$ is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts; V_{ccd} is the design value of the shear component of the force in the compression area, in the case of an inclined compression chord and V_{td} is the design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord.

For structural elements not requiring design shear reinforcement, verifications have to be done following the rules stated in ES EN-2 6.2.2. In addition, minimum shear reinforcement must be placed according to formulation collected in ES EN-2 section 9.2.2. For structural elements requiring design shear reinforcement, verifications and design have to be done following the rules stated in ES EN-2 section 6.2.3 to obtain the necessary transversal reinforcement. Since, the specific elements in this particular case are columns, some rules have to be fulfilled for this kind of elements such as minimum bar diameter or bar spacing. These rules can be found on ES EN-2 section 9.5.3: (1) the diameter of the transverse reinforcement such as links, loops or helical spiral reinforcement should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars, whichever is the greater. The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5 mm. (2) the transverse reinforcement should be anchored adequately. (3) the spacing of the transverse reinforcement along the column should not exceed $S_{cl,\ tmax}$ which can be determined as the least of: 20 times the longitudinal reinforcement diameter; the lesser dimension of the column and 400 mm. (4) the maximum spacing required in (3) should be reduced by a factor 0.6: (i) in sections within a distance equal to the larger dimension of the column cross-section above or below a beam or slab; (ii) near lapped joints, if the maximum diameter of the longitudinal bars is greater than 14 mm. A minimum of 3 bars evenly placed in the lap length is required. (5) where the direction of the longitudinal bars changes, (e.g. at changes in column size), the spacing of transverse reinforcement should be calculated, taking account of the lateral forces involved. These effects may be ignored if the change of direction is less than or equal to 1 in 12. (6) every longitudinal bar or bundle of bars placed in a corner should be held by transverse reinforcement. No bar within a compression zone should be further than 150 mm from a restrained bar.

After running structural analysis trying different options and configurations, the geometry and reinforcement distribution of the columns is shown in figure 3.15.

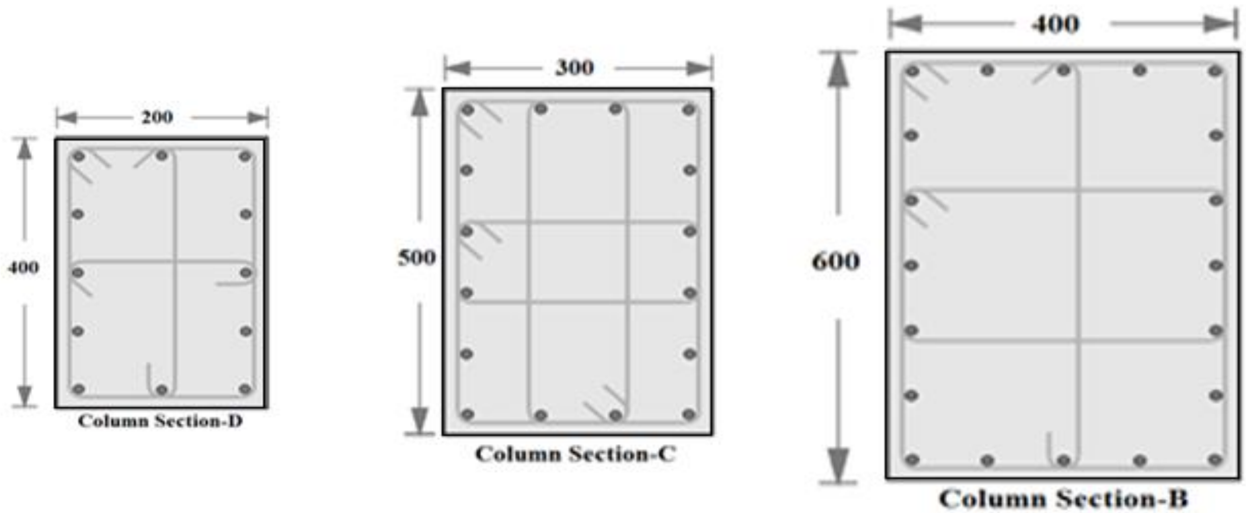


Figure 3. 32: Final Column Design Sections (ETABS 2016)

3.5.1.2 Design of Reinforced Concrete Beam

Beams is subjected to bending moments and shear forces due to the structural configuration of the building. As done for the column case, the design criteria follow the recommendations stablished in Ethiopian Standard-2.

a) Bending

Bending is considered taking into consideration the recommendations proposed in ES EN-2 6.1 to determine the ultimate bending resistance of the element. To do it, some assumptions is made:

- Plane sections remain plane and the strain in bonded reinforcement or bonded pre-stressing tendons, whether in tension or in compression, is the same as that in the surrounding concrete. The tensile strength of the concrete is ignored.
- The stresses in the concrete in compression are derived from the design stress/strain relationship (ES EN-2 3.1.7).
- The stresses in the reinforcing or pre-stressing steel are derived from the design curves in ES EN-2 3.2 and 3.3. The initial strain in pre-stressing tendons is taken into account when assessing the stresses in the tendons.

The design is done according to the ultimate limit states (ULS) configuration. The determination of the maximum and minimum longitudinal reinforcement is done following the specifications in ES EN-2 9.2.1.1: "The area of longitudinal tension reinforcement should not be taken as less than

As, min.” $A_{s,min} = 0.26 * \frac{f_{ctm}}{f_{yk}} * b_t * d$ But not less than $0.0013 b_t d$. “The cross-sectional area of tension or compression reinforcement should not exceed $A_{s,max}$ outside lap locations. The recommended value is 0,04Ac.”

b) Shear

In this case, the same general approach adopted for columns can be applied for beams since stirrups are also used to collect shear stresses. The general design approach is collected in ES EN-2 6.2. For structural elements not requiring design shear reinforcement, verifications have to be done following the rules stated in ES EN-2 6.2.2. In addition, minimum shear reinforcement must be placed according to formulation collected in ES EN-2 9.2.2.

The minimum reinforcement per unit of length A_{sw} can be computed by imposing the minimum value of the shear reinforcement ratio, by using expressions (9.4) in ES EN-2 9.2.2. For structural elements requiring design shear reinforcement, verifications and design have to be done following the rules stated in ES EN-2 6.2.3 to obtain the necessary transversal reinforcement. As per ES EN-2 9.2.2, the maximum separation between shear assemblies should not exceed $S_{l,max}$. $S_{l,max} = 0.75d(1 + \cot\alpha)$ Where, α is the inclination of the shear reinforcement to the longitudinal axis of the beam. The distribution and geometry of reinforced concrete beams corresponding to the top tie beam and floor beams shows in figure 3.16.

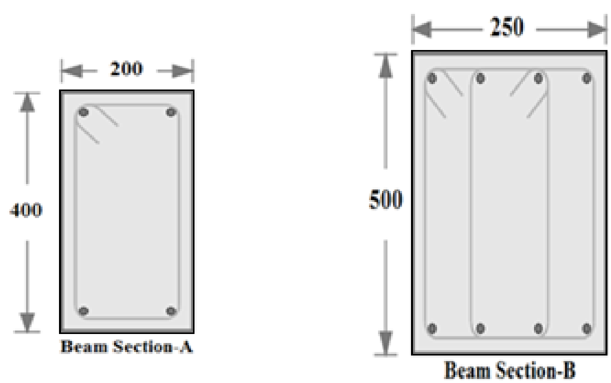


Figure 3. 33: Final Beam Design Sections (ETABS 2016)

3.5.1.3 Design of Reinforced Concrete Solid Slab

Another structural element to consider is the floor system. There are a variety of types of slabs according to whether transmit loads in one or two directions. According to the transmission of loads there are unidirectional or bidirectional slabs. Since, the frame system is bidirectional and the considered lateral loads can act in either X or Y direction is logical to consider a bidirectional slab system.

In this case, the commonly used bidirectional slab is two-way solid slabs with embedded beams. It has been used in a lot of structures for their constructive simplicity. Slabs have to withstand bending moments and shear forces provoked by the other structural elements. ES EN-2 9.3 describes the design rules for solid slabs. That section applies to one-way and two-way solid slabs for which b and l_{eff} are not less than 5h.

Shear reinforcement design for slab is done according to ES EN-2 9.3.2. The most important factor to take into account in this case is the depth of the slab. As stated in that section: “A slab in which shear reinforcement is provided should have a depth of at least 200 mm”. In this cause, shear reinforcement of the slab is not provided because depth of the slab is 150 mm.

3.5.2 Structural Steel Alternative

The design criteria of structural steel follow the recommendations stablished in Ethiopian Standard-3, it is also possible to create a frame system with this material. To get completely rigid frames are necessary to ensure that the joints between the beams and columns are suitable to consider it rigid.

Unions are one of the most important parts of a steel structure due to their important role in defining the structure behavior in front of the actions. During modelling unions is considered as rigid joints to be able to create the frame system but they are not being specifically designed. Steel unions design is out of the scope of this thesis.

As mentioned before, the frame system is formed by the interaction of beams and columns and there are different solutions that can be used for steel construction. The first strategy consists in using a molded steel profile to act as a column. Built-up sections are used for structural steel alternative and model as shown figure 3.17.

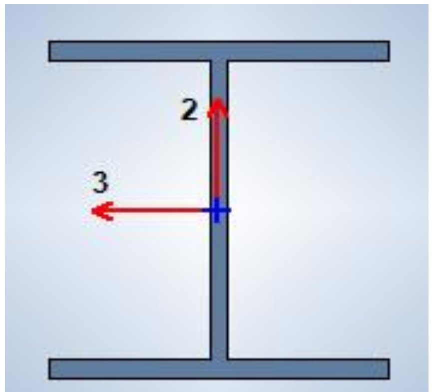


Figure 3. 34: Built-up Structural Steel Section (ETABS 2016)

3.5.2.1 Design of Structural Steel Column

Steel columns are used in the structural design of the buildings and the applicable standard in this case is Ethiopian standard-3.

a) Tension

(1) P The design value of the tension force N_{Ed} at each cross section shall satisfy: $\frac{N_{Ed}}{N_{t.Rd}} \leq 1.0$.

(2) For sections with holes the design tension resistance Nt,Rd should be taken as the smaller of:

a) the design plastic resistance of the gross cross-section: $N_{pl.Rd} = \frac{Af_y}{\gamma_{MO}}$

b) the design ultimate resistance of the net cross-section at holes for fasteners: $N_{u.Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}}$

(3) Where capacity design is requested, see EBCS EN 1998: 2013, the design plastic resistance $N_{pl,Rd}$ (as given in 6.2.3(2) a)) should be less than the design ultimate resistance of the net section at fasteners holes $N_{u,Rd}$ (as given in 6.2.3(2) b)).

(4) In category C connections (see ES EN-3 3.4.1(1), the design tension resistance $N_{t,Rd}$ in 6.2.3(1) of the net section at holes for fasteners should be taken as $N_{net,Rd}$, where: $N_{net.Rd} = \frac{A_{net}f_u}{\gamma_{M0}}$.

(5) For angles connected through one leg, see also EBCS EN 1993-1-8: 2013, 3.10.3. Similar consideration should also be given to other type of sections connected through outstands.

b) Compression

(1) P The design value of the tension force N_{Ed} at each cross section shall satisfy: $\frac{N_{Ed}}{N_{c.Rd}} \leq 1.0$.

(2) The design resistance of the cross-section for uniform compression Nc,Rd should be determined as follows: $N_{c.Rd} = \frac{Af_y}{\gamma_{MO}}$ for class 1, 2 or 3 cross section

$$M_{c.Rd} = M_{el.Rd} = \frac{A_{eff}\, f_y}{\gamma_{MO}} \quad \text{for class 4 cross section}$$

(3) Fastener holes except for oversize and slotted holes as defined in EN 1090 need not be allowed for in compression members, provided that they are filled by fasteners.

(4) In the case of unsymmetrical Class 4 sections, the method given in 6.2.9.3 should be used to allow for the additional moment ΔM Ed due to the eccentricity of the centroidal axis of the effective section, see 6.2.2.5(4).

3.5.2.2 Design of Steel Girder and Secondary Steel Beam

a) Bending

The design to make front the different bending actions is done following the rules collected in ES EN-3 section 6.2.5. The most important rules are: "The design value of the bending moment M_{ed} at each cross-section shall satisfy:" $\frac{M_{ed}}{M_{c.Rd}} \leq 1.0$.

"The design resistance for bending about one principal axis of a cross-section is determined as:

$$M_{c.Rd} = M_{pl.Rd} = \frac{W_{pl} f_y}{\gamma_{MO}} \quad \text{for class 1 or 2 cross section}$$

$$M_{c.Rd} = M_{el.Rd} = \frac{W_{el,min}\, f_y}{\gamma_{MO}} \quad \text{for class 3 cross section}$$

$$M_{c.Rd} = \frac{W_{eff,minl}\, f_y}{\gamma_{MO}} \quad \text{for class 4 cross section}$$

"For bending about both axes, the methods given in ES EN-3 6.2.9 should be used"

b) Shear

Shear design is held according to ES EN-3 6.2.6 and the most important considerations are: "The design value of the shear force V_{ed} at each cross section shall satisfy": $\frac{V_{ed}}{V_{c.Rd}} \leq 1.0$.

Where $V_{c.Rd}$ is the design shear resistance. For plastic design $V_{c.Rd}$ is the design plastic shear resistance $V_{pl.Rd}$ and is given by: $V_{pl,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{MO}}$. "For verifying the design elastic shear resistance $V_{c.Rd}$ the following criterion for a critical point of the cross section may be used unless the buckling verification applies”: $\frac{\tau_{ed}}{f_y/(\sqrt{3}\,\gamma_{MO})} \leq 1.0$. Where, τ_{ed} may be obtained from $\tau_{ed} = \frac{V_{ed}\,S}{I\,t}$

Where: V_{ed} is the design value for the shear force; S is the first moment of area about the centroidal axis of that portion of cross-section between the point at which the shear is required and the boundary of the cross-section; I is second moment of area of the whole cross section and t is the thickness at the examined point.

c) Bending and Shear Interaction

When these two actions are applied simultaneously or are susceptible to act like that, it is necessary to assess the resistance of the cross-section against this interaction.

Ethiopian Standard-3 takes it into account in section 6.2.8: (1) where the shear force is present allowance should be made for its effect on the moment resistance. (2) Where the shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected except where shear buckling reduces the section resistance. (3) Otherwise the reduced moment resistance should be taken as the design resistance of the cross-section, calculated using a reduced yield strength $(1-\rho)f_y$ for the shear area, where $\rho = (\frac{2V_{ed}}{V_{pl,Rd}} - 1)^2$. (4) When torsion is present p should be obtained from $\rho = (\frac{2V_{ed}}{V_{pl,Rd}} - 1)^2$, see ES EN-3 6.2.7, but should be taken as 0 for $V_{ed} \leq 0.5V_{pl,T,Rd}$.

Table 3. 14: Composite Beam Design and Properties (ETABS 2016)

Strength Checks					
	Combo	E_d	R_d	Ratio	Pass
Shear at Ends (kN)	DCmpS2	48.2189	149.2450	0.323	✓
Construction Bending (kN-m)	19. COMB---Envelope (Maximum Effect in RS)	70.8869	132.0000	0.537	✓
Positive Bending (kN-m)	DCmpS2	70.8869	132.0000	0.537	✓

Constructability and Serviceability Checks

	Actual	Allowable	Ratio	Pass
Constr. Dead Defl. (mm)	8.6	No Limit	N/A	N/A
Post-concrete Defl. (mm)	5.9	20	0.293	✓
Live Load Defl. (mm)	3.5	16.7	0.211	✓
Total Defl. (mm)	10	20.8	0.478	✓

3.5.2.3 Design of Composite Slab

In this case, slabs are composite members that are designed according to Ethiopian Standared-4. The section that describes the behavior of these slabs and dictates the rules for their design is ES EN-4 section 9. Some basic design parameters are followed as per ES EN-4 section 9.2.1.

The overall depth of the composite slab h shall be not less than 80 mm. The thickness of concrete h_c above the main flat surface of the top of the ribs of the sheeting shall be not less than 40 mm. (2) If the slab is acting compositely with the beam or is used as a diaphragm, the total depth shall be not less than 90 mm and h_c shall be not less than 50 mm. (3) Transverse and longitudinal reinforcement shall be provided within the depth hc of the concrete. (4) The amount of reinforcement in both directions should not be less than 80 mm^2/m. (5) The spacing of the reinforcement bars should not exceed 2h and 350 mm, whichever is the lesser.

3.5.3 Steel-concrete Composite Alternative

Composite columns are the best choice due to their regular geometry in both directions and major resistance. There are mainly two types of columns are used in composite structures such as concrete filled steel tubes and fully encased or partially encased columns. For construction simplicity and better general performance, it is considered a better choice to use tubular composite profiles instead of embedded profiles. Embedded profiles require formworks and a bigger amount of steel, which is more expensive than concrete. For that reason, composite rectangular tubular columns will be considered during the design phase.

In this research work, concrete filled steel tube columns of rectangular sections are used. In this case, concrete is later filled in the tubular steel columns. The collaboration of both steel and concrete is used to attain their capabilities in construction in a most effective manner. Both steel and concrete shares same frictional bond they make them glued together in a composite column. They resist the application of external forces and also bears initial loads at the earlier time of construction thus also generally acting as supports which may reduce setup such as supports and shuttering which is required initially at the time of construction before filling them with concrete. Both types of steel-concrete composite columns are shown in Figure 3.19.

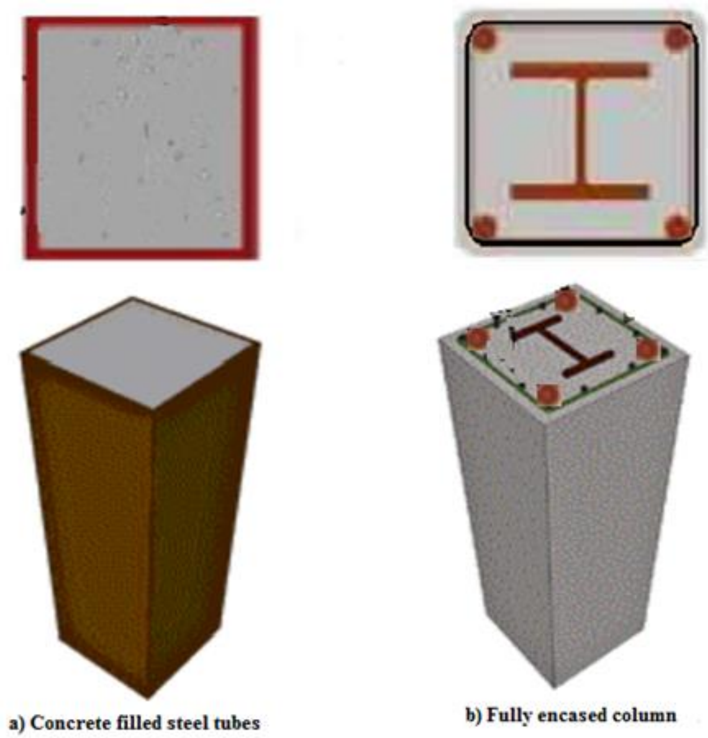


Figure 3. 35: Composite Columns: a) Concrete Filled Steel Tubes and b) Fully Encased Columns

With regarding to the beams, similar to structural steel alternatives built-up steel sections are used for girder and composite beams.

3.5.3.1 Design of Steel-concrete Composite Column

Composite steel-concrete columns are used in the structural design of the buildings and the applicable standard in that case is Ethiopian Standard-4. Composite column members have a determinate maximum and minimum amount of steel contribution to be considered as composite.

If not, the column can be considered as a concrete column or as a steel column. This range is defined in ES EN-4 6.7.1 (4).

$$0.2 \leq \delta \leq 0.9$$

$$\delta = \frac{A_a f_{yd}}{N_{pl,Rd}}$$

There are two design methods according ES EN-4 6.7.1 (6):

- A general method in ES EN-4 6.7.2 whose scope includes members with nonsymmetrical or non-uniform cross-sections over the column length and

- A simplified method in ES EN-4 6.7.3 for members of doubly symmetrical and uniform cross section over the member length.

In this particular case, the simplified method is applicable to design the columns. The resistance of the cross section of the columns should be evaluated according the criteria stated in ES EN-4 6.7.3.2 and taking into consideration the M-N interaction diagram ES EN-4 6.7.3.2 (5) figure 6.19.

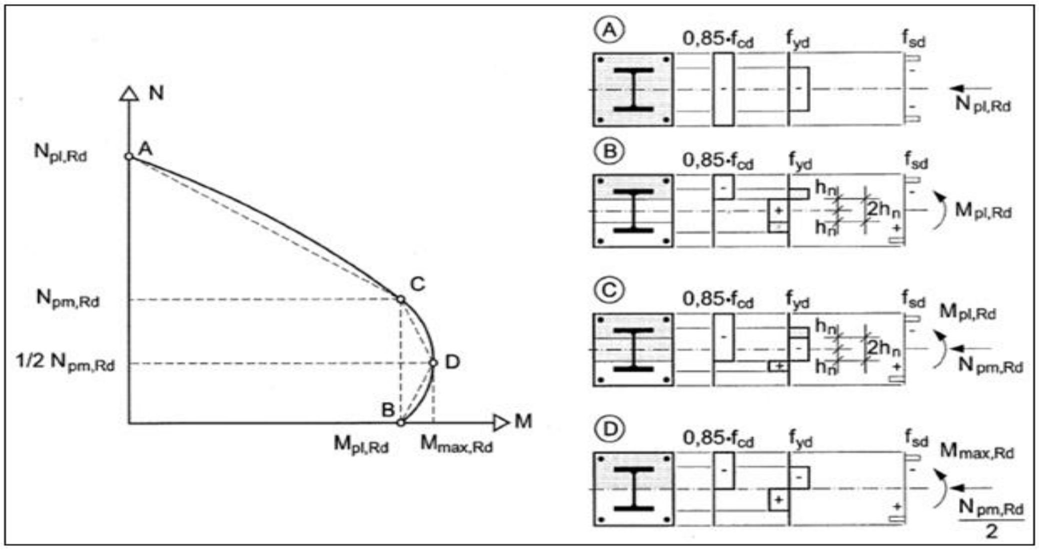


Figure 3. 36: Interaction Curve and Corresponding Stress Distributions (ES EN-4, Fig. 6.19)

a) Bending

Columns are subjected to compression and biaxial bending due to the frame system configuration. The considerations for the design and verification of these actions are taken into account in section 6.7.3.7 of the Ethiopian Standard-4: (1) For composite columns and compression members with biaxial bending the values μ_{dy} and μ_{dz} in ES EN-4 Figure 6.20 may be calculated according to ES EN-4 6.7.3.6 separately for each axis. Imperfections should be considered only in the plane in which failure is expected to occur. If it is not evident which plane is the more critical, checks should be made for both planes. (2) For combined compression and biaxial bending the following conditions should be satisfied for the stability check within the column length and for the check at the end: $\frac{M_{y,Ed}}{\mu_{dy}\, M_{pl,y,Rd}} \leq \alpha_{M,y}$ $\qquad$ $\frac{M_{z,Ed}}{\mu_{dz}\, M_{pl,z,Rd}} \leq \alpha_{M,z}$

$$\frac{M_{y,Ed}}{\mu_{dy}\, M_{pl,y,Rd}} + \frac{M_{z,Ed}}{\mu_{dz}\, M_{pl,z,Rd}} \leq 1$$

Where, $M_{pl,y,Rd}$ and $M_{pl,y,Rd}$ are the plastic bending resistances of the relevant plane of bending; $M_{y,Ed}$ and $M_{z,Ed}$ are the design bending moments including second-order effects and imperfections according to 6.7.3.4; μ_{dy} and μ_{dz} are defined in ES EN-4 6.7.3.6 and $\alpha_{M,y} = \alpha_M$; $\alpha_{M,y} = \alpha_M$ are given in ES EN-4 6.7.3.6 (1)

b) Shear

Shear in composite columns is taken into account in ES EN-4 section 6.7.4.2(3) for composite columns and compression members no shear connection need be provided for load introduction by end plates if the full interface between the concrete section and endplate is permanently in compression, taking account of creep and shrinkage. Otherwise the load introduction should be verified according to section 6.7.4.2(5).

To consider longitudinal shear and the interaction between concrete and steel on the interface, recommendations given in ES EN-4 6.7.4.3: (1) outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified where it is caused by transverse loads and /or end moments. Shear connectors should be provided based on the distribution of the design value of longitudinal shear, where this exceeds the design shear strength τ_{Rd}. (2) in absence of a more accurate method, elastic analysis, considering long term effects and cracking of concrete, may be used to determine the longitudinal shear at the interface. (3) Provided that the surface of the steel section in contact with the concrete is unpainted and free from oil, grease and loose scale or rust, the values given in ES EN-4 Table 6.6 may be assumed for τ_{Rd}.

Table 3. 15: Design Shear Strength (ES EN-4, Table 6.6)

Type of cross section	**τ_{Rd} (N/mm^2)**
Completely concrete encased steel sections	0.30
Concrete filled circular hollow sections	0.55
Concrete filled rectangular hollow sections	0.40
Flanges of partially encased sections	0.20
Webs of partially encased sections	0.00

3.5.3.2 Design of Steel Girder and Secondary Steel Beam

For this alternative, the design procedure of girder beam and secondary beams or joists is done following exactly the same recommendations and rules as stated in section 3.5.2.2 of this paper.

3.5.3.3 Design of Composite Slab

For this alternative, the slab design is done following exactly the same recommendations and rules stated for the reinforced concrete alternative in section 3.5.2.3 of this paper.

3.6 Material Usage

Once the structural analysis and design of all alternatives is done, the next step is to evaluate the total direct cost of the structural frame by considering the price of the mere material for each alternative. Comparison, analysis and discussion have been performed using all data obtained from the selected nine (9) structural modeling types of the same plan for the same function.

Material usage of structural materials such as reinforced concrete, structural steel and composite alternative is exported from ETABS software to the Microsoft excel software as a tabular form. The packaged software only gives the total quantity of materials such as concrete and structural steel. The total material usage for each alternative is discussed in Appendix-B. The quantity of formwork also calculated using MS-Excel program as shown in Appendix-B.

To evaluate the cost of materials in an adequate and reliable manner it is necessary to use current price databases created by the competent authorities. These databases realistically reflect the current market prices of the different construction materials and their associated costs.

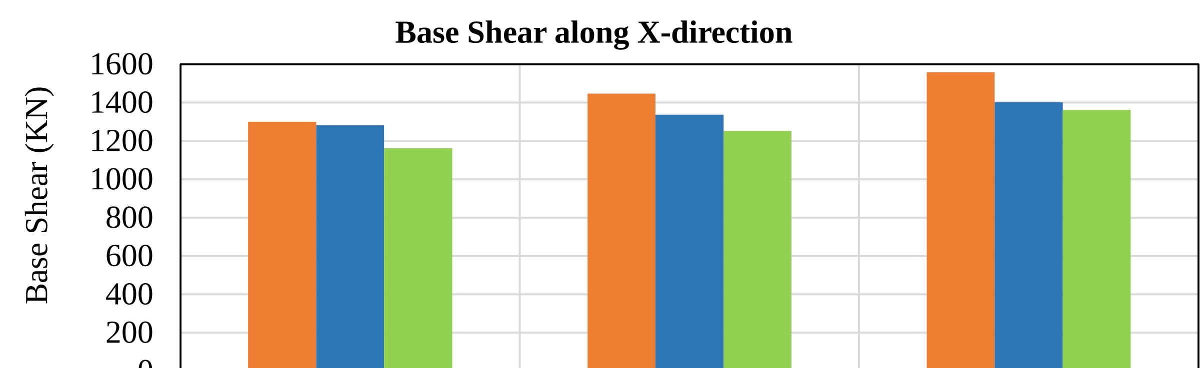
CHAPTER FOUR

RESULTS AND DISCUSSIONS

The story responses such as base shear, storey drift, storey displacements, time period and modal frequency; self-weight and direct cost analysis are studied and comparison of these are done. Results of response spectrum analysis have been used to observe and compare story response of all models. The results and discussions are recorded in terms of tables, charts and graphs in the coming paragraphs.

4.1 Base Shear

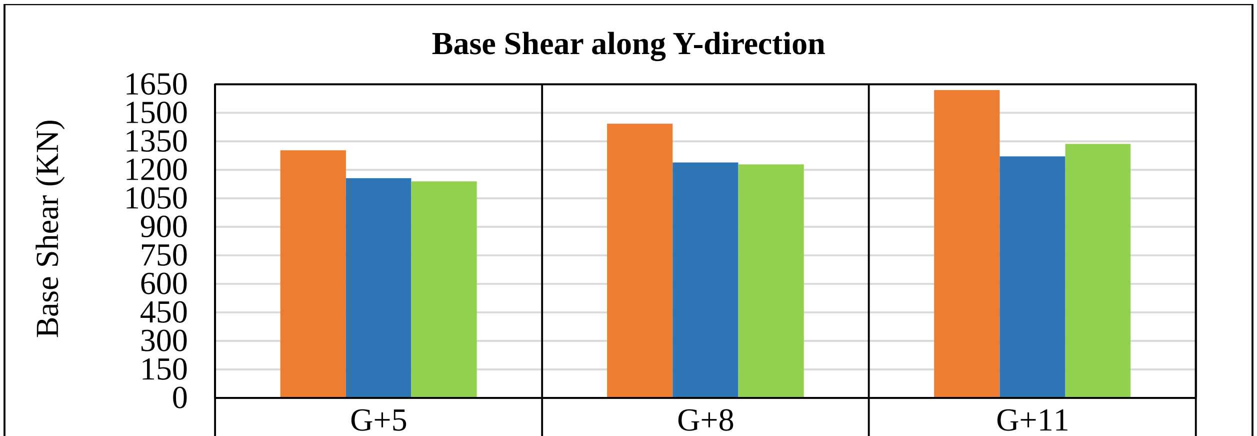
Base shear is an estimate of the expected maximum lateral force that would occur due to land seismic movement at the base of the structure. The base shear is the basic parameter for deciding the earthquake resistant structure. To make the structure safe, the base shear should be kept as low as possible. The results of base shear for all structural alternative is recorded in figure 4.1 and 4.2 for both directions.



	G+5	G+8	G+11
RCC	1298.95	1446.68	1557.70
Steel	1280.87	1336.05	1401.11
Composite	1162.14	1251.56	1362.21

Figure 4. 1: Comparison of Base Shear along X-Direction for each Structural Alternatives

From the above chart the results show that; In case of G+5 buildings, the base shear is increased by 10.53% for reinforced concrete, and 9.27% for structural steel as compared with composite alternative whereas reinforced concrete alternative is increased by 1.39% as compared with structural steel alternatives. In case of G+8 buildings, the base shear is increased by 13.49% for reinforced concrete, and 6.32% for structural steel as compared with composite alternative whereas reinforced concrete alternative is increased by 7.65% as compared with steel alternatives. In case of G+11 buildings, the base shear is increased by 12.55% for reinforced concrete, and 2.78% for structural steel as compared with composite alternative whereas reinforced concrete alternative is increased by 10.05% as compared with structural steel alternatives.



	G+5	G+8	G+11
RCC	1303.0506	1443.9766	1620.6598
Steel	1157.3174	1238.9493	1270.9647
Composite	1139.8951	1228.9848	1337.3052

Figure 4. 2: Comparison of Base Shear along Y-Direction for each Structural Alternatives

From the above chart the results of base shear show that; In case of G+5 buildings, the base shear is increased by 12.52% for reinforced concrete, and 1.51% for steel as compared with composite alternative whereas reinforced concrete alternative is increased by 11.18% as compared with steel alternatives. In case of G+8 buildings, the base shear is increased by 14.89% for reinforced concrete, and 0.80% for structural steel as compared with composite alternative whereas reinforced concrete alternative is increased by 14.20% as compared with steel alternatives. In case of G+11 buildings, the base shear is increased by 17.48% for reinforced concrete, and decreased by 4.96% for steel compared with composite alternative whereas reinforced concrete is increased by 21.58% compared with steel alternatives. Based on the above results, it is found that the base shear is more in case of reinforced concrete and minimum in case of structural steel due to the more seismic weight of reinforced concrete frame as represented by Figure 4.1 and 4.2. Reason being as the weight increases, base shear values are also boosted.

4.2 Story Drift

Drift is the lateral displacement of one level of multi-story building relative to the other level above or below it. According to ES EN-8 section 4.4.3.2(1a) the limitation of interstory drift for buildings having non-structural elements of brittle materials attached to the structure is given by: $d_r v \leq 0.005h$. Where, d_r is the design interstorey drift as defined in 4.4.2.2(2), h is the storey height and v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. In this case, the building is classified as importance class II and the corresponding reduction factor v is 0.5 as per ES EN-8 section 4.4.3.2(2). The story height is 3m then the interstory drift is limited to: $d_r = \frac{0.005*h}{v} = 0.03$. The results of maximum story drift are recorded from figure 4.3 to figure 4.11 in both directions.

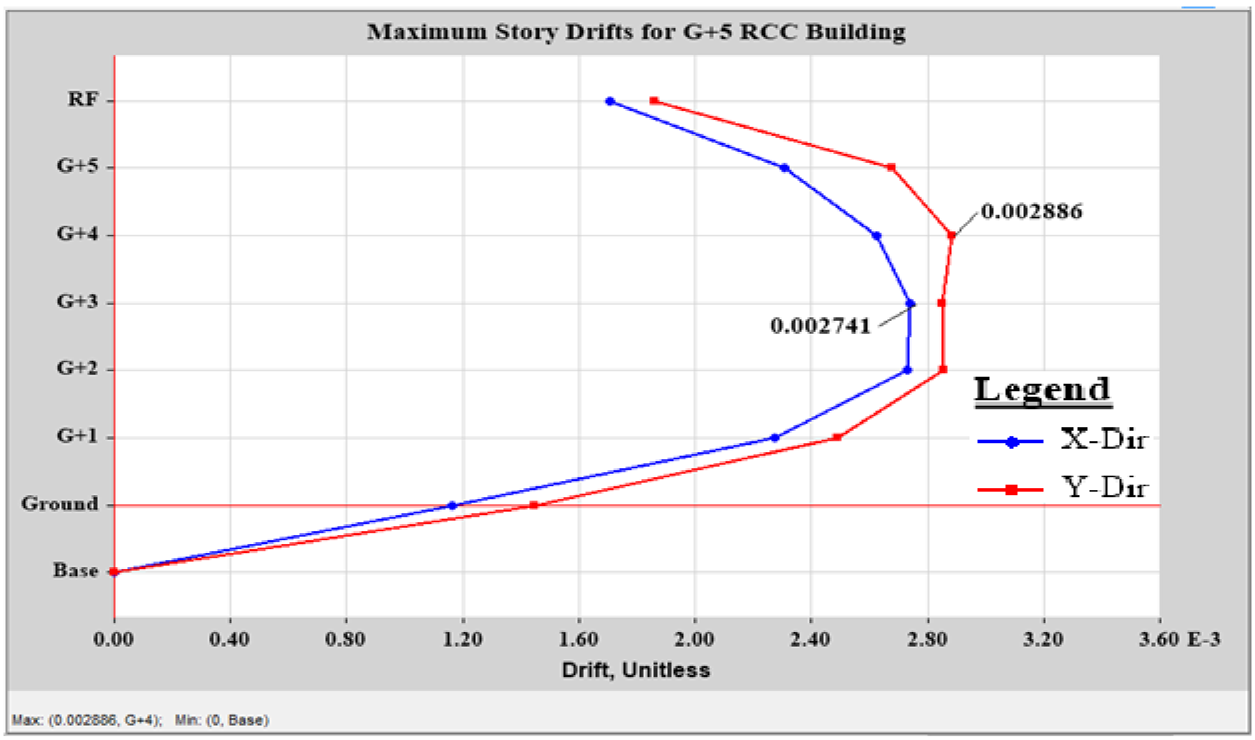


Figure 4. 3: Maximum Story Drift for G+5 Building in case of RCC Alternative

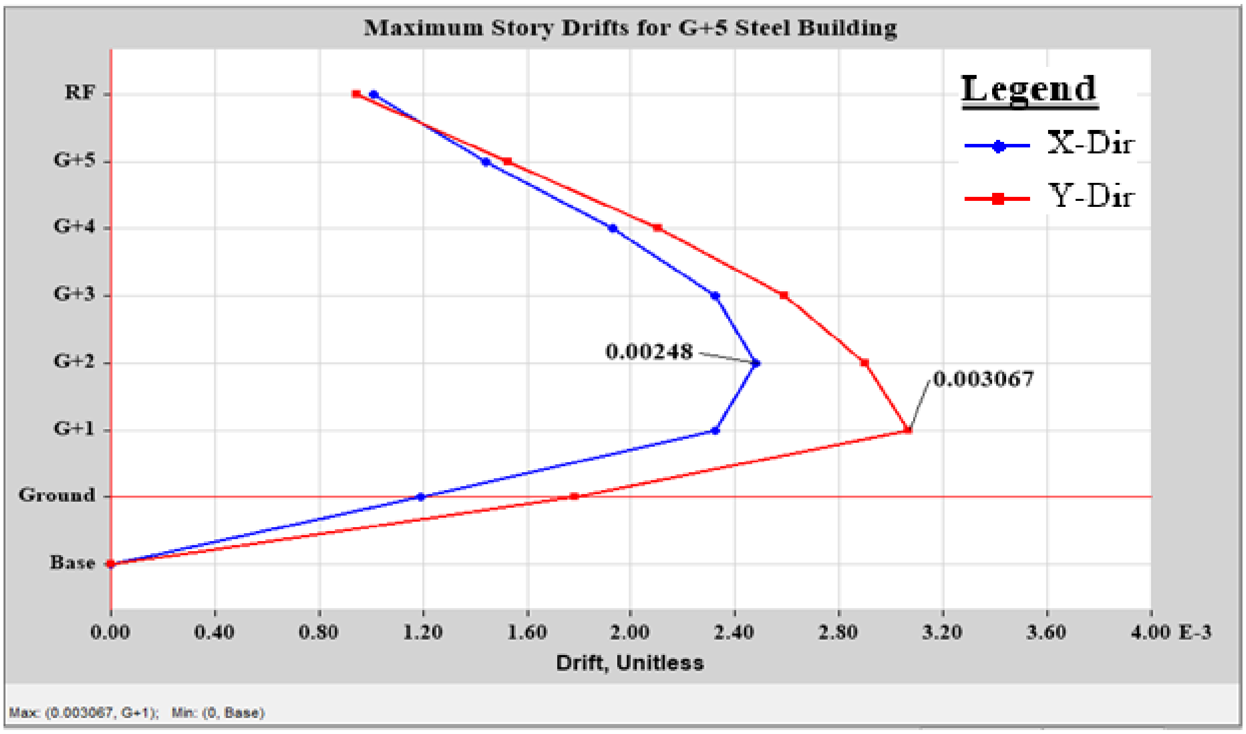


Figure 4. 4: Maximum Story Drift for G+5 Building in case of Steel Alternative

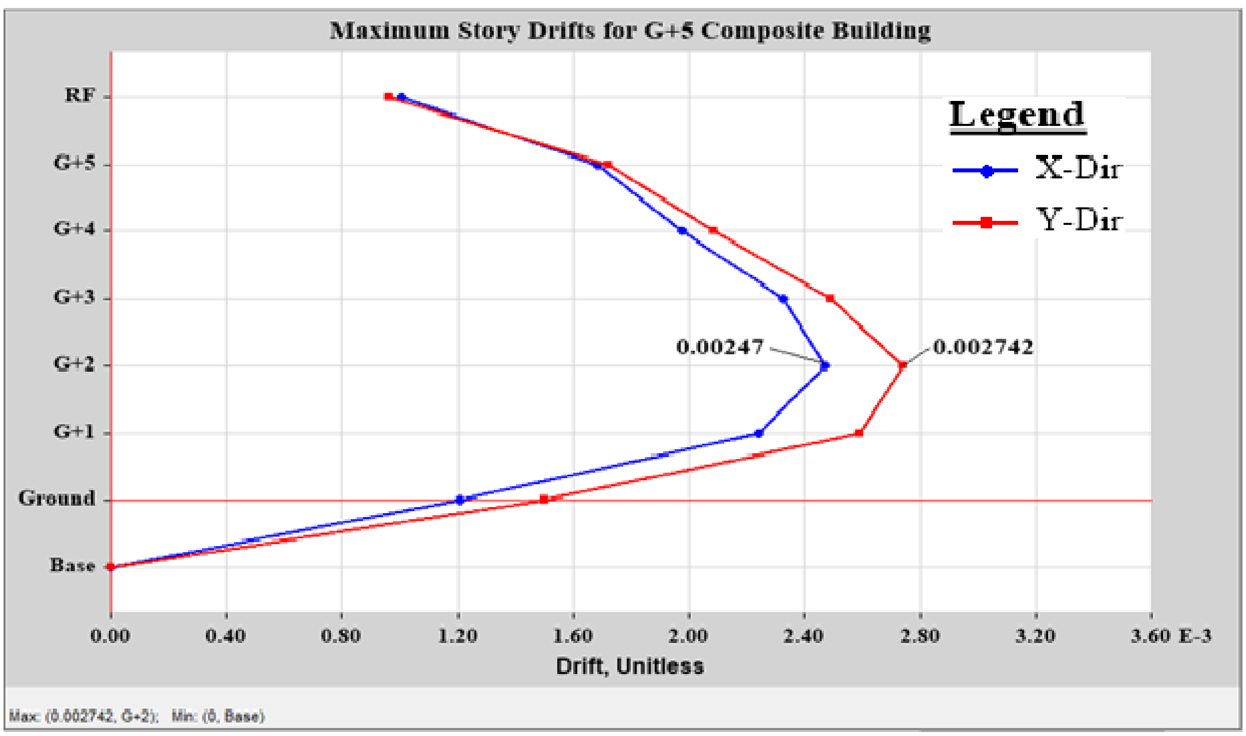


Figure 4. 5: Maximum Story Drift for G+5 Building in case of Composite Alternative

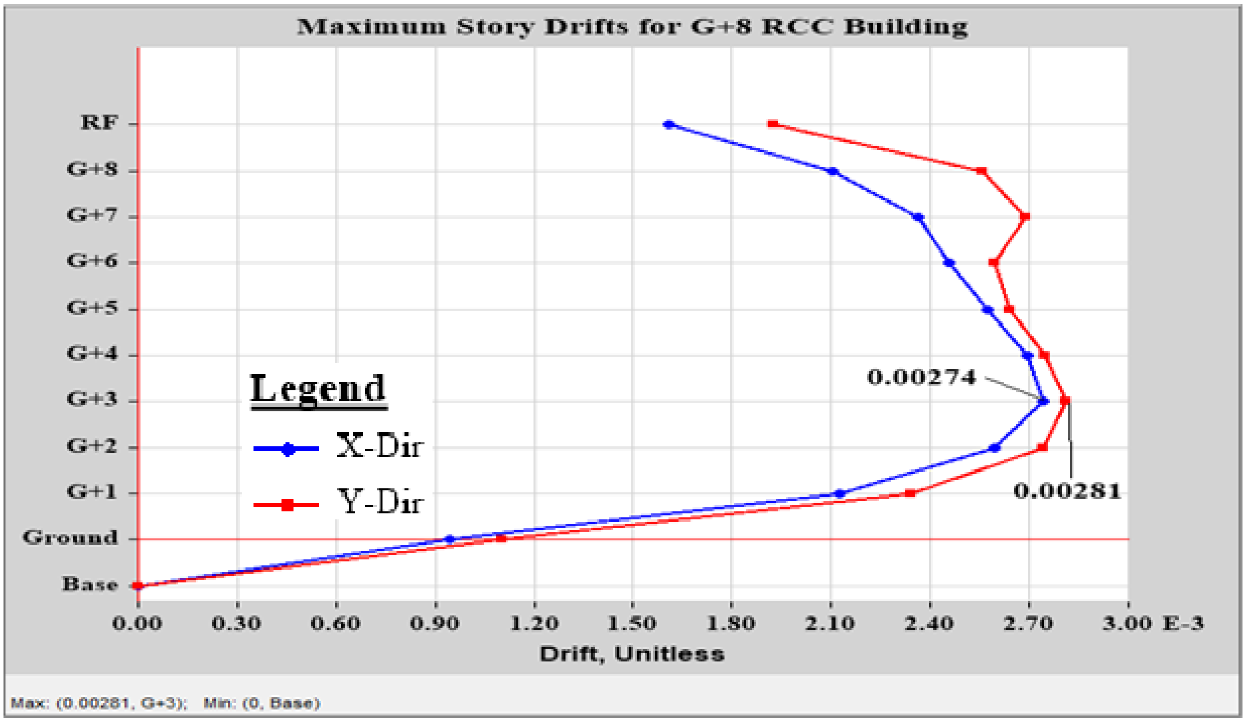


Figure 4. 6: Maximum Story Drift for G+8 Building in case of RCC Alternative

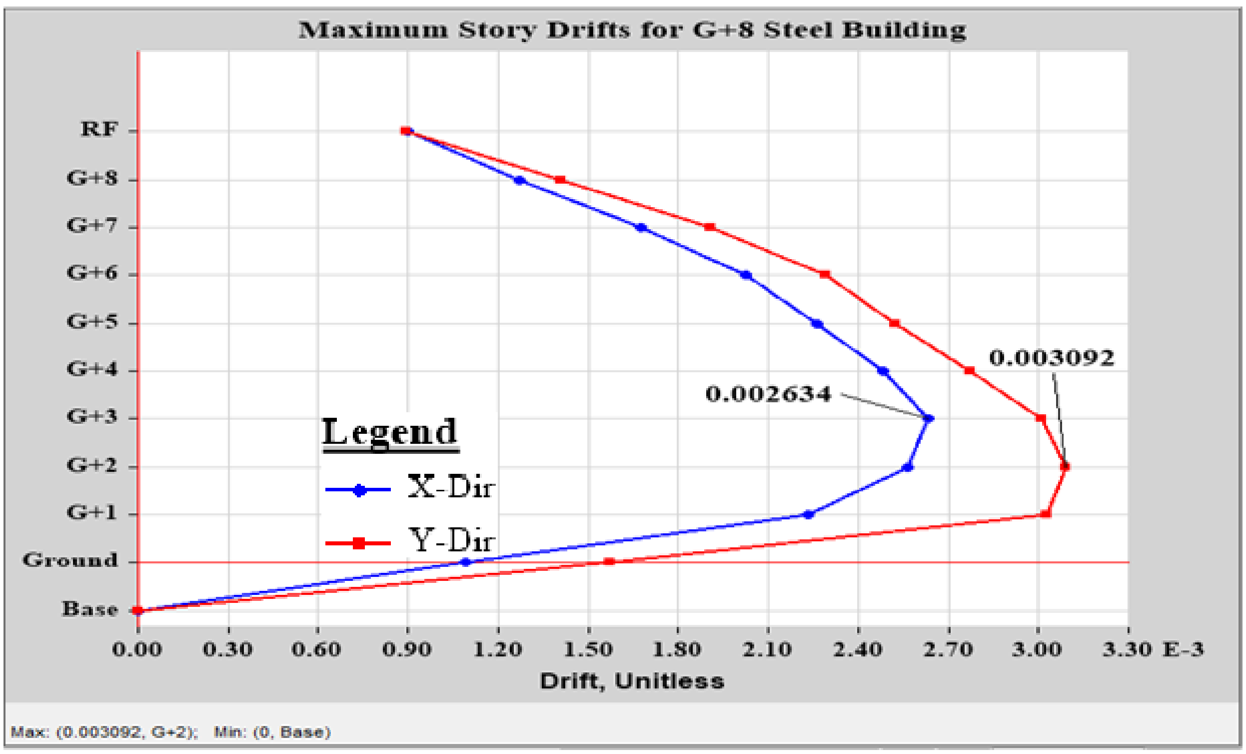


Figure 4. 7: Maximum Story Drift for G+8 Building in case of Steel Alternative

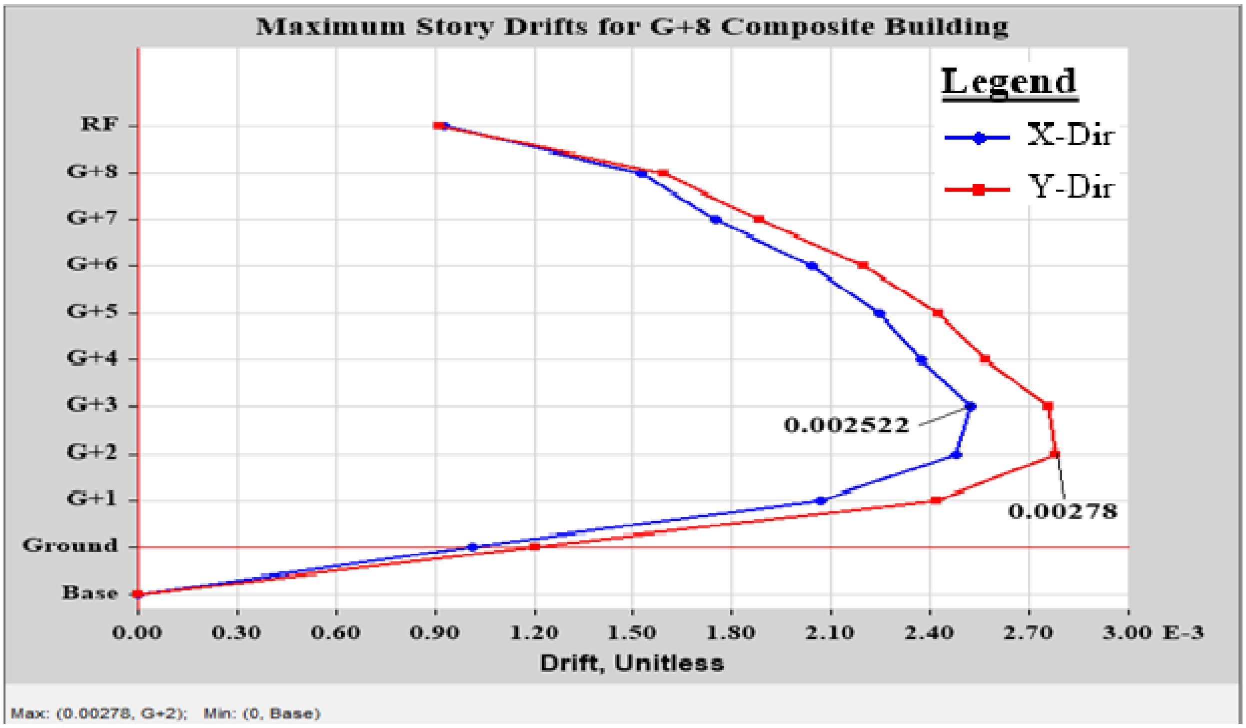


Figure 4. 8: Maximum Story Drift for G+8 Building in case of Composite Alternative

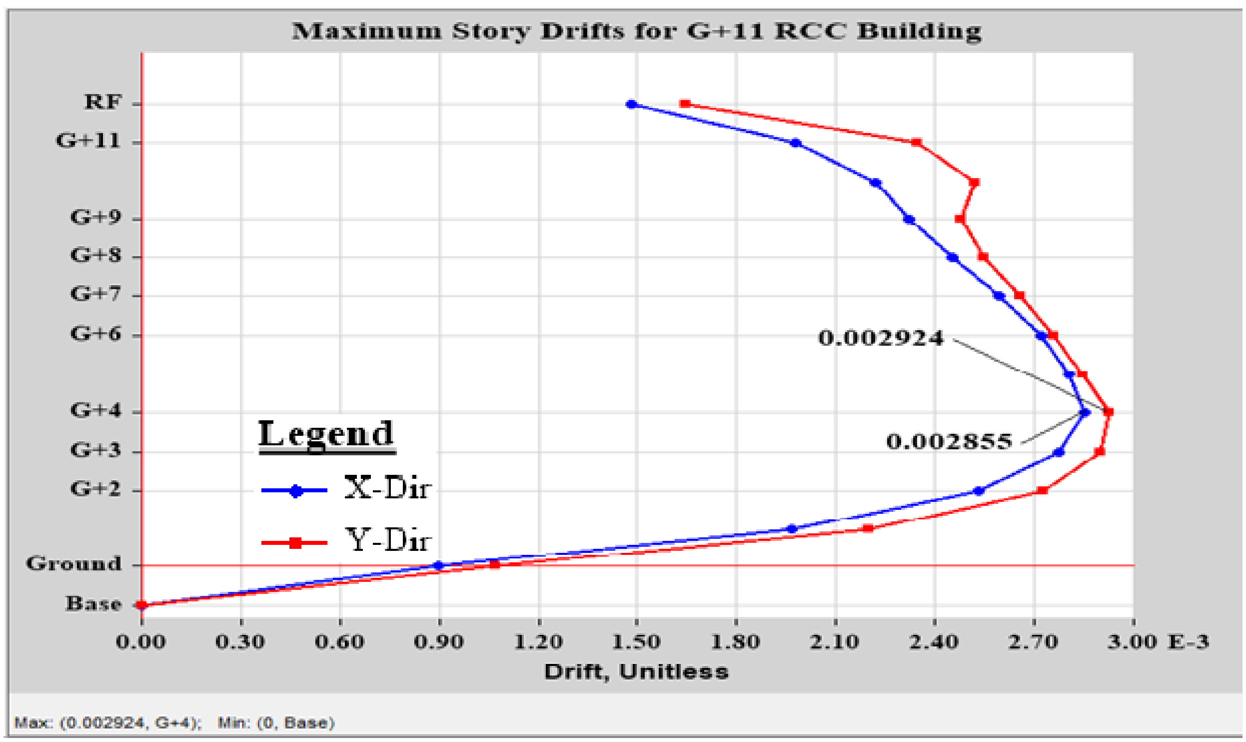


Figure 4. 9: Maximum Story Drift for G+11 Building in case of RCC Alternative

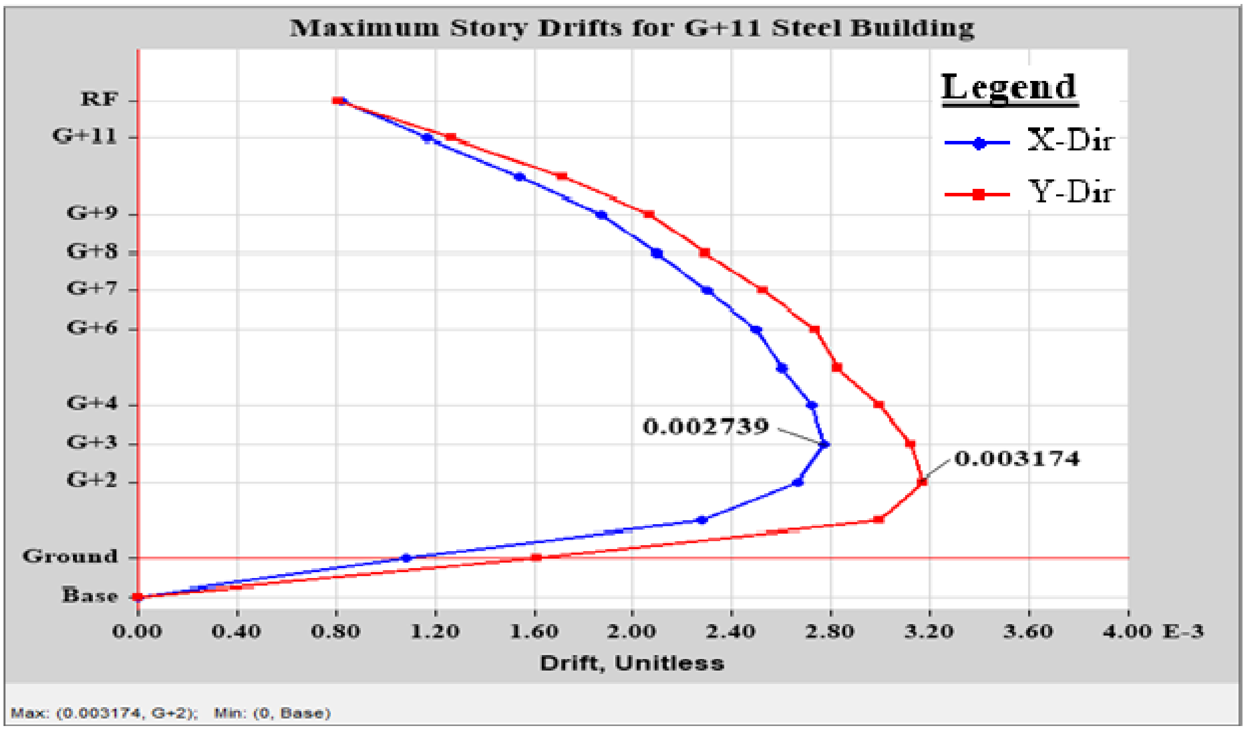


Figure 4. 10: Maximum Story Drift for G+11 Building in case of Steel Alternative

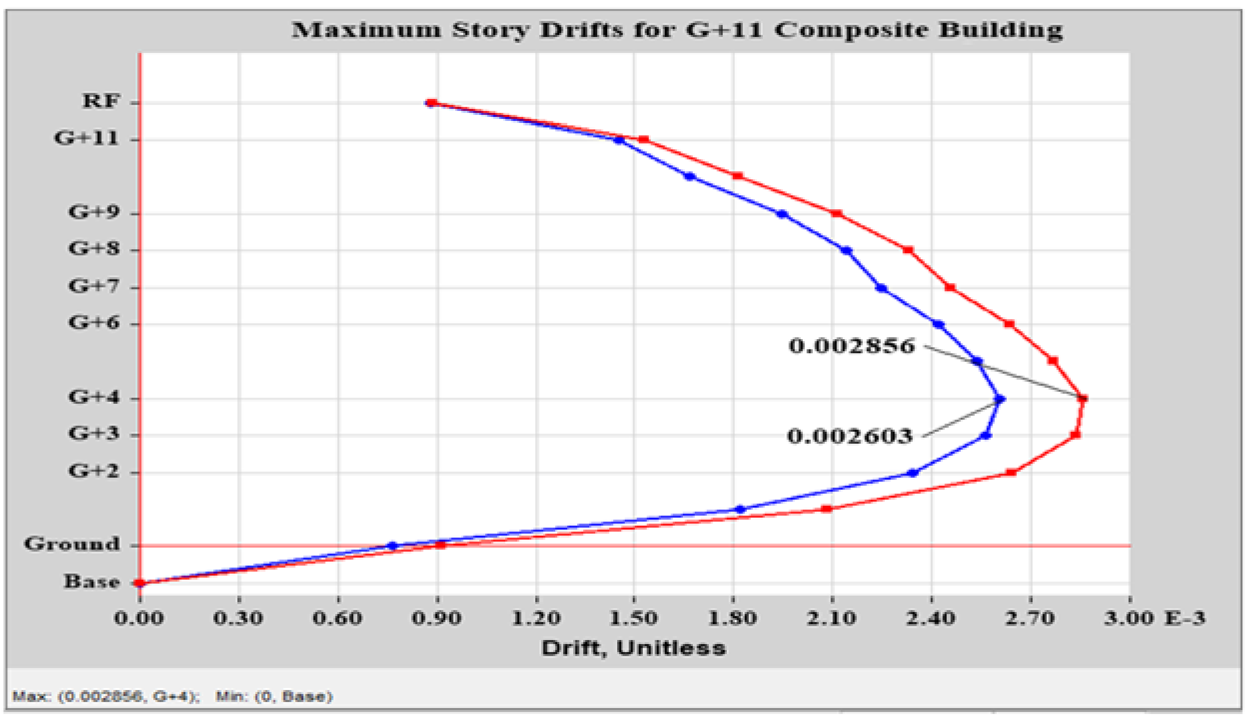


Figure 4. 11: Maximum Story Drift for G+11 Building in case of Composite Alternative

From the above figure 4.3-4.11 the results of storey drift show that; In case of G+5 buildings, the increment of maximum story drift in reinforced concrete building is 9.89% and 4.99% in X and Y direction, respectively and in steel building, the increment of maximum story drift is 0.4% and 10.60% in X and Y direction, respectively when compared to composite building whereas reinforced concrete building is increased by 9.52% and decreased by 5.90% compared to steel building in X and Y direction respectively. In case of G+8 buildings, the increment of maximum story drift in reinforced concrete building is 7.96% in X and by 1.07% in Y direction, respectively and in steel building, the increment of maximum story drift is 4.25% and 10.09% in X and Y direction, respectively compared to composite building whereas reinforced concrete building is increased by 3.87% and decreased by 9.12% compared to steel building in X and Y direction respectively. In case of G+11 buildings, the increment of maximum story drift in reinforced concrete building is 8.83% in X and by 2.33% in Y direction and in steel building, the increment of maximum story drift is 4.97% and 10.02% in X direction and Y direction, respectively compared to composite building whereas reinforced concrete building is increased by 4.06% and decreased by 8.55% compared to steel building in X and Y direction respectively. Based on the above results, it is found that the composite buildings have lowest storey drift values as compared to the reinforced concrete and steel buildings due to the higher stiffness of members in a composite structure compared to the reinforced concrete and steel building. Reinforced concrete buildings have less values of storey drift in comparison with steel buildings The variability in storey drift values in X and Y directions is due to the column orientation which leads to the different moments of inertia. So stiffness is playing the lead role in storey drift factor.

4.3 Storey Displacements

The results of maximum story displacement are recorded from figure 4.12 to figure 4.20 in both directions.

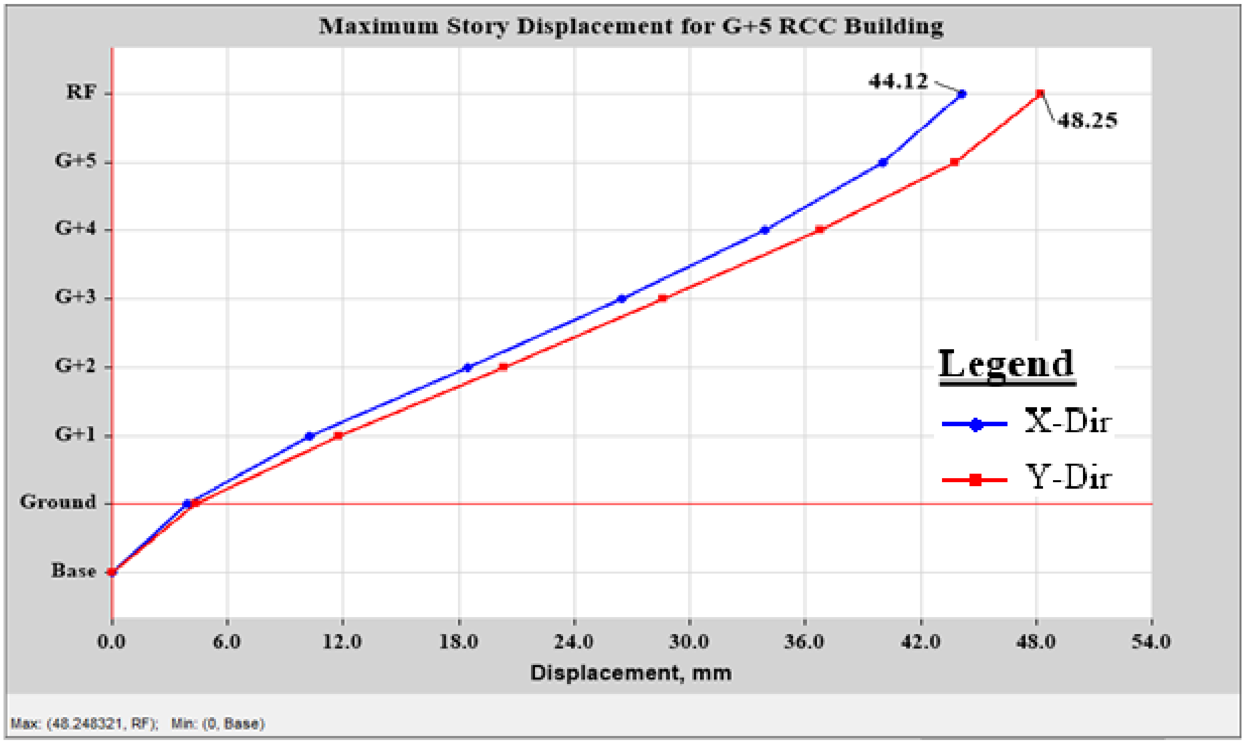


Figure 4. 12: Maximum Story Displacement for G+5 Building in case of RCC Alternative

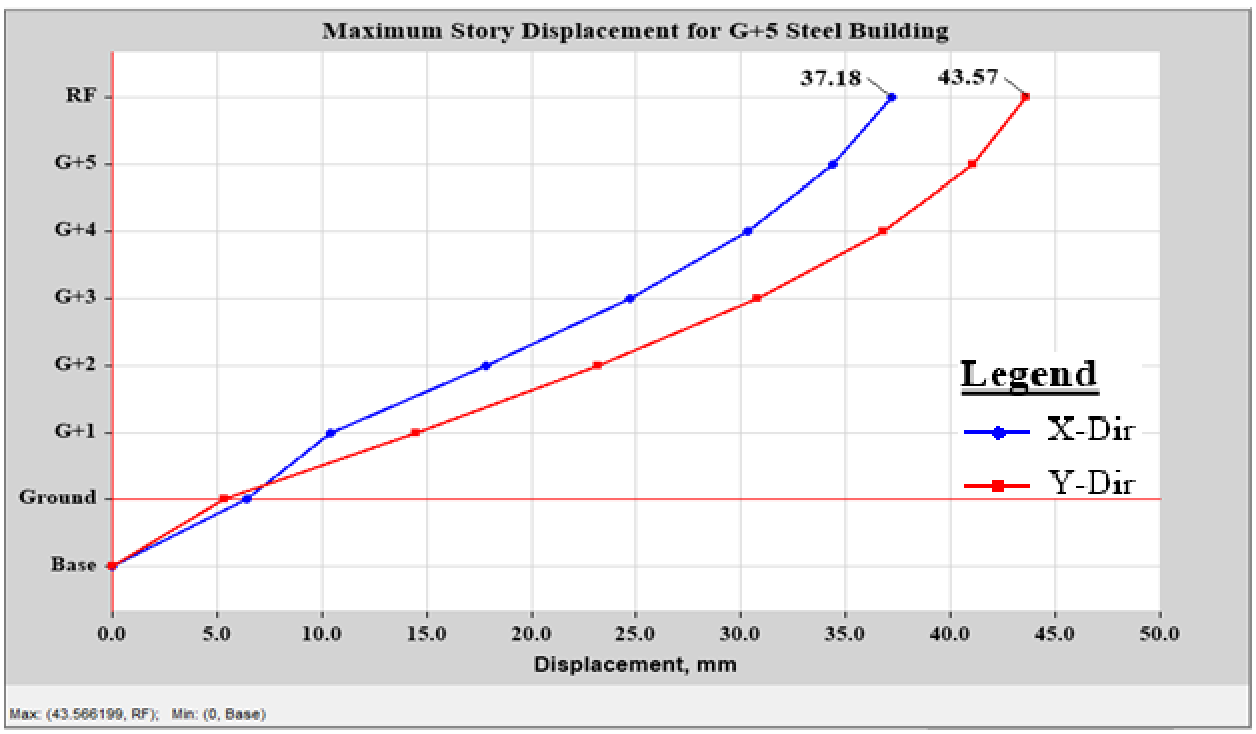


Figure 4. 13: Maximum Story Displacement for G+5 Building in case of Steel Alternative

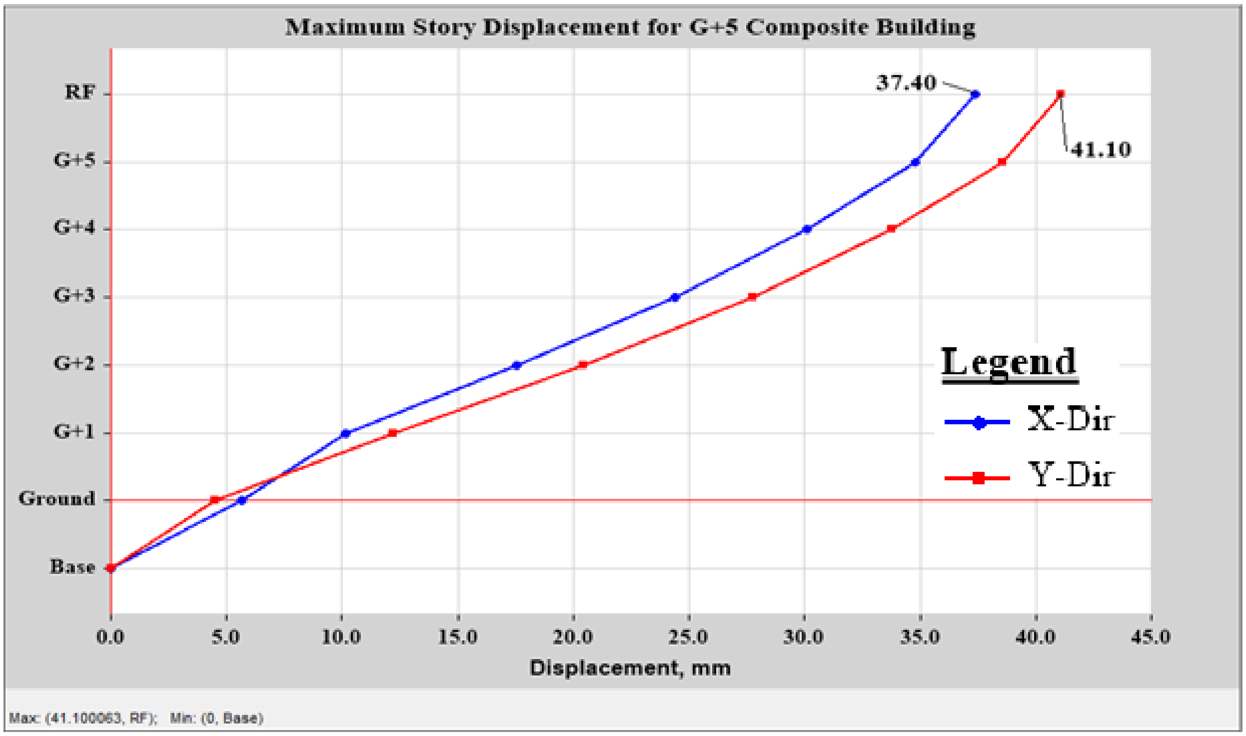


Figure 4. 14: Maximum Story Displacement for G+5 Building in Composite Alternative

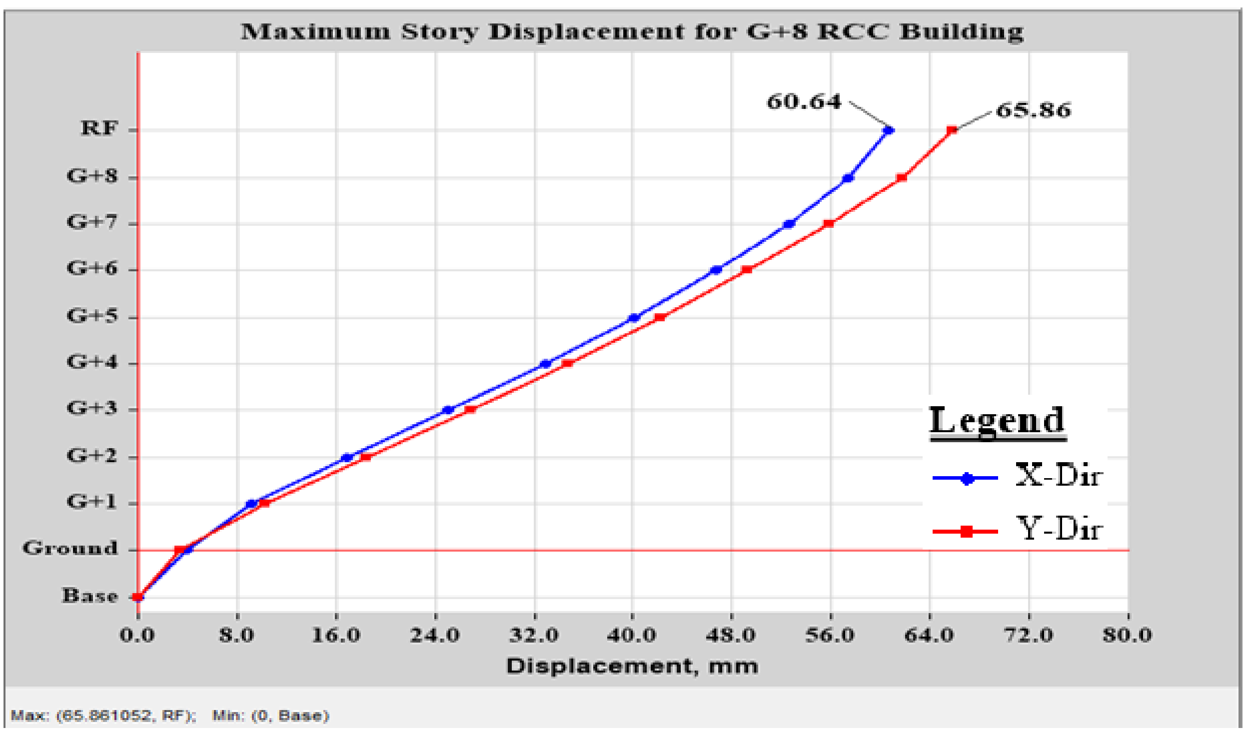


Figure 4. 15: Maximum Story Displacement for G+8 Building in case of RCC Alternative

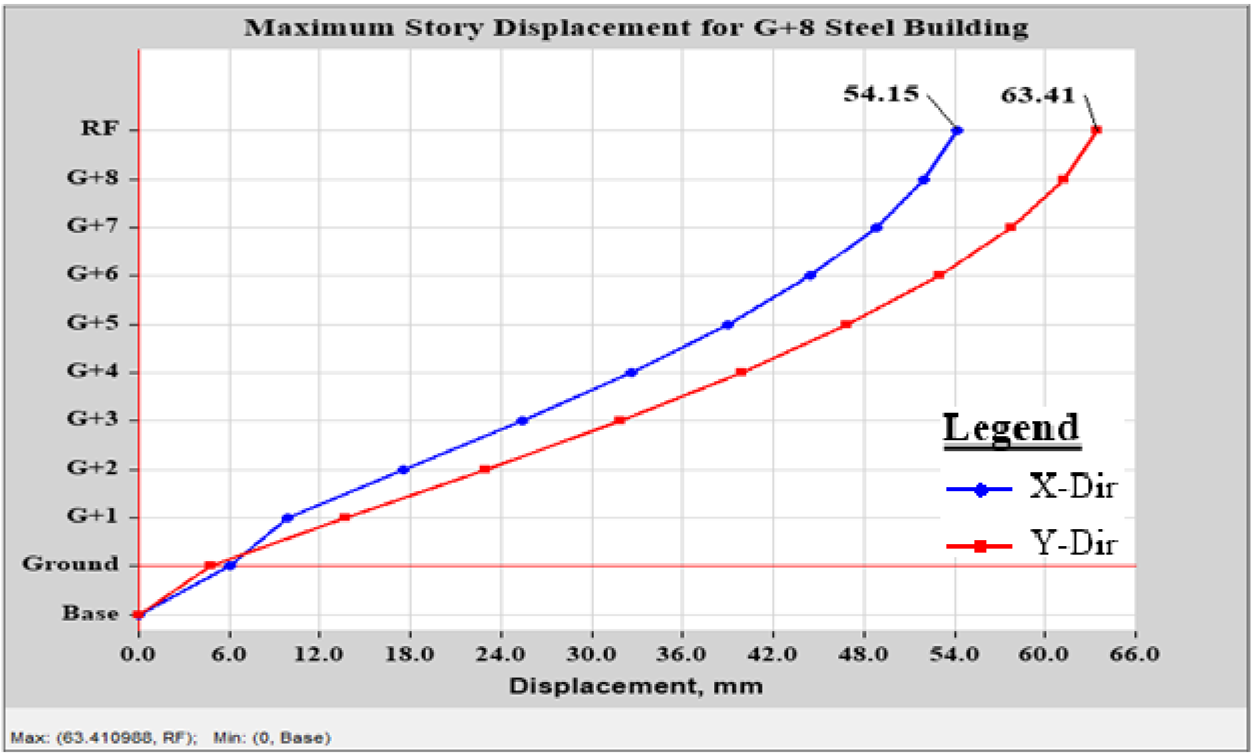


Figure 4. 16: Maximum Story Displacement for G+8 Building in case of Steel Alternative

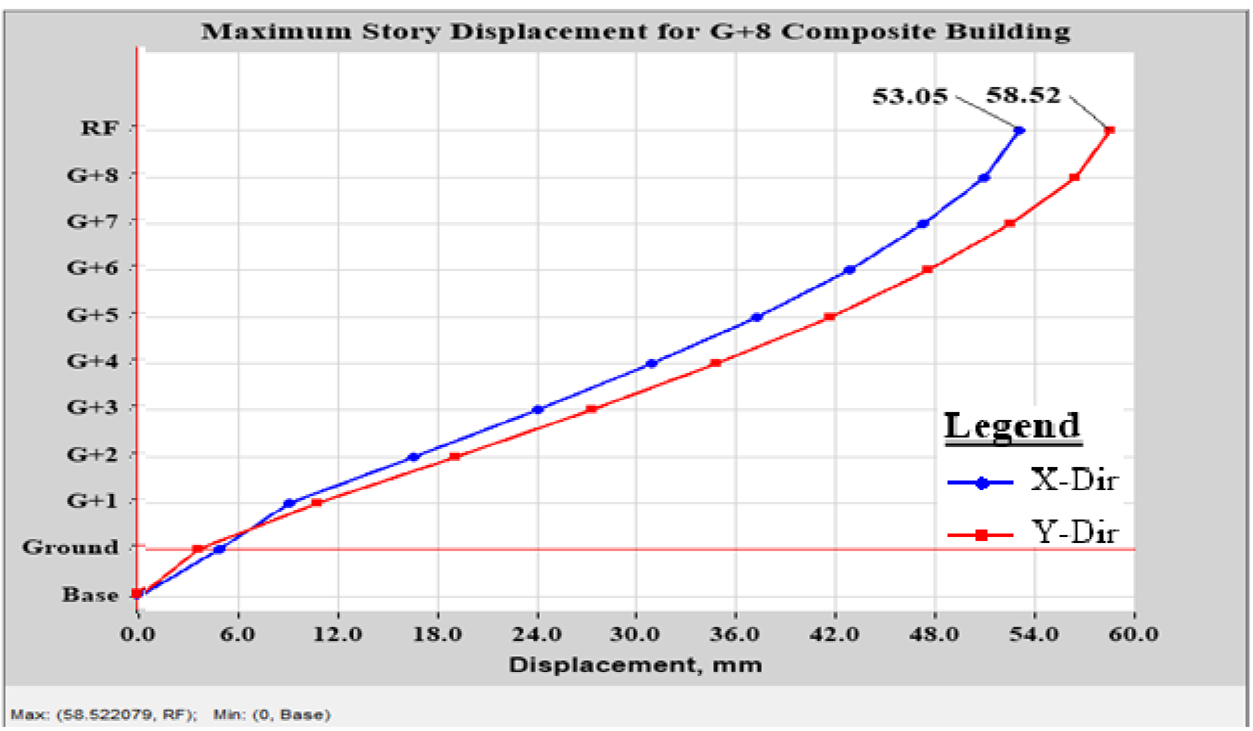


Figure 4. 17: Maximum Story Displacement for G+8 Building in Composite Alternative

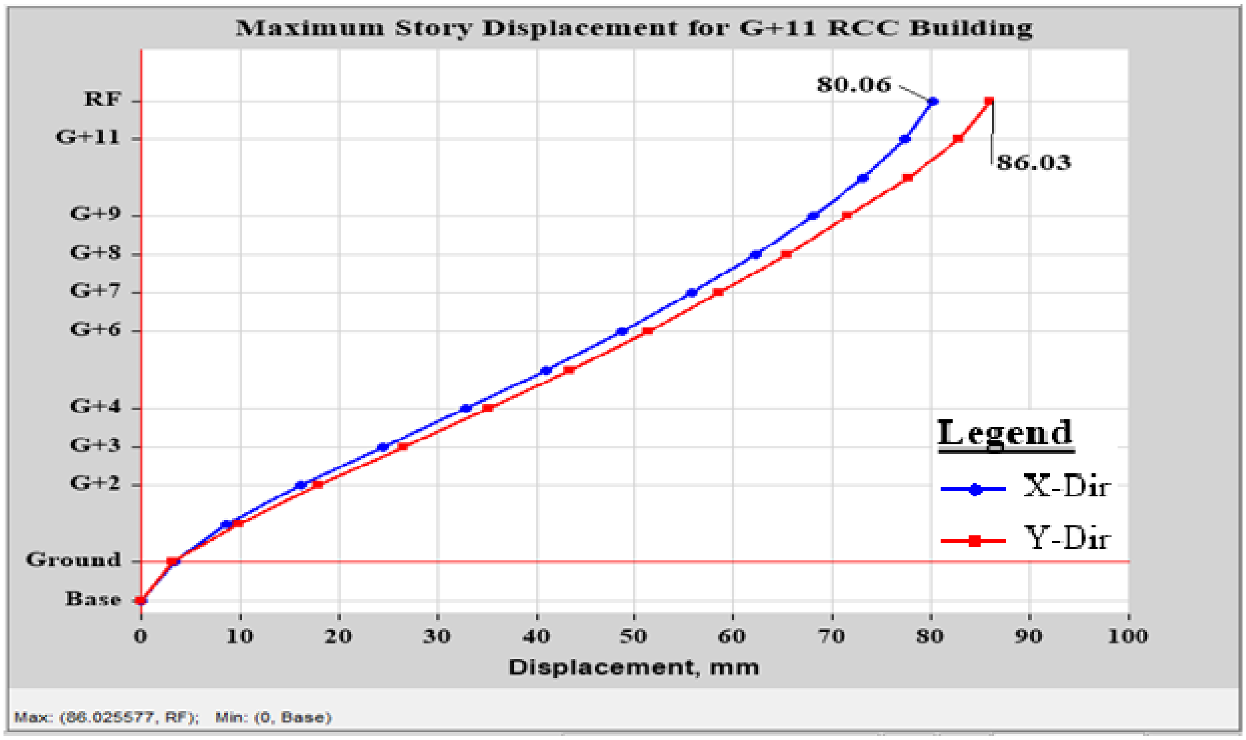


Figure 4. 18: Maximum Story Displacement for G+11 Building in case of RCC Alternative

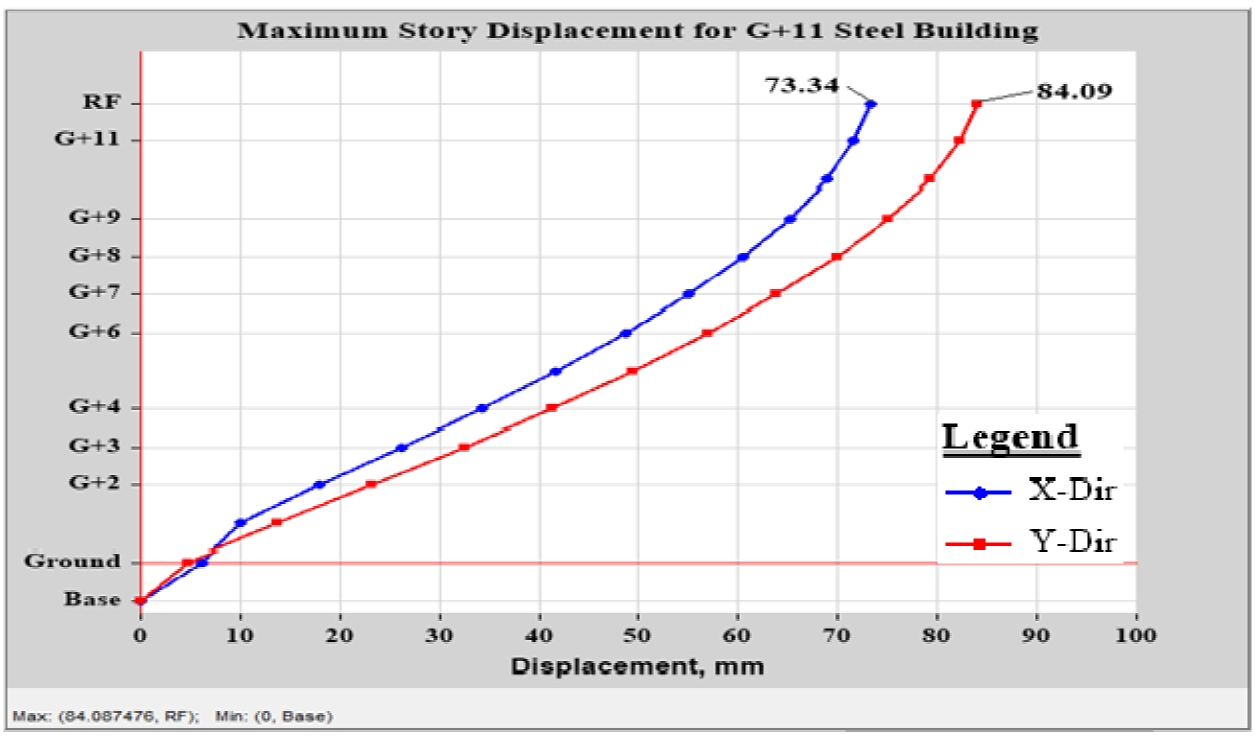


Figure 4. 19: Maximum Story Displacement for G+11 Building in case of Steel Alternative

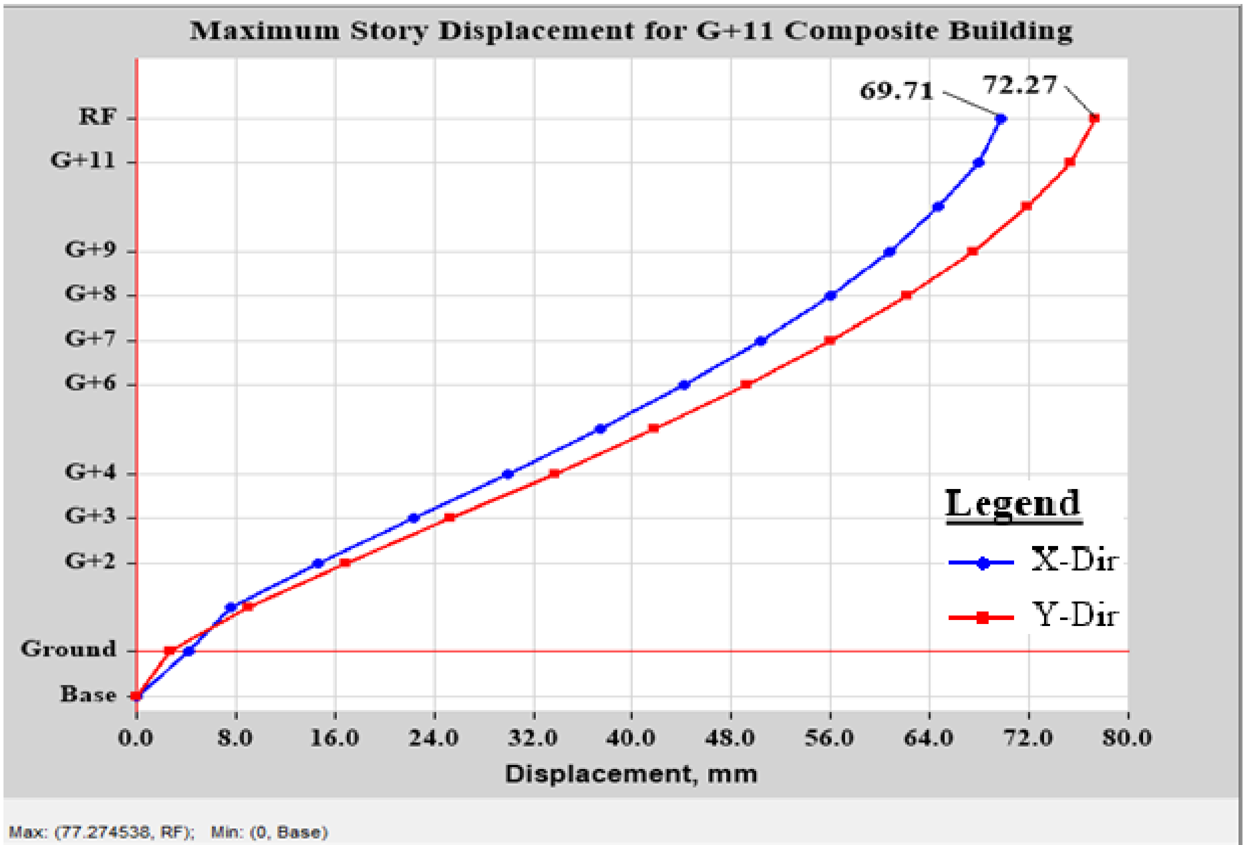


Figure 4. 20: Maximum Story Displacement for G+11 Building in Composite Alternative

From the above figure 4.12-4.20 the results of storey displacement show that; In case of G+5 buildings, the maximum story displacement is increased by 15.23% in X-direction and 14.82% in Y-direction for reinforced concrete building and decreased by 0.56% in X-direction and increased by 5.67% in Y-direction for steel building as we compare them with composite building. On the other hand, reinforced concrete building is increased by 15.73% in X-direction and 9.7% in Y-direction compared to steel building. In case of G+8 buildings, the maximum story displacements are increased by 12.52% in X-direction and 11.14% in Y-direction for reinforced concrete building and 2.03% in X-direction and 7.71% in Y-direction for steel building as we compare them with composite building. On the other hand, reinforced concrete building is increased by 10.70% in X and 3.72% in Y direction compared to steel building. In case of G+11 buildings, the maximum story displacements are increased by 12.93% in X-direction and 15.99% in Y-direction for reinforced concrete building and 4.95% in X-direction and 14.06% in Y-direction for steel building as we compare them with composite building. On the other hand, reinforced concrete building is increased by 7.89% in X and 2.26% in Y direction compared to steel building. Based on the above results, it is found that the composite buildings have lowest storey displacement values as compared to the reinforced concrete and steel buildings due to the higher stiffness of members in a composite buildings compared to the reinforced concrete and steel building. Reinforced concrete buildings have less values of storey displacement in comparison with steel buildings. The variability in storey displacement values in X and Y directions is due to the column orientation which leads to the different moments of inertia. So the same as story drift, stiffness is playing the lead role in storey displacement factor.

4.4 Time Period

The time required to complete one complete cycle of vibration is called time period. Under free vibration the structure always vibrates in single mode called its fundamental mode and the corresponding time period is fundamental period of the structure. The fundamental period is the longest period of the structure. The building natural time period is obtained as: $T = 2\pi * \sqrt{(m/k)}$ Where, m is mass of the structure and k is stiffness of the building. Time period depends upon the mass and stiffness of the structure. The higher time period the heavier the modal mass and the less stiff the structure is and vice-versa. By performing the modal response spectrum analysis, time period is found out for a corresponding mode shapes and their fundamental time period taken from ETABS software as shown in Appendix-D. The time period for each structural alternative are recorded from figure 4.21 to figure 4.23.

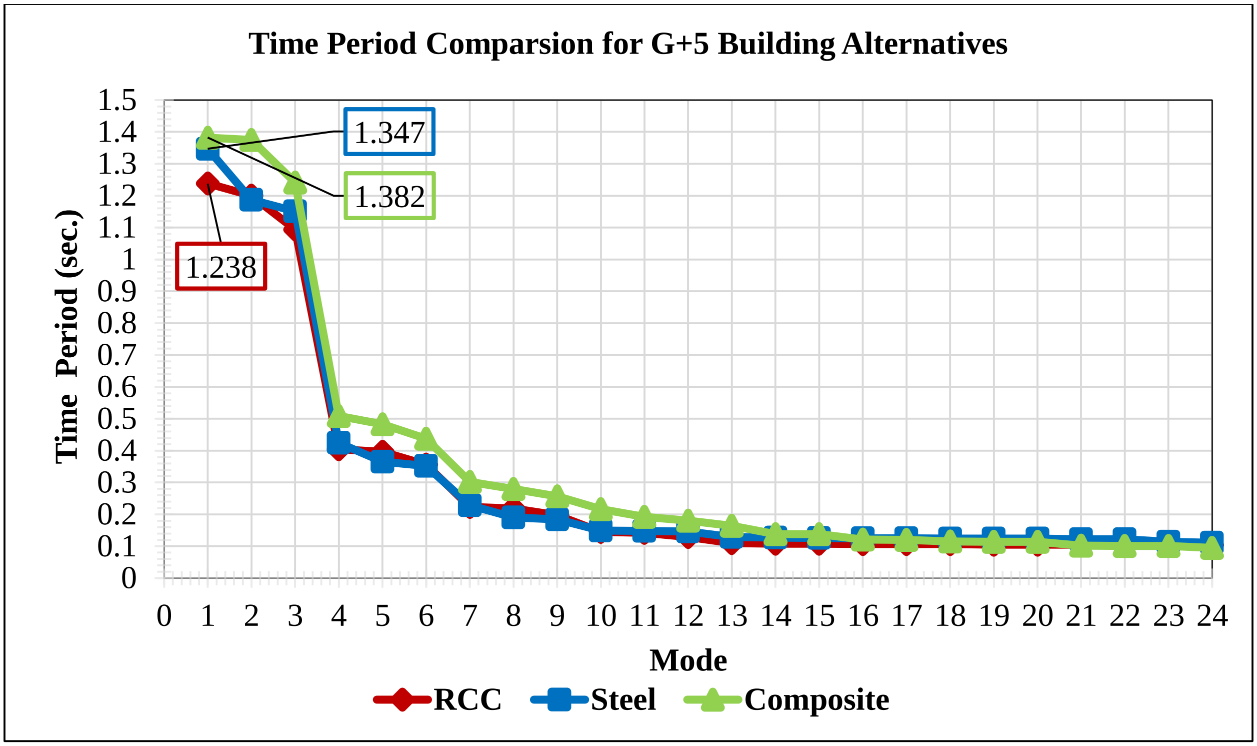


Figure 4. 21: Time Period Comparison in Case of G+5 Buildings Alternative

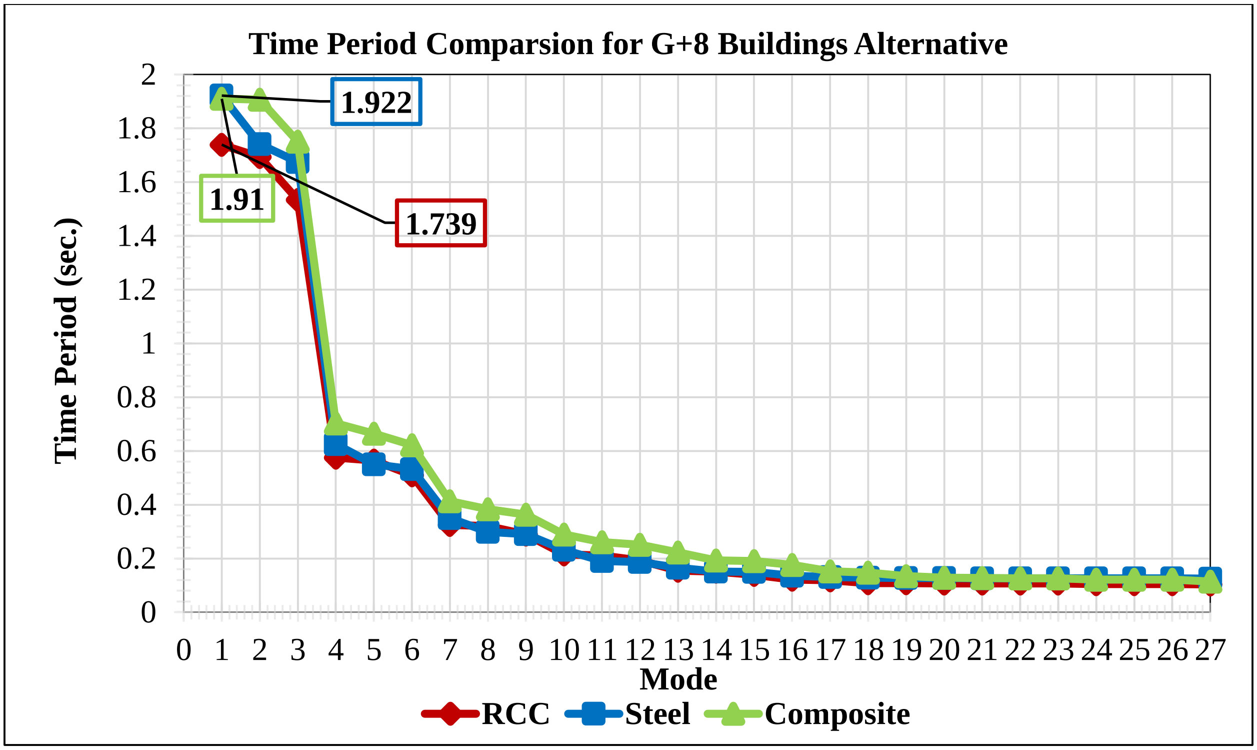


Figure 4. 22: Time Period Comparison in Case of G+8 Buildings Alternative

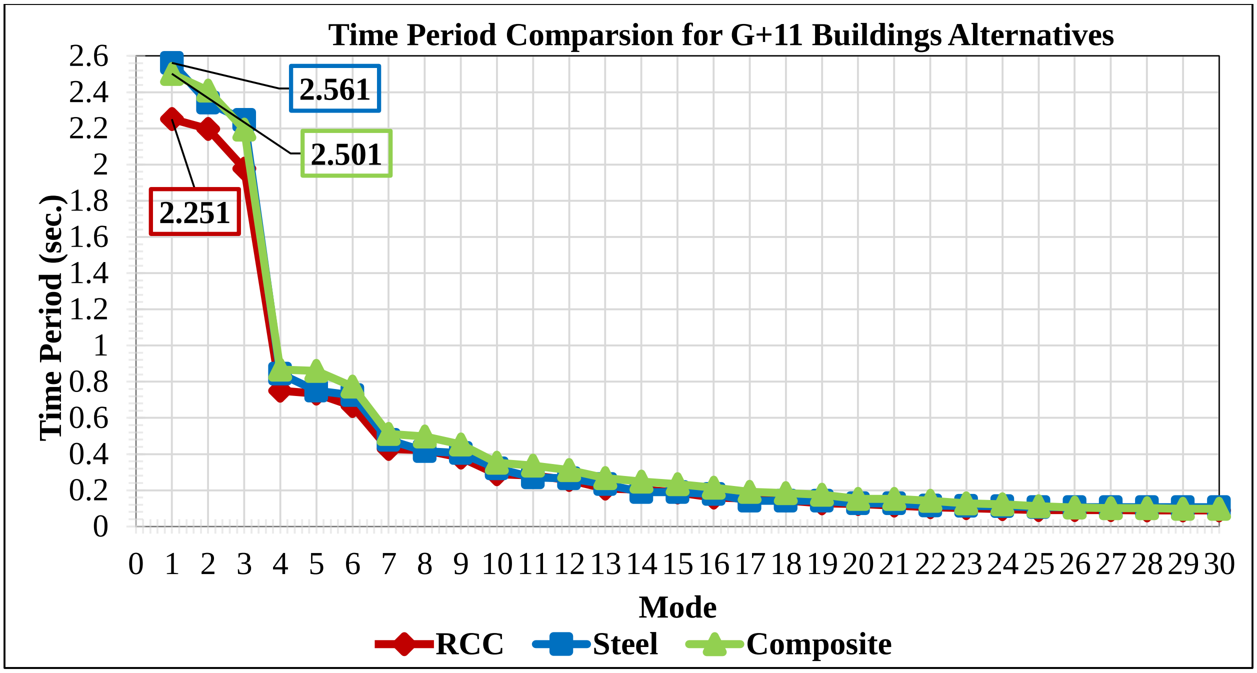
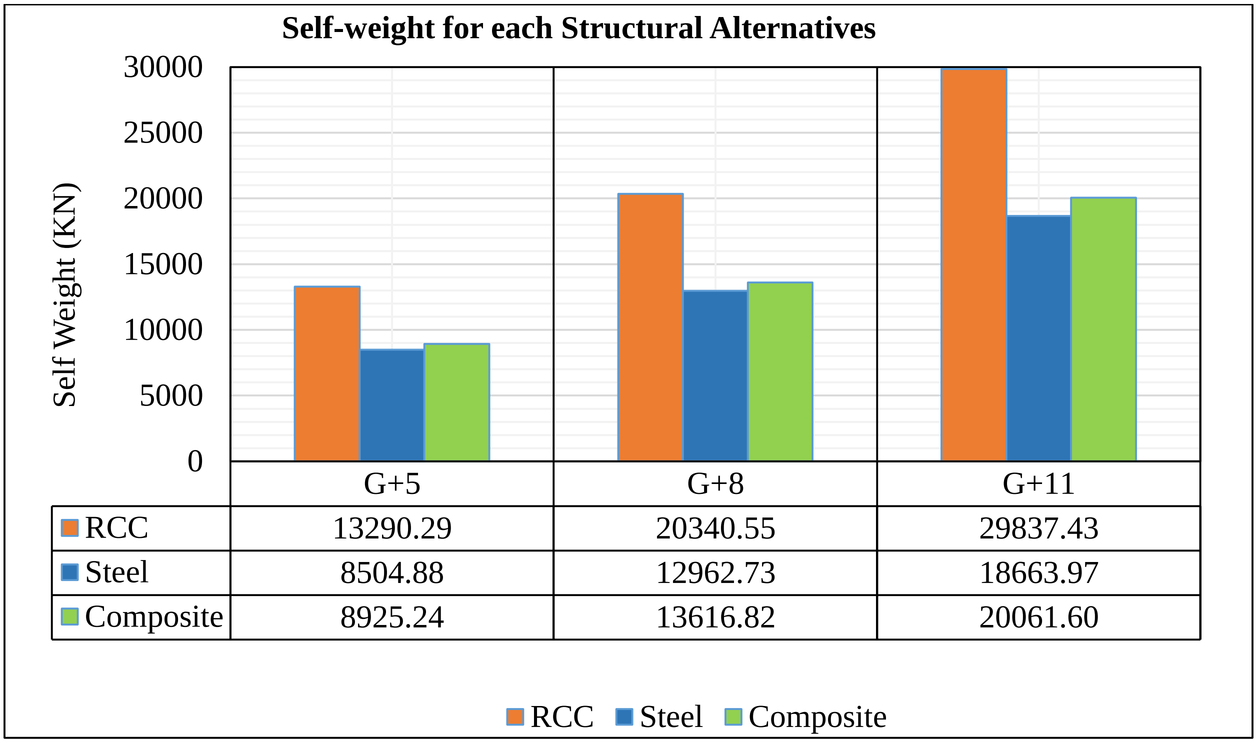


Figure 4. 23: Time Period Comparison in Case of G+11 Buildings Alternative

From the above figure 4.21-4.23 the results of time period show that; In case of G+5 buildings, the time period of reinforced concrete building is decreased by 10.42% and 8.09% compared to composite and steel building respectively whereas time period of steel building is increased by 2.53% compared to composite building. In case of G+8 buildings, the time period of reinforced concrete building is decreased by 8.95% and 9.52% compared to composite and steel building respectively whereas time period of composite building is decreased by 0.62% compared to steel building. In case of G+11 buildings, the time period of reinforced concrete building is decreased by 9.99% and 12.10% compared to composite and steel building respectively whereas time period of composite building is decreased by 2.34% compared to steel building. Based on the above results, it is found that the structural steel building has maximum time period, it means it is more flexible to oscillate back and forth when lateral force act on the building compared to reinforced concrete and composite building. Reinforced concrete has least time period which says that it is less flexible compared to structural steel and composite building.

4.5 Self-weight

Self-weight of structure depends entirely upon the type of material used in constructing one. As in case of reinforced concrete frame the weight of building is more compared to steel and composite frame. So criteria for material selection can be selected by keeping in mind the results of this study. The self-weight of the skeletal building is recorded in figure 4.24 for each structural alternative.



	G+5	G+8	G+11
RCC	13290.29	20340.55	29837.43
Steel	8504.88	12962.73	18663.97
Composite	8925.24	13616.82	20061.60

Figure 4. 24: Self-weight Comparison of each Structural Alternatives

Based on the above figure 4.24 the results of the self-weight show that; In case of G+5 buildings, the self-weight of steel building is lighter by 36.01% than reinforced concrete building and by 4.71% that of composite building and composite building is 32.84% lighter than reinforced concrete building. In case of G+8 buildings, the self-weight of steel building is lighter by 36.27% than reinforced concrete building and by 4.80% that of composite building and composite building is 33.06% lighter than reinforced concrete building. In case of G+11 buildings, the self-weight of steel building is lighter by 37.45% than reinforced concrete building and by 6.97% that of composite building and composite building is 32.76% lighter than reinforced concrete building. The results show that the dead weight of a steel building is less compared to reinforced concrete composite building. The dead weight of the composite building is less compared to reinforced concrete building it is subjected to less amount of forces induced due to the earthquake.

4.6 Cost Comparison

Cost is one parameter as a comparison factor of this thesis. In this study material cost, labor cost and equipment cost are considered in a cost comparison. Construction costs such as transport cost, finishing cost, electrician and sanitary cost are not considered in the cost comparison, since the rate unit analysis in table 4.2 is not including these construction costs. The quantity of material usage for each structural alternative discussed briefly in appendix B. Here, the comparison of final material usage and final material cost of each alternative is discussed using a tabular form and graphical representation.

Table 4. 1: Total Material Usage for each Alternative

Alternative	Total Concrete (m^3)	Total Reinforcing Bar (kg)	Total Structural Steel (kg)	Total Formwork (m^2)
G+5 Buildings (seven story)				
RCC Alternative	732.40	122287.98	-	7013.28
Steel Alternative	265.06	-	234133.60	-
Composite Alternative	312.30	-	168112.34	-
G+8 Buildings (ten story)				
RCC Alternative	1104.69	218095.60	-	12856.00
Steel Alternative	408.34	-	349494.42	-
Composite Alternative	490.43	-	252731.52	-
G+11 Buildings (thirteen story)				
RCC Alternative	1512.50	285452.35	-	17081.14
Steel Alternative	542.07	-	468833.96	-
Composite Alternative	672.44	-	347357.39	-

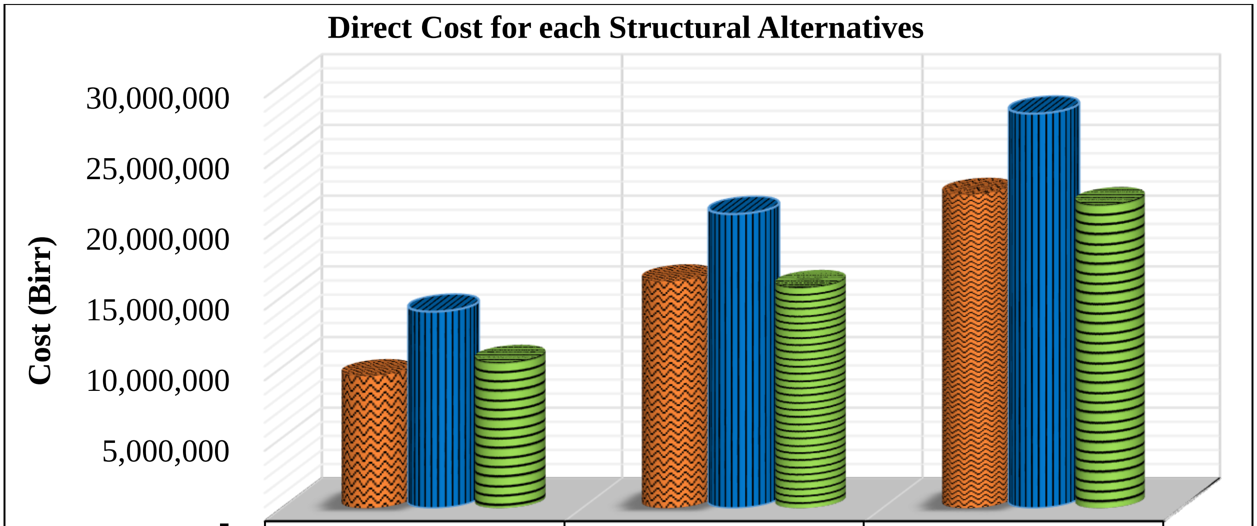
Based on the above table 4.1; In case of G+5 buildings, the reinforced concrete alternative becomes evident which consumes highest material usage. Reinforced concrete alternative comes with the higher concrete consumption of 732.40 m^3 and reinforcing bar usage more than 134.85 tons and with additional material usage of a formwork consumption 7013.28 m^2 compared to steel and composite alternative. The structural steel alternative comes with the lower concrete consumption of 265.06 m^3 and higher structural steel consumption 258.18 tons compared to reinforced concrete and composite alternatives, respectively. Since, composite alternative presents the lowest amount of concrete 312.30 and structural steel usage 185.38 tons compared to reinforced concrete and structural steel alternative, respectively. In case of G+8 buildings; the reinforced concrete alternative becomes evident which consumes highest material usage. Reinforced concrete alternative comes with the higher concrete consumption of 1,104.69 m^3 and reinforcing bar usage more than 240.49 tons and with additional material usage of a formwork consumption of 12856 m^2 compared to steel and composite alternative. The structural steel alternative comes with the lower concrete consumption of 408.34 m^3 and higher structural steel consumption 385.38 tons compared to reinforced concrete and composite alternatives, respectively. Since, steel-concrete composite alternative presents the lowest amount of concrete 490.43 and structural steel usage 278.68 tons compared to reinforced concrete and structural steel alternative, respectively. In case of G+11 buildings; the reinforced concrete alternative becomes evident which consumes highest material usage. Reinforced concrete alternative takes higher concrete consumption of 1,512.50 m^3 and reinforcing bar usage more than 314.76 tons and with additional material usage of a formwork consumption of 17081.14 m^2 compared to steel and composite alternative. The structural steel alternative comes with the lower concrete consumption of 542.07 m^3 and higher structural steel consumption 516.98 tons compared to reinforced concrete and composite alternatives, respectively. Since, steel-concrete composite alternative presents the lowest amount of concrete 672.44 m^3 and structural steel usage 383.03 tons compared to reinforced concrete and structural steel alternative, respectively.

Due to this reason, steel-concrete composite alternative is the main saving in material consumption compared to other two structural alternatives.

Table 4. 2: Unit Price of Materials (Addis Ababa City Construction Bureau)

No	Material	Unit	Unit Price (Birr)	Unit Price ($)
1	Concrete Grade (C-25)	m^3	3100	103.3
2	Concrete Grade (C-30)	m^3	3260	108.7
2	Reinforcing Bar	Kg	54	1.8
3	Formwork	m^2	320	10.7
4	Structural Steel	Kg	56	1.9

The items shown in table 4.2 contain the necessary current material price to build the final product with the corresponding material cost. Based on the item list it is possible to determine the final material cost of the structures for each building and their different structural variations. To simplify the data analysis, the final material cost for each structural alternatives is summarized in table 4.3.



	G+5	G+8	G+11
RCC	9,369,887	16,072,670	22,189,941
Steel	13,933,160	20,837,533	27,935,118
Composite	10,382,416	15,673,308	21,536,573

RCC **Steel** **Composite**

Figure 4. 25: Final Cost Comparison for each Structural Alternatives

Based on the above charts the results show that; In case of G+5 buildings, reinforced concrete alternative is cheaper by 9.76% and 32.75% compared to steel-concrete composite and structural steel alternative, respectively. In other hand, steel-concrete composite alternative is cheaper than by 25.48% compared to structural steel alternative. In case of G+7 buildings, steel-concrete composite alternative is cheaper by 2.48% and 24.78% compared to reinforced concrete and structural steel alternative, respectively. In other hand, reinforced concrete alternative is cheaper than by 22.87% compared to structural steel alternative. In case of G+11 buildings, steel-concrete composite alternative is cheaper by 2.94% and 22.91% compared to reinforced concrete and structural steel alternative, respectively. In other hand, reinforced concrete alternative is cheaper than by 20.57% compared to structural steel alternative. The result shows that the composite building is economical as compared to reinforced concrete and steel building in case of G+8 and G+11 buildings. For medium to high rise buildings steel-concrete composite buildings it is a better choice over reinforced concrete and steel building from economy point of view.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

From the comparative study of reinforced concrete, steel and composite buildings for G+5, G+8 and G+11 story for the same plan; major findings and conclusions are:

The composite building showed a reduction in base shear by 10.53% and 9.27% in X and by 12.52% and 1.51% in Y-direction for G+5; by 13.49% and 6.32% in X and by 14.89% and 0.80% in Y for G+8 and by 12.55% and 2.78% in X and by 17.48% and -4.96% in Y for G+11 compared to reinforced concrete and steel building respectively.

The composite building showed a reduction in story drift by 9.89% and 0.40% in X and by 4.99% and 10.60% in Y-direction for G+5; by 7.96% and 4.25% in X and by 1.07% and 10.09% in Y for G+8 and by 8.83% and 4.97% in X and by 2.33% and 10.02% in Y for G+11 compared to reinforced concrete and steel building respectively.

The composite building showed a reduction in story displacement by 15.23% and 0.56% in X and by 14.82% and 5.67% in Y-direction for G+5; by 12.52% and 2.03% in X and by 11.14% and 7.71% in Y for G+8 and by 12.93% and 4.95% in X and by 15.99% and 14.06% in Y for G+11 compared to reinforced concrete and steel building respectively.

The reinforced concrete building showed a reduction in time period by 10.42% and 8.09% for G+5; increased by 8.95% and reduced by 9.52% for G+8; increased by 9.99% and reduced by 12.10% for G+11 as compared to the composite and structural steel building respectively.

The steel building showed a lighter in self-weight by 36.01% and 4.71% for G+5; by 36.27% and 4.80% for G+8; by 37.45% and 6.97% for G+11 as compared to the reinforced concrete and composite building, respectively.

The steel-concrete composite building showed an increased in direct cost analysis by 9.76% and cheaper by 25.48% for G+5; cheaper by 2.48% and 24.78% G+8; cheaper by 2.94% and 22.91% for G+11 as compared to the reinforced concrete and steel building, respectively whereas reinforced concrete building is cheaper by 32.75%, 22.87% and 20.57% compared to steel building at seven, ten and thirteen story respectively.

5.2 Recommendations

This study has a wide scope in comparative study of reinforced concrete, structural steel and steel-concrete composite buildings. Among the possibilities for future study, the following will the main points that deserve attention.

- The comparative study of reinforced concrete, structural steel and steel-concrete composite buildings is done for the same span length, interstorey height and loading system (office purpose). A study using at various span length, interstorey height and loading system is left for future researcher.
- Among the comparative study, the modeling and analysis of reinforced concrete building system was carried out using a solid slab floor system. A study using flat slab or ribbed slab is left for future investigation.
- Among the comparative study, the modeling and analysis of steel-concrete composite building system was carried out using a steel concrete filled tubes column sections. A study using composite sections such as fully encased steel-concrete composite sections is left for future investigation.
- The comparative study was carried out for self-weight of the super structure. A study using a total self-weight including self-weight of foundation is left for future investigation.
- The comparative study was carried out under seismic load, leaving a comparison under wind load for future researchers.
- Analytical and Experimental investigation in design of connections for reinforced concrete, structural steel and steel-concrete composite buildings.
- The comparative study can be done, keeping in view all the factors by adding construction costs such as foundation cost, structural connection cost, transport cost, finishing cost, electrician cost, sanitary cost and others.

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Appendix A

A.1 Load Combination

Table A. 1: Load Combinations

Comb	Description
1	DL + LL
2	1.35 DL + 1.5LL
3	GS +RSXT+0.3RSYL
4	GS +RSXT+0.3RSYL
5	GS +RSXT+0.3RSYR
6	GS +RSXT+0.3RSYR
7	GS -RSXT+0.3RSYL
8	GS -RSXT+0.3RSYL
9	GS -RSXT+0.3RSYR
10	GS -RSXT+0.3RSYR
11	GS +RSXB+0.3RSYL
12	GS +RSXB+0.3RSYL
13	GS +RSXB+0.3RSYR
14	GS +RSXB+0.3RSYR
15	GS -RSXB+0.3RSYL
16	GS -RSXB+0.3RSYL
17	GS -RSXB+0.3RSYR
18	GS -RSXB+0.3RSYR
19	GS +RSYL+0.3RSXT
20	GS +RSYL+0.3RSXT
21	GS +RSYL+0.3RSXB
22	GS +RSYL+0.3RSXB
23	GS -RSYL+0.3RSXT
24	GS -RSYL+0.3RSXT
25	GS -RSYL+0.3RSXB
26	GS -RSYL+0.3RSXB
27	GS +RSYR+0.3RSXT
28	GS +RSYR+0.3RSXT
29	GS +RSYR+0.3RSXB
30	GS +RSYR+0.3RSXB
31	GS -RSYR+0.3RSXT
32	GS -RSYR+0.3RSXT
33	GS -RSYR+0.3RSXB
34	GS -RSYR+0.3RSXB

GS – is gravity load for seismic action

Appendix B

B.1 Material Usage

B.1.1 Reinforced Concrete Alternative

B.1.1.1 Quantity of Concrete

Table B. 1: Quantity of Concrete for Columns in case of G+11 Buildings

Section	Type	No.	Width (m)	Depth (m)	Total Length (m)	Volume (m^3)
R-Col 250X400	Column	24	0.2	0.4	72	5.76
R-Col 250X450	Column	18	0.25	0.5	54	6.075
R-Col 300X500	Column	24	0.3	0.5	72	10.8
R-Col 350X550	Column	24	0.35	0.6	72	13.86
R-Col 400X600	Column	18	0.4	0.6	54	12.96
R-Col 450X650	Column	30	0.45	0.7	90	26.33
R-Col 500X700	Column	34	0.5	0.7	100	35
R-Col 550X750	Column	6	0.55	0.8	18	7.425
R-Col 600X800	Column	20	0.6	0.8	60	28.8
R-Col 650X850	Column	26	0.65	0.9	73	40.33
R-Col 700X900	Column	6	0.7	0.9	18	11.34
R-Col 750X950	Column	12	0.75	1	36	25.65
R-Col 800X1000	Column	18	0.8	1	51	40.8
Total Volume of Concrete						**265.1**

Table B. 2: Quantity of Concrete for Columns in case of G+8 Buildings

Section	Type	No.	Width (m)	Depth (m)	Total Length (m)	Volume (m^3)
R-Col 250X400	Column	24	0.2	0.4	72	5.76
R-Col 250X450	Column	18	0.25	0.5	54	6.075
R-Col 300X500	Column	24	0.3	0.5	72	10.8
R-Col 350X550	Column	24	0.35	0.6	72	13.86
R-Col 400X600	Column	18	0.4	0.6	52	12.48
R-Col 450X650	Column	26	0.45	0.7	78	22.82
R-Col 500X700	Column	26	0.5	0.7	78	27.3
R-Col 550X750	Column	6	0.55	0.8	18	7.425
R-Col 600X800	Column	10	0.6	0.8	25	12
R-Col 650X850	Column	6	0.65	0.9	18	9.945
R-Col 700X900	Column	6	0.7	0.9	18	11.34
R-Col 750X950	Column	12	0.75	1	33	23.51
Total Volume of Concrete						**163.3**

Table B. 3: Quantity of Concrete for Columns in case of G+5 Buildings

Section	Type	No.	Width (m)	Depth (m)	Total Length (m)	Volume (m^3)
R-Col 250X400	Column	24	0.25	0.4	72	7.2
R-Col 250X450	Column	18	0.25	0.45	54	6.08
R-Col 300X500	Column	24	0.3	0.5	72	10.8
R-Col 350X550	Column	20	0.35	0.55	58	11.2
R-Col 400X600	Column	10	0.4	0.6	30	7.2
R-Col 450X650	Column	26	0.45	0.65	73	21.4
R-Col 500X700	Column	6	0.5	0.7	18	6.3
R-Col 550X750	Column	6	0.55	0.75	18	7.43
R-Col 650X850	Column	6	0.65	0.8	15	7.8
Total Volume of Concrete						**85.32**

Table B. 4: Quantity of Concrete for Beams in case of G+11 Buildings

Section	Type	No.	Width (m)	Depth (m)	Length (m)	Volume (m^3)
250X400	Beam	31	0.25	0.4	171	17.10
250X500	Beam	341	0.25	0.5	1881	235.13
300X500	Beam	31	0.3	0.5	171	25.65
Total Volume of Concrete						**277.88**

Table B. 5: Quantity of Concrete for Beams in case of G+8 Buildings

Section	Type	No.	Width (m)	Depth (m)	Length (m)	Volume (m^3)
250X400	Beam	31	0.25	0.4	171	17.10
250X500	Beam	279	0.25	0.5	1539	192.38
Total Volume of Concrete						**209.48**

Table B. 6: Quantity of Concrete for Beams in case of G+5 Buildings

Section	Type	No.	Width (m)	Depth (m)	Length (m)	Volume (m^3)
250X400	Beam	31	0.25	0.4	171	17.10

250X500	Beam	155	0.25	0.5	855	106.88
300X500	Beam	31	0.3	0.5	171	25.65
Total Volume of Concrete						**149.63**

Table B. 7: Quantity of Concrete for Slabs in case of G+11 Buildings

Section	Type	No.	C/C Width (m)	C/C Length (m)	Depth (m)	Volume (m^3)
BM-250X400	Beam	144	4.5	5.5	0.15	534.6
BM-300X500	Beam	12	4.4	5.4	0.15	42.768
Total Volume of Concrete						**577.368**

Table B. 8: Quantity of Concrete for Slabs in case of G+8 Buildings

Section	Type	No.	C/C Width (m)	C/C Length (m)	Depth (m)	Volume (m^3)
BM-250X400	Beam	120	4.5	5.5	0.15	579.15
Total Volume of Concrete						**445.5**

Table B. 9: Quantity of Concrete for Slabs in case of G+5 Buildings

Section	Type	No.	C/C Width (m)	C/C Length (m)	Depth (m)	Volume (m^3)
BM-250X400	Beam	84	4.5	5.5	0.15	311.85
Total Volume of Concrete						**311.85**

B.1.1.2 Quantity of formwork

Table B. 10: Quantity of Column Formwork in case of G+11 Buildings

Section	Type	No	Width (m)	Depth (m)	Perimeter (m)	Total Length (m)	Quantity (m^3)
R-Col 250X400	Column	24	0.2	0.4	1.3	72	93.60
R-Col 250X450	Column	18	0.3	0.5	1.5	54	81.00
R-Col 300X500	Column	24	0.3	0.5	1.7	72	122.40
R-Col 350X550	Column	24	0.4	0.6	1.9	72	136.80
R-Col 400X600	Column	18	0.4	0.6	2.1	54	113.40
R-Col 450X650	Column	30	0.5	0.7	2.3	90	207.00

R-Col 500X700	Column	34	0.5	0.7	2.5	100	250.00
R-Col 550X750	Column	6	0.6	0.8	2.7	18	48.60
R-Col 600X800	Column	20	0.6	0.8	2.9	60	174.00
R-Col 650X850	Column	26	0.7	0.9	3.1	73	226.30
R-Col 700X900	Column	6	0.7	0.9	3.3	18	59.40
R-Col 750X950	Column	12	0.8	1	3.5	36	126.00
R-Col 800X1000	Column	18	0.8	1	3.7	51	188.70
Total Area of Formwork							**1827.20**

Table B. 11: Quantity of Column Formwork in case of G+8 Buildings

Section	Type	No.	Width (m)	Depth (m)	Perimeter (m)	Total Length (m)	Quantity (m^2)
R-Col 250X400	Column	24	0.2	0.4	1.3	72	93.60
R-Col 250X450	Column	18	0.3	0.5	1.5	54	81.00
R-Col 300X500	Column	24	0.3	0.5	1.7	72	122.40
R-Col 350X550	Column	24	0.4	0.6	1.9	72	136.80
R-Col 400X600	Column	18	0.4	0.6	2.1	52	109.20
R-Col 450X650	Column	26	0.5	0.7	2.3	78	179.40
R-Col 500X700	Column	26	0.5	0.7	2.5	78	195.00
R-Col 550X750	Column	6	0.6	0.8	2.7	18	48.60
R-Col 600X800	Column	10	0.6	0.8	2.9	25	72.50
R-Col 650X850	Column	6	0.7	0.9	3.1	18	55.80
R-Col 700X900	Column	6	0.7	0.9	3.3	18	59.40
R-Col 750X950	Column	12	0.8	1	3.5	33	115.50
Total Area of Formwork							**1269.20**

Table B. 12: Quantity of Column Formwork in case of G+5 Buildings

Section	Type	No.	Width (m)	Depth (m)	Perimeter (m)	Total Length (m)	Quantity (m^3)
R-Col 250X400	Column	24	0.25	0.4	1.4	72	100.80
R-Col 250X450	Column	18	0.25	0.5	1.5	54	81.00
R-Col 300X500	Column	24	0.3	0.5	1.7	72	122.40
R-Col 350X550	Column	20	0.35	0.6	1.9	58	110.20
R-Col 400X600	Column	10	0.4	0.6	2.1	30	63.00
R-Col 450X650	Column	26	0.45	0.7	2.3	73	167.90
R-Col 500X700	Column	6	0.5	0.7	2.5	18	45.00
R-Col 550X750	Column	6	0.55	0.8	2.7	18	48.60

R-Col 650X850	Column	6	0.6	0.8	2.9	15	43.50
Total Area of Formwork							**782.40**

Table B. 13: Quantity of Beam Formwork in case of G+11 Buildings

Section	Type	No.	Width (m)	Depth (m)	Perimeter (m)	Length (m)	Quantity (m^2)
250X400	Beam	31	0.25	0.4	1.1	171	188.10
250X500	Beam	341	0.25	0.5	1.3	1881	2445.30
300X500	Beam	31	0.3	0.5	1.35	171	230.85
Total Formwork Area							**2864.25**

Table B. 14: Quantity of Beam Formwork in case of G+8 Buildings

Section	Type	No.	Width (m)	Depth (m)	Perimeter (m)	Length (m)	Quantity (m^2)
250X400	Beam	31	0.25	0.4	1.1	171	188.10
250X500	Beam	279	0.25	0.5	1.3	1539	2000.70
Total Formwork Area							**2188.80**

Table B. 15: Quantity of Slab Formwork in case of G+11 Buildings

Section (mm)	Type	No.	Width (m)	Length (m)	Quantity (m^2)
S-150	BM-250X400	144	4.5	5.5	3564
S-150	BM-300X500	12	4.4	5.4	285.12
Total Formwork Area					**3849.12**

Table B. 16: Quantity of Slab Formwork in case of G+8 Buildings

Section (mm)	Type	No.	Width (m)	Length (m)	Quantity (m^2)
S-150	BM-250X400	156	4.5	5.5	3861
Total Formwork Area					**3861**

Table B. 17: Quantity of Slab Formwork in case of G+5 Buildings

Section (mm)	Type	No.	Width (m)	Length (m)	Quantity (m^2)
S-150	BM-250X400	84	4.5	5.5	2079
Total Formwork Area					**2079**

B.1.2 Structural Steel Alternative

B.1.2.1 Quantity of Structural Steel

Table B. 18: Quantity of Structural Steel for Column in case of G+11 Buildings

Section	Element Type	# Pieces	Total Length	Total Weight (kN)	Total Weight (kg)
SC-450	Column	18	54	162.54	16568.828
SC-500	Column	18	51	175.104	17849.511
SC-400	Column	104	310	801.847	81737.676
SC-400'	Column	50	150	512.702	52263.15
SC-450'	Column	34	100	399.538	40727.584
SC-500'	Column	36	105	492.424	50196.168
Total Weight				**2544.15**	**259342.92**

Table B. 19: Quantity of Structural Steel for Column in case of G+8 Buildings

Section	Element Type	# Pieces	Total Length	Total Weight	Total Weight (kg)
SC-450	Column	18	51	153.51	15648.34
SC-400	Column	92	274	708.73	72245.57
SC-400'	Column	50	150	512.70	52263.15
SC-450'	Column	22	64	255.70	26065.66
SC-500'	Column	18	51	239.18	24380.99
Total Weight				**1869.82**	**190603.70**

Table B. 20: Quantity of Structural Steel for Column in case of G+5 Buildings

Section	Element Type	# Pieces	Total Length	Total Weight	Total Weight (kg)
SC-400	Column	80	235	607.8515	61962.437
SC-400'	Column	42	124	423.8332	43204.2

SC-450'	Column	18	51	203.7642	20771.071
Total Weight				**1235.45**	**125937.71**

Table B. 21: Quantity of Structural Steel for Beams in case of G+11 Buildings

Section	Element Type	# Pieces	Total Length (m)	Total Weight (kN)	Total Weight (kg)
SB-350	Beam	95	483	206.1	21009.205
SB-400	Beam	212	1164	491.28	50079.511
SB-450	Beam	96	576	347.67	35440.408
SB-Com-300	Beam	264	1320	499.953	50963.629
SB-Com-200	Beam	24	120	34.3649	3503.0479
Total Weight				**1579.37**	**160995.80**

Table B. 22: Quantity of Structural Steel for Beams in case of G+8 Buildings

Section	Element Type	# Pieces	Total Length (m)	Total Weight (kN)	Total Weight (kg)
SB-350	Beam	77	393	167.983	17123.649
SB-400	Beam	161	885	374.643	38189.878
SB-450	Beam	72	432	261.321	26638.257
SB-Com-300	Beam	192	960	363.60	37064.45
SB-Com-200	Beam	24	120	34.36	3503.05
Total Weight				**1201.91**	**122519.29**

Table B. 23: Quantity of Structural Steel for Beams in case of G+5 Buildings

Section	Element Type	# Pieces	Total Length	Total Weight (kN)	Total Weight (kg)
SB-350	Beam	59	303	129.682	13219.327
SB-400	Beam	110	606	257.392	26237.747
SB-450	Beam	48	288	174.842	17822.865
SB-Com-300	Beam	120	600	227.252	23165.291
SB-Com-200	Beam	24	120	34.3649	3503.0479
Total Weight				**823.53**	**83948.28**

Table B. 24: Quantity of Concrete and Steel for Floors in case of G+11 Buildings

Section	Element Type	Total Weight (kN)	Total Weight (kg)	Net Weight of Concrete (m^3)
Com-Floor	Floor	13770.00	1355174.50	542.07
Com-Floor	Metal Deck	475.74	48495.24	-
Total Weight			**1403669.74**	**542.07**

Table B. 25: Quantity of Structural Steel for Floors in case of G+8 Buildings

Section	Element Type	Total Weight (kN)	Total Weight (kg)	Net Weight of Concrete (m^3)
Com-Floor	Floor	10327.5	1016381	408.34
Com-Floor	Metal Deck	356.8037	36371.4	-
Total Weight		**10684.3037**	**1052752**	**408.34**

Table B. 26: Quantity of Structural Steel for Floors in case of G+5 Buildings

Section	Element Type	Total Weight (kN)	Total Weight (kg)	Net Weight of Concrete (m^3)
Com-Floor	Floor	6716.25	660385.42	269.93
Com-Floor	Metal Deck	237.87	24247.61	-

B.1.3 Steel-concrete Composite Alternative

B.1.3.1 Quantity of Structural Steel and Concrete

Table B. 27: Quantity of Concrete and Steel for Columns in G+11 Buildings

Section	# Pieces	L_{total} (m)	W_{total} (kN)	W_{total} (kg)	t (mm)	Width (m)	Depth (m)	Volume (m^3)
Com-Col 250X300	56	168	425.2	43348.2	12	0.226	0.276	10.5
Com-Col 300X350	38	114	388.3	39580.2	12	0.276	0.326	10.3
Com-Col 350X400	62	186	859.8	87649.7	15	0.326	0.376	22.8
ComCol400X450	48	142	820.6	83647.7	15	0.376	0.426	22.7
Com-Col 450X500	22	66	465.9	47496.5	15	0.426	0.476	13.4
Com-Col 500X550	16	43	364.0	37107.4	15	0.476	0.526	10.8
Com-Col 550x600	6	18	179.9	18342.4	15	0.526	0.576	5.5
Com-Col 600X650	12	33	388.6	39613.6	16	0.576	0.626	11.9

Total Weight	**3892**	**396786**		**107.8**

Table B. 28: Quantity of Concrete and Steel for Columns in G+8 Buildings

Section	No	L_{total} (m)	W_{total} (kN)	W_{total} (kg)	t (mm)	Width (m)	Depth (m)	Volume (m^3)
Com-Col 250X300	56	168	425.2	43348.2	12	0.226	0.276	10.5
Com-Col 300X350	38	114	388.3	39580.2	12	0.276	0.326	10.3
Com-Col 350X400	54	160	739.7	75397.6	15	0.326	0.376	19.6
ComCol400X450	34	97	560.5	57139.6	15	0.376	0.426	15.5
Com-Col 450X500	12	36	254.2	25907.2	15	0.426	0.476	7.3
Com-Col 500X550	6	15	127.0	12944.4	15	0.476	0.526	3.8
Total Weight			**2495**	**254317**				**66.9**

Table B. 29: Quantity of Concrete and Steel for Columns in G+5 Buildings

Section	No	L_{total} (m)	W_{total} (kN)	W_{total} (kg)	t (mm)	Width (m)	Depth (m)	Volume (m^3)
Com-Col 250X300	56	168	425.2	43348.2	12	0.226	0.276	10.5
Com-Col 300X350	34	102	347.4	35413.9	12	0.276	0.326	9.2
Com-Col 350X400	32	96	443.8	45238.6	15	0.326	0.376	11.8
Com-Col400X450	18	54	312.1	31809.7	15	0.376	0.426	8.6
Total Weight			**1528**	**155810.3**				**40.1**

Table B. 30: Quantity of Structural Steel for Beams in G+11 Buildings

Section	Element Type	# Pieces	Total Length	Total Weight (kN)	Total Weight (kg)
SB-350	Beam	95	483	207.1	21110.1
SB-400	Beam	212	1164	489.9	49936
SB-450	Beam	96	576	345.7	35241.7
SB-Com-300	Beam	264	1320	499.95	50963.6
SB-Com-200	Beam	24	120	34.365	3503.05
Total Weight				**1577.0**	**160754.5**

Table B. 31: Quantity of Structural Steel for Beams in G+8 Buildings

Section	Element Type	# Pieces	Total Length	Total Weight (kN)	Total Weight (kg)

SB-350	Beam	77	393	169.43	17271.47
SB-400	Beam	161	885	374.98	38224.61
SB-450	Beam	72	432	261.32	26638.26
SB-Com-300	Beam	192	960	363.602	37064.45
SB-Com-200	Beam	24	120	34.3649	3503.048
Total Weight				**1203.71**	**122701.84**

Table B. 32: Quantity of Structural Steel for Beams in G+5 Buildings

Section	Element Type	# Pieces	Total Length	Total Weight (kN)	Total Weight (kg)
SB-350	Beam	59	303	131.2	13376.5
SB-400	Beam	110	606	258.2	26323.4
SB-450	Beam	48	288	175.2	17862.6
SB-Com-300	Beam	120	600	227.25	23165.29
SB-Com-200	Beam	24	120	34.365	3503.048
Total Weight				**826.3**	**84230.9**

Table B. 33: Quantity of Concrete and Steel for Floors in G+11 Buildings

Section	Element Type	Total Weight (kN)	Total Weight (kg)	Net Weight of Concrete (m^3)
Com-Floor	Floor	13770	1355174	564.66
Com-Floor	Metal Deck	475.738	48495.2	-

Table B. 34: Quantity of Structural Steel for Floors in G+8 Buildings

Section	Element Type	Total Weight (kN)	Total Weight (kg)	Net Weight of Concrete (m^3)
Com-Floor	Floor	10327.5	1016381	423.49
Com-Floor	Metal Deck	356.8037	36371.4	-

Table B. 35: Quantity of Structural Steel for Floors in G+5 Buildings

Section	**Element Type**	**Total Weight (kN)**	**Total Weight (kg)**	**Net Weight of Concrete (m^3)**
Com-Floor	Floor	6885	677587	282.33
Com-Floor	Metal Deck	237.8691	24247.6	-
Total Weight		7122.8691	701835	282.33

Appendix C

C.1 Damage Limitation Check

Table C. 1: Drift Requirement Check for G+11 RCC Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.001482	0.001646	0.00049	0.00055	OK!	OK!
G+11	RS Max	0.001977	0.002349	0.00066	0.00078	OK!	OK!
G+10	RS Max	0.002219	0.002521	0.00074	0.00084	OK!	OK!
G+9	RS Max	0.002321	0.00248	0.00077	0.00083	OK!	OK!
G+8	RS Max	0.00245	0.002547	0.00082	0.00085	OK!	OK!
G+7	RS Max	0.002592	0.002657	0.00086	0.00089	OK!	OK!
G+6	RS Max	0.00272	0.002756	0.00091	0.00092	OK!	OK!
G+5	RS Max	0.002807	0.002846	0.00094	0.00095	OK!	OK!
G+4	RS Max	0.002855	0.002924	0.00095	0.00097	OK!	OK!
G+3	RS Max	0.002774	0.0029	0.00092	0.00097	OK!	OK!
G+2	RS Max	0.00253	0.002728	0.00084	0.00091	OK!	OK!
G+1	RS Max	0.001969	0.002198	0.00066	0.00073	OK!	OK!
Ground	RS Max	0.000896	0.001072	0.00036	0.00043	OK!	OK!

Table C. 2: Drift Requirement Check for G+11 Steel Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.000824	0.000808	0.00027	0.00027	OK!	OK!
G+11	RS Max	0.001166	0.001268	0.00039	0.00042	OK!	OK!
G+10	RS Max	0.001541	0.001712	0.00051	0.00057	OK!	OK!
G+9	RS Max	0.001867	0.00207	0.00062	0.00069	OK!	OK!
G+8	RS Max	0.002092	0.002292	0.00070	0.00076	OK!	OK!
G+7	RS Max	0.002301	0.002524	0.00077	0.00084	OK!	OK!
G+6	RS Max	0.002493	0.002731	0.00083	0.00091	OK!	OK!
G+5	RS Max	0.0026	0.002825	0.00087	0.00094	OK!	OK!
G+4	RS Max	0.002723	0.002996	0.00091	0.00100	OK!	OK!
G+3	RS Max	0.002769	0.00312	0.00092	0.00104	OK!	OK!
G+2	RS Max	0.002664	0.003174	0.00089	0.00106	OK!	OK!
G+1	RS Max	0.002275	0.002992	0.00076	0.00100	OK!	OK!
Ground	RS Max	0.001086	0.001607	0.00043	0.00064	OK!	OK!

Table C. 3: Drift Requirement Check for G+11 Composite Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.00088	0.000884	0.00029	0.00029	OK!	OK!
G+11	RS Max	0.001448	0.00153	0.00048	0.00051	OK!	OK!
G+10	RS Max	0.001667	0.001812	0.00056	0.00060	OK!	OK!
G+9	RS Max	0.001947	0.002116	0.00065	0.00071	OK!	OK!
G+8	RS Max	0.002139	0.002332	0.00071	0.00078	OK!	OK!
G+7	RS Max	0.002248	0.002455	0.00075	0.00082	OK!	OK!
G+6	RS Max	0.00242	0.002638	0.00081	0.00088	OK!	OK!
G+5	RS Max	0.002535	0.002767	0.00085	0.00092	OK!	OK!
G+4	RS Max	0.002603	0.002856	0.00087	0.00095	OK!	OK!
G+3	RS Max	0.002564	0.002837	0.00085	0.00095	OK!	OK!
G+2	RS Max	0.002343	0.00264	0.00078	0.00088	OK!	OK!
G+1	RS Max	0.001818	0.002083	0.00061	0.00069	OK!	OK!
Ground	RS Max	0.000765	0.000913	0.00031	0.00037	OK!	OK!

Table C. 4: Drift Requirement Check for G+8 RCC Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.001609	0.001924	0.00054	0.00064	OK!	OK!
G+8	RS Max	0.002104	0.002557	0.00070	0.00085	OK!	OK!
G+7	RS Max	0.00236	0.002688	0.00079	0.00090	OK!	OK!
G+6	RS Max	0.002455	0.002596	0.00082	0.00087	OK!	OK!
G+5	RS Max	0.002573	0.002644	0.00086	0.00088	OK!	OK!
G+4	RS Max	0.002693	0.002746	0.00090	0.00092	OK!	OK!
G+3	RS Max	0.00274	0.00281	0.00091	0.00094	OK!	OK!
G+2	RS Max	0.002595	0.002739	0.00087	0.00091	OK!	OK!
G+1	RS Max	0.002123	0.002338	0.00071	0.00078	OK!	OK!
Ground	RS Max	0.000944	0.001103	0.00038	0.00044	OK!	OK!

Table C. 5: Drift Requirement Check for G+8 Steel Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.0009	0.000893	0.00030	0.00030	OK!	OK!
G+8	RS Max	0.001272	0.001412	0.00042	0.00047	OK!	OK!
G+7	RS Max	0.001675	0.001906	0.00056	0.00064	OK!	OK!
G+6	RS Max	0.002021	0.002292	0.00067	0.00076	OK!	OK!

G+5	RS Max	0.002262	0.00252	0.00075	0.00084	OK!	OK!
G+4	RS Max	0.002481	0.002773	0.00083	0.00092	OK!	OK!
G+3	RS Max	0.002634	0.003012	0.00088	0.00100	OK!	OK!
G+2	RS Max	0.002565	0.003092	0.00086	0.00103	OK!	OK!
G+1	RS Max	0.002233	0.003028	0.00074	0.00101	OK!	OK!
Ground	RS Max	0.00109	0.001572	0.00044	0.00063	OK!	OK!

Table C. 6: Drift Requirement Check for G+8 Composite Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.0009	0.0009	0.00031	0.00030	OK!	OK!
G+8	RS Max	0.0015	0.0016	0.00051	0.00053	OK!	OK!
G+7	RS Max	0.0017	0.0019	0.00058	0.00063	OK!	OK!
G+6	RS Max	0.002	0.0022	0.00068	0.00073	OK!	OK!
G+5	RS Max	0.0022	0.0024	0.00075	0.00081	OK!	OK!
G+4	RS Max	0.0024	0.0026	0.00079	0.00086	OK!	OK!
G+3	RS Max	0.0025	0.0028	0.00084	0.00092	OK!	OK!
G+2	RS Max	0.0025	0.0028	0.00083	0.00093	OK!	OK!
G+1	RS Max	0.0021	0.0024	0.00069	0.00081	OK!	OK!
Ground	RS Max	0.001	0.0012	0.00041	0.00048	OK!	OK!

Table C. 7: Drift Requirement Check for G+5 RCC Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.001708	0.001862	0.00057	0.00062	OK!	OK!
G+5	RS Max	0.002309	0.002676	0.00077	0.00089	OK!	OK!
G+4	RS Max	0.002625	0.002886	0.00088	0.00096	OK!	OK!
G+3	RS Max	0.002741	0.00285	0.00091	0.00095	OK!	OK!
G+2	RS Max	0.002732	0.002857	0.00091	0.00095	OK!	OK!
G+1	RS Max	0.002274	0.00249	0.00076	0.00083	OK!	OK!
Ground	RS Max	0.001164	0.001446	0.00047	0.00058	OK!	OK!

Table C. 8: Drift Requirement Check for G+5 Steel Building Alternative

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.001	0.0009	0.00034	0.00032	OK!	OK!

G+5	RS Max	0.0014	0.0015	0.00048	0.00051	OK!	OK!
G+4	RS Max	0.0019	0.0021	0.00064	0.00070	OK!	OK!
G+3	RS Max	0.0023	0.0026	0.00077	0.00086	OK!	OK!
G+2	RS Max	0.0025	0.0029	0.00083	0.00097	OK!	OK!
G+1	RS Max	0.0023	0.0031	0.00078	0.00102	OK!	OK!
Ground	RS Max	0.0012	0.0018	0.00048	0.00071	OK!	OK!

Table C. 9: Drift Requirement Check for G+5 Composite Building

Story	Load Case	Interstorey Drift		(d_r/h)		Check Status	
		d_{rx}	d_{ry}	X	Y	X-d/n	Y-d/n
RF	RS Max	0.001003	0.001003	0.00033	0.00033	OK!	OK!
G+5	RS Max	0.001683	0.001683	0.00056	0.00056	OK!	OK!
G+4	RS Max	0.001974	0.001974	0.00066	0.00066	OK!	OK!
G+3	RS Max	0.002325	0.002325	0.00078	0.00078	OK!	OK!
G+2	RS Max	0.00247	0.00247	0.00082	0.00082	OK!	OK!
G+1	RS Max	0.002238	0.002238	0.00075	0.00075	OK!	OK!
Ground	RS Max	0.001204	0.001204	0.00048	0.00048	OK!	OK!

C.2 Stability Index Check

Table C. 10: Stability Index Check for G+11 RCC Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	(d_{rx}/h)*q	θ_x	Status Check
RF	Envelope (RS)	2936.679	292.16	0.0015	0.00	No P-Δ effects
G+11	Envelope (RS)	7651.202	534.27	0.002	0.01	No P-Δ effects
G+10	Envelope (RS)	12412.19	696.11	0.0022	0.01	No P-Δ effects
G+9	Envelope (RS)	17243.97	819.65	0.0023	0.02	No P-Δ effects
G+8	Envelope (RS)	22143.65	927.88	0.0025	0.02	No P-Δ effects
G+7	Envelope (RS)	27106.61	1026.5	0.0026	0.02	No P-Δ effects
G+6	Envelope (RS)	32140.94	1116.1	0.0027	0.03	No P-Δ effects
G+5	Envelope (RS)	37249.06	1200.3	0.0028	0.03	No P-Δ effects
G+4	Envelope (RS)	42406.05	1285.5	0.0029	0.03	No P-Δ effects
G+3	Envelope (RS)	47655.85	1375.4	0.0028	0.03	No P-Δ effects
G+2	Envelope (RS)	52958.47	1466.1	0.0025	0.03	No P-Δ effects
G+1	Envelope (RS)	58329.74	1541.5	0.002	0.02	No P-Δ effects
Ground	Envelope (RS)	60596.8	1557.7	0.0009	0.01	No P-Δ effects

Table C. 11: Stability Index Check for G+11 RCC Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2936.68	288.76	0.0016	0.01	No P-Δ effects
G+11	Envelope (RS)	7651.2	542.44	0.0023	0.01	No P-Δ effects
G+10	Envelope (RS)	12412.2	711.36	0.0025	0.01	No P-Δ effects
G+9	Envelope (RS)	17244	842.24	0.0025	0.02	No P-Δ effects
G+8	Envelope (RS)	22143.6	959.52	0.0025	0.02	No P-Δ effects
G+7	Envelope (RS)	27106.6	1066.9	0.0027	0.02	No P-Δ effects
G+6	Envelope (RS)	32140.9	1164.4	0.0028	0.03	No P-Δ effects
G+5	Envelope (RS)	37249.1	1255.2	0.0028	0.03	No P-Δ effects
G+4	Envelope (RS)	42406	1344.8	0.0029	0.03	No P-Δ effects
G+3	Envelope (RS)	47655.9	1437.3	0.0029	0.03	No P-Δ effects
G+2	Envelope (RS)	52958.5	1528.7	0.0027	0.03	No P-Δ effects
G+1	Envelope (RS)	58329.7	1604	0.0022	0.03	No P-Δ effects
Ground	Envelope (RS)	60596.8	1620.7	0.0011	0.01	No P-Δ effects

Table C. 12: Stability Index Check for G+11 Steel Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check
RF	Envelope (RS)	2469.506	216.8898	0.000824	0.003	No P-Δ effects
G+11	Envelope (RS)	6668.446	452.2544	0.001166	0.01	No P-Δ effects
G+10	Envelope (RS)	10867.39	627.6681	0.001541	0.01	No P-Δ effects
G+9	Envelope (RS)	15066.33	757.4169	0.001867	0.01	No P-Δ effects
G+8	Envelope (RS)	19275.33	857.5004	0.002092	0.02	No P-Δ effects
G+7	Envelope (RS)	23484.33	941.2903	0.002301	0.02	No P-Δ effects
G+6	Envelope (RS)	27693.33	1016.978	0.002493	0.02	No P-Δ effects
G+5	Envelope (RS)	31921.91	1090.633	0.0026	0.03	No P-Δ effects
G+4	Envelope (RS)	36150.5	1167.033	0.002723	0.03	No P-Δ effects
G+3	Envelope (RS)	40385.86	1246.692	0.002769	0.03	No P-Δ effects
G+2	Envelope (RS)	44628.63	1325.024	0.002664	0.03	No P-Δ effects
G+1	Envelope (RS)	48871.4	1389.239	0.002275	0.03	No P-Δ effects
Ground	Envelope (RS)	50349.97	1401.106	0.001086	0.01	No P-Δ effects

Table C. 13: Stability Index Check for G+11 Steel Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2469.506	185.7302	0.000808	0.004	No P-Δ effects
G+11	Envelope (RS)	6668.446	396.1659	0.001268	0.01	No P-Δ effects
G+10	Envelope (RS)	10867.39	555.2835	0.001712	0.01	No P-Δ effects
G+9	Envelope (RS)	15066.33	673.9636	0.00207	0.02	No P-Δ effects

G+8	Envelope (RS)	19275.33	768.0138	0.002292	0.02	No P-Δ effects
G+7	Envelope (RS)	23484.33	849.5608	0.002524	0.02	No P-Δ effects
G+6	Envelope (RS)	27693.33	923.2288	0.002731	0.03	No P-Δ effects
G+5	Envelope (RS)	31921.91	991.6931	0.002825	0.03	No P-Δ effects
G+4	Envelope (RS)	36150.5	1059.076	0.002996	0.03	No P-Δ effects
G+3	Envelope (RS)	40385.86	1127.872	0.00312	0.04	No P-Δ effects
G+2	Envelope (RS)	44628.63	1197.245	0.003174	0.04	No P-Δ effects
G+1	Envelope (RS)	48871.4	1258.655	0.002992	0.04	No P-Δ effects
Ground	Envelope (RS)	50349.97	1270.965	0.001607	0.02	No P-Δ effects

Table C. 14: Stability Index Check for G+11 Composite Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check
RF	Envelope (RS)	2341.307	229.13	0.00088	0.003	No P-Δ effects
G+11	Envelope (RS)	6165.235	458.37	0.00145	0.01	No P-Δ effects
G+10	Envelope (RS)	10041.69	616.63	0.00167	0.01	No P-Δ effects
G+9	Envelope (RS)	13928.48	737.2	0.00195	0.01	No P-Δ effects
G+8	Envelope (RS)	17835.76	831.7	0.00214	0.02	No P-Δ effects
G+7	Envelope (RS)	21789.58	912.58	0.00225	0.02	No P-Δ effects
G+6	Envelope (RS)	25743.4	987.79	0.00242	0.02	No P-Δ effects
G+5	Envelope (RS)	29719.94	1062.2	0.00254	0.02	No P-Δ effects
G+4	Envelope (RS)	33731.6	1139.9	0.0026	0.03	No P-Δ effects
G+3	Envelope (RS)	37781.95	1219.4	0.00256	0.03	No P-Δ effects
G+2	Envelope (RS)	41859.53	1294.7	0.00234	0.03	No P-Δ effects
G+1	Envelope (RS)	46006.87	1352.7	0.00182	0.02	No P-Δ effects
Ground	Envelope (RS)	47445.6	1362.2	0.00077	0.01	No P-Δ effects

Table C. 15: Stability Index Check for G+11 Composite Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2341.307	219.8529	0.000884	0.003	No P-Δ effects
G+11	Envelope (RS)	6165.235	446.5	0.00153	0.01	No P-Δ effects
G+10	Envelope (RS)	10041.69	603.624	0.001812	0.01	No P-Δ effects
G+9	Envelope (RS)	13928.48	722.6216	0.002116	0.01	No P-Δ effects
G+8	Envelope (RS)	17835.76	815.6602	0.002332	0.02	No P-Δ effects

G+7	Envelope (RS)	21789.58	895.1045	0.002455	0.02	No P-Δ effects
G+6	Envelope (RS)	25743.4	968.5815	0.002638	0.02	No P-Δ effects
G+5	Envelope (RS)	29719.94	1040.895	0.002767	0.03	No P-Δ effects
G+4	Envelope (RS)	33731.6	1116.367	0.002856	0.03	No P-Δ effects
G+3	Envelope (RS)	37781.95	1194.277	0.002837	0.03	No P-Δ effects
G+2	Envelope (RS)	41859.53	1268.97	0.00264	0.03	No P-Δ effects
G+1	Envelope (RS)	46006.87	1327.609	0.002083	0.02	No P-Δ effects
Ground	Envelope (RS)	47445.6	1337.305	0.000913	0.01	No P-Δ effects

Table C. 16: Stability Index Check for G+8 RCC Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check
RF	Envelope (RS)	2926.679	317.1912	0.001609	0.00	No P-Δ effects
G+8	Envelope (RS)	7490.202	583.1332	0.002104	0.01	No P-Δ effects
G+7	Envelope (RS)	12100.19	761.1991	0.00236	0.01	No P-Δ effects
G+6	Envelope (RS)	16780.97	890.0867	0.002455	0.02	No P-Δ effects
G+5	Envelope (RS)	21529.65	1000.099	0.002573	0.02	No P-Δ effects
G+4	Envelope (RS)	26341.61	1109.091	0.002693	0.02	No P-Δ effects
G+3	Envelope (RS)	31224.94	1223.584	0.00274	0.02	No P-Δ effects
G+2	Envelope (RS)	36182.06	1336.121	0.002595	0.02	No P-Δ effects
G+1	Envelope (RS)	41188.05	1426.215	0.002123	0.02	No P-Δ effects
Ground	Envelope (RS)	43521.55	1446.679	0.000944	0.01	No P-Δ effects

Table C. 17: Stability Index Check for G+8 RCC Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2926.679	312.0497	0.001924	0.01	No P-Δ effects
G+8	Envelope (RS)	7490.202	577.8112	0.002557	0.01	No P-Δ effects
G+7	Envelope (RS)	12100.19	752.1768	0.002688	0.01	No P-Δ effects
G+6	Envelope (RS)	16780.97	882.2841	0.002596	0.02	No P-Δ effects
G+5	Envelope (RS)	21529.65	997.0277	0.002644	0.02	No P-Δ effects
G+4	Envelope (RS)	26341.61	1108.517	0.002746	0.02	No P-Δ effects
G+3	Envelope (RS)	31224.94	1222.439	0.00281	0.02	No P-Δ effects
G+2	Envelope (RS)	36182.06	1333.671	0.002739	0.02	No P-Δ effects
G+1	Envelope (RS)	41188.05	1423.127	0.002338	0.02	No P-Δ effects
Ground	Envelope (RS)	43521.55	1443.977	0.001103	0.01	No P-Δ effects

Table C. 18: Stability Index Check for G+8 Steel Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check

RF	Envelope (RS)	2469.506	247.1333	0.0009	0.003	No P-Δ effects
G+8	Envelope (RS)	6544.455	503.1065	0.001272	0.01	No P-Δ effects
G+7	Envelope (RS)	10619.4	692.1096	0.001675	0.01	No P-Δ effects
G+6	Envelope (RS)	14694.35	830.2502	0.002021	0.01	No P-Δ effects
G+5	Envelope (RS)	18779.36	938.7181	0.002262	0.02	No P-Δ effects
G+4	Envelope (RS)	22864.37	1039.461	0.002481	0.02	No P-Δ effects
G+3	Envelope (RS)	26949.38	1141.451	0.002634	0.02	No P-Δ effects
G+2	Envelope (RS)	31053.98	1240.875	0.002565	0.02	No P-Δ effects
G+1	Envelope (RS)	35158.57	1320.996	0.002233	0.02	No P-Δ effects
Ground	Envelope (RS)	36629.73	1336.049	0.00109	0.01	No P-Δ effects

Table C. 19: Stability Index Check for G+8 Steel Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2469.506	220.1238	0.000893	0.003	No P-Δ effects
G+8	Envelope (RS)	6544.455	459.6391	0.001412	0.01	No P-Δ effects
G+7	Envelope (RS)	10619.4	639.3885	0.001906	0.01	No P-Δ effects
G+6	Envelope (RS)	14694.35	767.8907	0.002292	0.01	No P-Δ effects
G+5	Envelope (RS)	18779.36	865.8516	0.00252	0.02	No P-Δ effects
G+4	Envelope (RS)	22864.37	954.9136	0.002773	0.02	No P-Δ effects
G+3	Envelope (RS)	26949.38	1046.786	0.003012	0.03	No P-Δ effects
G+2	Envelope (RS)	31053.98	1140.749	0.003092	0.03	No P-Δ effects
G+1	Envelope (RS)	35158.57	1222.508	0.003028	0.03	No P-Δ effects
Ground	Envelope (RS)	36629.73	1238.949	0.001572	0.02	No P-Δ effects

Table C. 20: Stability Index Check for G+8 Composite Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check
RF	Envelope (RS)	2341.307	246.1508	0.000926	0.003	No P-Δ effects
G+8	Envelope (RS)	6041.244	486.5395	0.001523	0.01	No P-Δ effects
G+7	Envelope (RS)	9793.708	652.7259	0.001749	0.01	No P-Δ effects
G+6	Envelope (RS)	13556.51	778.2219	0.00204	0.01	No P-Δ effects
G+5	Envelope (RS)	17339.79	880.0988	0.002248	0.01	No P-Δ effects
G+4	Envelope (RS)	21169.62	976.7073	0.00237	0.02	No P-Δ effects
G+3	Envelope (RS)	24999.45	1074.592	0.002522	0.02	No P-Δ effects
G+2	Envelope (RS)	28852.01	1167.353	0.002479	0.02	No P-Δ effects
G+1	Envelope (RS)	32739.68	1239.614	0.002066	0.02	No P-Δ effects
Ground	Envelope (RS)	34025.82	1251.563	0.001014	0.01	No P-Δ effects

Table C. 21: Stability Index Check for G+8 Composite Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2341.307	236.1506	0.000913	0.00	No P-Δ effects
G+8	Envelope (RS)	6041.244	474.5349	0.001592	0.01	No P-Δ effects
G+7	Envelope (RS)	9793.708	640.0995	0.001884	0.01	No P-Δ effects
G+6	Envelope (RS)	13556.51	763.3799	0.002198	0.01	No P-Δ effects
G+5	Envelope (RS)	17339.79	861.8465	0.002428	0.02	No P-Δ effects
G+4	Envelope (RS)	21169.62	954.7018	0.00257	0.02	No P-Δ effects
G+3	Envelope (RS)	24999.45	1050.073	0.002757	0.02	No P-Δ effects
G+2	Envelope (RS)	28852.01	1142.598	0.00278	0.02	No P-Δ effects
G+1	Envelope (RS)	32739.68	1216.658	0.002418	0.02	No P-Δ effects
Ground	Envelope (RS)	34025.82	1228.985	0.001202	0.01	No P-Δ effects

Table C. 22: Stability Index Check for G+5 RCC Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check
RF	Envelope (RS)	2936.679	340.2647	0.001708	0.00	No P-Δ effects
G+5	Envelope (RS)	7510.202	627.4561	0.002309	0.01	No P-Δ effects
G+4	Envelope (RS)	12130.19	831.6022	0.002625	0.01	No P-Δ effects
G+3	Envelope (RS)	16820.97	999.4444	0.002741	0.02	No P-Δ effects
G+2	Envelope (RS)	21579.65	1150.928	0.002732	0.02	No P-Δ effects
G+1	Envelope (RS)	26401.61	1270.319	0.002274	0.02	No P-Δ effects
Ground	Envelope (RS)	28614.29	1298.945	0.001164	0.01	No P-Δ effects

Table C. 23: Stability Index Check for G+5 RCC Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2936.679	336.4598	0.001862	0.01	No P-Δ effects
G+5	Envelope (RS)	7510.202	630.6373	0.002676	0.01	No P-Δ effects
G+4	Envelope (RS)	12130.19	831.2362	0.002886	0.01	No P-Δ effects
G+3	Envelope (RS)	16820.97	995.12	0.00285	0.02	No P-Δ effects
G+2	Envelope (RS)	21579.65	1148.239	0.002857	0.02	No P-Δ effects
G+1	Envelope (RS)	26401.61	1272.378	0.00249	0.02	No P-Δ effects
Ground	Envelope (RS)	28614.29	1303.051	0.001446	0.01	No P-Δ effects

Table C. 24: Stability Index Check for G+5 Steel Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check

RF	Envelope (RS)	2443.458	276.8037	0.001008	0.003	No P-Δ effects
G+5	Envelope (RS)	6492.359	572.0938	0.001444	0.01	No P-Δ effects
G+4	Envelope (RS)	10541.26	805.5443	0.001927	0.01	No P-Δ effects
G+3	Envelope (RS)	14590.16	993.2035	0.002324	0.01	No P-Δ effects
G+2	Envelope (RS)	18649.12	1145.826	0.00248	0.01	No P-Δ effects
G+1	Envelope (RS)	22708.09	1259.577	0.002325	0.01	No P-Δ effects
Ground	Envelope (RS)	24152.88	1280.872	0.001193	0.01	No P-Δ effects

Table C. 25: Stability Index Check for G+5 Steel Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2443.458	240.9757	0.000948	0.00	No P-Δ effects
G+5	Envelope (RS)	6492.359	506.6108	0.001527	0.01	No P-Δ effects
G+4	Envelope (RS)	10541.26	715.7034	0.002105	0.01	No P-Δ effects
G+3	Envelope (RS)	14590.16	881.8818	0.002593	0.01	No P-Δ effects
G+2	Envelope (RS)	18649.12	1021.648	0.002902	0.02	No P-Δ effects
G+1	Envelope (RS)	22708.09	1134.311	0.003067	0.02	No P-Δ effects
Ground	Envelope (RS)	24152.88	1157.317	0.001785	0.01	No P-Δ effects

Table C. 26: Stability Index Check for G+5 Composite Building in X-direction

Story	Load Case	P (kN)	V_x (kN)	$(d_{rx}/h)*q$	θ_x	Status Check
RF	Envelope (RS)	2341.307	268.2873	0.001003	0.003	No P-Δ effects
G+5	Envelope (RS)	6041.244	538.927	0.001683	0.01	No P-Δ effects
G+4	Envelope (RS)	9793.708	740.3822	0.001974	0.01	No P-Δ effects
G+3	Envelope (RS)	13556.51	906.9688	0.002325	0.01	No P-Δ effects
G+2	Envelope (RS)	17339.79	1043.716	0.00247	0.01	No P-Δ effects
G+1	Envelope (RS)	21169.62	1145.355	0.002238	0.01	No P-Δ effects
Ground	Envelope (RS)	22359.24	1162.142	0.001204	0.01	No P-Δ effects

Table C. 27: Stability Index Check for G+5 Composite Building in Y-direction

Story	Load Case	P (kN)	V_y (kN)	$(d_{ry}/h)*q$	θ_y	Status Check
RF	Envelope (RS)	2341.307	254.5177	0.001003	0.003	No P-Δ effects
G+5	Envelope (RS)	6041.244	519.9781	0.001683	0.01	No P-Δ effects
G+4	Envelope (RS)	9793.708	718.5447	0.001974	0.01	No P-Δ effects
G+3	Envelope (RS)	13556.51	882.4404	0.002325	0.01	No P-Δ effects
G+2	Envelope (RS)	17339.79	1018.699	0.00247	0.01	No P-Δ effects
G+1	Envelope (RS)	21169.62	1122.541	0.002238	0.01	No P-Δ effects
Ground	Envelope (RS)	22359.24	1139.895	0.001204	0.01	No P-Δ effects

Appendix D

D.1 Modal Participation Mass Ratio Check

Table D. 1: Modal Participating Mass Ratio for G+11 RCC Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**2.251**	0.764	0	0	0.2513	0
2	2.197	0.764	0.7694	0.2468	0.2513	0
3	1.977	0.764	0.7694	0.2468	0.2513	0.7681
4	0.75	0.764	0.8758	0.6684	0.2513	0.7681
5	0.735	0.8696	0.8758	0.6684	0.6699	0.7681
6	0.665	0.8696	0.8758	0.6684	0.6699	0.8737
7	0.428	0.8696	0.9187	0.7602	0.6699	0.8737
8	0.421	0.9139	0.9187	0.7602	0.7515	0.8737
9	0.381	0.9139	0.9187	0.7602	0.7515	0.9161
10	0.288	0.9139	0.941	0.8279	0.7515	0.9161
11	0.282	0.9378	0.941	0.8279	0.822	0.9161
12	0.257	0.9378	0.941	0.8279	0.822	0.9387
13	0.209	0.9378	0.9541	0.8598	0.822	0.9387
14	0.204	0.9525	0.9541	0.8598	0.8574	0.9387
15	0.188	0.9525	0.9541	0.8598	0.8574	0.9527
16	0.159	0.9525	0.9629	0.8869	0.8574	0.9527
17	0.155	0.9629	0.9629	0.8869	0.8877	0.9527
18	0.144	0.9629	0.9629	0.8869	0.8877	0.9618
19	0.128	0.9629	0.9702	0.9077	0.8877	0.9618
20	0.124	0.9629	0.9702	0.9077	0.8877	0.9687
21	0.153	0.9706	0.9702	0.9077	0.9095	0.9687
22	0.142	0.9706	0.9768	0.9269	0.9095	0.9687
23	0.127	0.9706	0.9768	0.9269	0.9095	0.9744
24	0.122	0.9771	0.9768	0.9269	0.9283	0.9744
25	0.112	0.9771	0.9819	0.9426	0.9283	0.9744
26	0.104	0.9771	0.9819	0.9426	0.9283	0.9797
27	0.102	0.9771	0.9819	0.9426	0.9283	0.9797
28	0.102	0.9771	0.9819	0.9426	0.9284	0.9797
29	0.098	0.9771	0.9819	0.9426	0.9284	0.9797
30	0.097	0.9782	0.9819	0.9426	0.9318	0.9797

Table D. 2: Modal Participating Mass Ratio for G+11 Steel Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**2.561**	0	0.8243	0.1806	0	0
2	2.342	0.8048	0.8243	0.1806	0.2004	7.05E-07
3	2.246	0.8048	0.8243	0.1806	0.2004	0.8111
4	0.843	0.8048	0.922	0.7658	0.2004	0.8111
5	0.751	0.9055	0.922	0.7658	0.7362	0.8111
6	0.725	0.9055	0.922	0.7658	0.7362	0.9086
7	0.478	0.9055	0.9541	0.8293	0.7362	0.9086
8	0.416	0.9438	0.9541	0.8293	0.8061	0.9086
9	0.405	0.9438	0.9541	0.8293	0.8061	0.9441
10	0.321	0.9438	0.9697	0.8997	0.8061	0.9441
11	0.271	0.9631	0.9697	0.8997	0.8821	0.9441
12	0.266	0.9631	0.9697	0.8997	0.8821	0.9624
13	0.234	0.9631	0.9783	0.9221	0.8821	0.9624
14	0.19	0.9743	0.9783	0.9221	0.9117	0.9624
15	0.19	0.9743	0.9783	0.9221	0.9117	0.9732
16	0.179	0.9743	0.9835	0.9435	0.9117	0.9732
17	0.142	0.98	0.9835	0.9435	0.9323	0.9745
18	0.142	0.9813	0.9835	0.9435	0.9369	0.9801
19	0.141	0.9813	0.9869	0.9534	0.9369	0.9801
20	0.127	0.9813	0.9869	0.9534	0.9369	0.9801

Table D. 3: Modal Participating Mass Ratio for G+11 Composite Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**2.501**	0	0.7855	0.2209	0	0
2	2.409	0.7799	0.7855	0.2209	0.2267	0
3	2.193	0.7799	0.7855	0.2209	0.2267	0.7832
4	0.867	0.7799	0.8902	0.7075	0.2267	0.7832
5	0.86	0.8843	0.8902	0.7075	0.6966	0.7832
6	0.773	0.8843	0.8902	0.7075	0.6966	0.8873
7	0.511	0.8843	0.9299	0.778	0.6966	0.8873
8	0.497	0.9257	0.9299	0.778	0.7694	0.8873
9	0.453	0.9257	0.9299	0.778	0.7694	0.9274
10	0.353	0.9257	0.9513	0.8518	0.7694	0.9274
11	0.336	0.9484	0.9513	0.8518	0.8447	0.9274
12	0.311	0.9484	0.9513	0.8518	0.8447	0.9489
13	0.268	0.9484	0.9642	0.8841	0.8447	0.9489

14	0.248	0.9624	0.9642	0.8841	0.8797	0.9489
15	0.233	0.9624	0.9642	0.8841	0.8797	0.962
16	0.214	0.9624	0.9722	0.9111	0.8797	0.962
17	0.192	0.971	0.9722	0.9111	0.9079	0.962
18	0.185	0.971	0.9722	0.9111	0.9079	0.9702
19	0.176	0.971	0.9773	0.9249	0.9079	0.9702
20	0.153	0.9766	0.9773	0.9249	0.9233	0.9702

Table D. 4: Modal Participating Mass Ratio for G+8 RCC Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**1.739**	0.7701	0	0	0.2512	0
2	1.694	0.7701	0.7706	0.2519	0.2512	0
3	1.534	0.7701	0.7706	0.2519	0.2512	0.7744
4	0.575	0.7701	0.8801	0.6626	0.2512	0.7744
5	0.564	0.8765	0.8801	0.6626	0.672	0.7744
6	0.51	0.8765	0.8801	0.6626	0.672	0.8808
7	0.325	0.8765	0.9221	0.7584	0.672	0.8808
8	0.32	0.9192	0.9221	0.7584	0.7558	0.8808
9	0.289	0.9192	0.9221	0.7584	0.7558	0.9221
10	0.216	0.9192	0.9426	0.8259	0.7558	0.9221
11	0.211	0.9419	0.9426	0.8259	0.8261	0.9221
12	0.193	0.9419	0.9426	0.8259	0.8261	0.9436
13	0.155	0.9419	0.9553	0.859	0.8261	0.9436
14	0.151	0.9565	0.9553	0.859	0.864	0.9436
15	0.139	0.9565	0.9553	0.859	0.864	0.9559
16	0.121	0.9565	0.9657	0.8903	0.864	0.9559
17	0.119	0.9565	0.9657	0.8903	0.864	0.9654
18	0.108	0.9667	0.9657	0.8903	0.8945	0.9654
19	0.108	0.9667	0.9748	0.9182	0.8945	0.9654
20	0.107	0.9667	0.9748	0.9182	0.8945	0.9654
21	0.107	0.9668	0.9748	0.9182	0.8947	0.9654
22	0.107	0.9668	0.9748	0.9182	0.8947	0.9654
23	0.107	0.9668	0.9748	0.9182	0.8947	0.9654
24	0.105	0.9668	0.9748	0.9182	0.8947	0.9654
25	0.105	0.967	0.9748	0.9182	0.8951	0.9654
26	0.105	0.967	0.9748	0.9182	0.8951	0.9736
27	0.104	0.9734	0.9748	0.9182	0.9145	0.9736

Table D. 5: Modal Participating Mass Ratio for G+8 Steel Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**1.922**	0	0.8335	0.1753	0	0
2	1.741	0.8116	0.8335	0.1753	0.1985	1.64E-06
3	1.675	0.8116	0.8335	0.1753	0.1985	0.8178
4	0.625	0.8116	0.9283	0.7795	0.1985	0.8178
5	0.55	0.9124	0.9283	0.7795	0.7478	0.8178
6	0.532	0.9124	0.9283	0.7795	0.7478	0.9148
7	0.35	0.9124	0.9591	0.8407	0.7478	0.9148
8	0.299	0.9495	0.9591	0.8407	0.8187	0.9148
9	0.291	0.9495	0.9591	0.8407	0.8187	0.9497
10	0.232	0.9495	0.973	0.9063	0.8187	0.9497
11	0.19	0.9679	0.973	0.9063	0.8912	0.9497
12	0.187	0.9679	0.973	0.9063	0.8912	0.967
13	0.166	0.9679	0.9805	0.9271	0.8912	0.967
14	0.151	0.9679	0.9805	0.9271	0.8912	0.967
15	0.15	0.968	0.9805	0.9271	0.8917	0.967
16	0.136	0.9754	0.9805	0.9271	0.9132	0.967
17	0.131	0.9754	0.9805	0.9271	0.9132	0.9769
18	0.129	0.9754	0.9805	0.9271	0.9132	0.9769
19	0.128	0.9755	0.9805	0.9271	0.9136	0.9769
20	0.127	0.9756	0.9805	0.9271	0.914	0.9769
21	0.126	0.9756	0.9805	0.9271	0.914	0.9769
22	0.126	0.9756	0.9805	0.9271	0.914	0.9769
23	0.126	0.9756	0.9805	0.9271	0.914	0.9769
24	0.126	0.9756	0.9805	0.9271	0.914	0.9769
25	0.126	0.9756	0.9805	0.9271	0.914	0.9769
26	0.126	0.9756	0.9805	0.9271	0.914	0.9769
27	0.125	0.9756	0.9805	0.9271	0.914	0.9769

Table D. 6: Modal Participating Mass Ratio for G+8 Composite Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**1.91**	0	0.814	0.1925	0	0
2	1.907	0.8043	0.814	0.1925	0.2028	0
3	1.751	0.8043	0.814	0.1925	0.2028	0.8088
4	0.703	0.8043	0.9134	0.7422	0.2028	0.8088
5	0.665	0.9058	0.9134	0.7422	0.7262	0.8088

6	0.622	0.9058	0.9134	0.7422	0.7262	0.9087
7	0.413	0.9058	0.9485	0.8149	0.7262	0.9087
8	0.383	0.9438	0.9485	0.8149	0.8045	0.9087
9	0.363	0.9438	0.9485	0.8149	0.8045	0.9444
10	0.29	0.9438	0.9656	0.8839	0.8045	0.9444
11	0.261	0.9631	0.9656	0.8839	0.8769	0.9444
12	0.251	0.9631	0.9656	0.8839	0.8769	0.9621
13	0.223	0.9631	0.9747	0.9087	0.8769	0.9621
14	0.193	0.9733	0.9747	0.9087	0.9049	0.9621
15	0.19	0.9733	0.9747	0.9087	0.9049	0.9719
16	0.176	0.9733	0.9799	0.9291	0.9049	0.9719
17	0.152	0.9787	0.9799	0.9291	0.925	0.9719
18	0.148	0.9787	0.9799	0.9291	0.925	0.9719
19	0.134	0.9787	0.9799	0.9291	0.925	0.9719
20	0.129	0.9787	0.9799	0.9291	0.925	0.9719
21	0.128	0.9794	0.9799	0.9291	0.9274	0.9719
22	0.126	0.9794	0.9799	0.9291	0.9274	0.9719
23	0.126	0.9794	0.9799	0.9291	0.9274	0.9778
24	0.122	0.9794	0.9799	0.9291	0.9274	0.9778
25	0.122	0.9794	0.9799	0.9291	0.9274	0.9778
26	0.121	0.9798	0.9799	0.9291	0.9289	0.9778
27	0.114	0.9798	0.9799	0.9291	0.9289	0.9778

Table D. 7: Modal Participating Mass Ratio for G+5 RCC Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**1.238**	0	0.7902	0.238	0	0
2	1.2	0.7856	0.7902	0.238	0.2423	0
3	1.094	0.7856	0.7902	0.238	0.2423	0.7986
4	0.404	0.7856	0.9016	0.6934	0.2423	0.7986
5	0.396	0.8936	0.9016	0.6934	0.6885	0.7986
6	0.356	0.8936	0.9016	0.6934	0.6885	0.9044
7	0.223	0.8936	0.9375	0.7762	0.6885	0.9044
8	0.218	0.9335	0.9375	0.7762	0.7749	0.9044
9	0.197	0.9335	0.9375	0.7762	0.7749	0.9412
10	0.145	0.9335	0.9549	0.8434	0.7749	0.9412
11	0.142	0.9545	0.9549	0.8434	0.846	0.9412
12	0.129	0.9545	0.9549	0.8434	0.846	0.9583
13	0.11	0.9545	0.9683	0.8871	0.846	0.9583

14	0.108	0.9686	0.9683	0.8871	0.89	0.9583
15	0.108	0.9686	0.9683	0.8871	0.89	0.9702
16	0.107	0.9686	0.9683	0.8871	0.89	0.9702
17	0.107	0.9687	0.9683	0.8871	0.8905	0.9702
18	0.107	0.9687	0.9683	0.8871	0.8905	0.9702
19	0.105	0.9689	0.9683	0.8871	0.8909	0.9702
20	0.105	0.9689	0.9803	0.9285	0.8909	0.9702
21	0.105	0.9689	0.9803	0.9285	0.8909	0.9826
22	0.104	0.9689	0.9803	0.9285	0.8909	0.9826
23	0.104	0.9696	0.9803	0.9285	0.8937	0.9826
24	0.104	0.9812	0.9803	0.9285	0.9323	0.9826

Table D. 8: Modal Participating Mass Ratio for G+5 Steel Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**1.397**	0	0.8594	0.1563	0	0
2	1.187	0.8314	0.8594	0.1563	0.1872	5.38E-06
3	1.151	0.8314	0.8594	0.1563	0.1872	0.8372
4	0.424	0.8314	0.9443	0.8101	0.1872	0.8372
5	0.365	0.9296	0.9443	0.8101	0.7744	0.8372
6	0.352	0.9296	0.9443	0.8101	0.7744	0.9303
7	0.229	0.9296	0.9687	0.8632	0.7744	0.9303
8	0.19	0.9627	0.9687	0.8632	0.8507	0.9303
9	0.184	0.9627	0.9687	0.8632	0.8507	0.9616
10	0.149	0.9627	0.9687	0.8632	0.8507	0.9616
11	0.148	0.9628	0.9687	0.8632	0.8508	0.9616
12	0.146	0.9628	0.9794	0.9193	0.8508	0.9616
13	0.129	0.9666	0.9794	0.9193	0.8675	0.9616
14	0.127	0.9666	0.9794	0.9193	0.8675	0.9616
15	0.126	0.9668	0.9794	0.9193	0.8686	0.9616
16	0.125	0.9668	0.9794	0.9193	0.8686	0.9616
17	0.125	0.9668	0.9794	0.9193	0.8686	0.9616
18	0.124	0.9668	0.9794	0.9193	0.8686	0.9616
19	0.124	0.9668	0.9794	0.9193	0.8686	0.9616
20	0.124	0.9668	0.9794	0.9193	0.8686	0.9616
21	0.122	0.9671	0.9794	0.9193	0.8696	0.9616
22	0.122	0.9674	0.9794	0.9193	0.8702	0.9616
23	0.114	0.9674	0.9794	0.9193	0.8702	0.9766
24	0.111	0.9776	0.9794	0.9193	0.9136	0.9766

Table D. 9: Modal Participating Mass Ratio for G+5 Composite Alternatives

Mode	Period (sec)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	**1.382**	0	0.8445	0.164	0	0
2	1.375	0.8296	0.8445	0.164	0.1801	8.28E-07
3	1.241	0.8296	0.8445	0.164	0.1801	0.8355
4	0.509	0.8296	0.9346	0.7719	0.1801	0.8355
5	0.483	0.9266	0.9346	0.7719	0.7522	0.8355
6	0.437	0.9266	0.9346	0.7719	0.7522	0.9278
7	0.302	0.9266	0.9631	0.8425	0.7522	0.9278
8	0.28	0.9598	0.9631	0.8425	0.8349	0.9278
9	0.257	0.9598	0.9631	0.8425	0.8349	0.9579
10	0.216	0.9598	0.9742	0.8989	0.8349	0.9579
11	0.192	0.9727	0.9742	0.8989	0.8939	0.9579
12	0.18	0.9727	0.9742	0.8989	0.8939	0.9714
13	0.164	0.9762	0.9742	0.8989	0.905	0.9714
14	0.138	0.9762	0.9742	0.8989	0.905	0.9714
15	0.138	0.9762	0.9742	0.8989	0.9052	0.9714
16	0.121	0.9762	0.9742	0.8989	0.9052	0.9714
17	0.12	0.9762	0.9808	0.92	0.9052	0.9714
18	0.115	0.9762	0.9808	0.92	0.9052	0.9714
19	0.114	0.9769	0.9808	0.92	0.9077	0.9714
20	0.114	0.977	0.9808	0.92	0.908	0.9714
21	0.102	0.977	0.9808	0.92	0.908	0.9714
22	0.101	0.977	0.9808	0.92	0.908	0.9714
23	0.101	0.977	0.9808	0.92	0.908	0.9714
24	0.095	0.977	0.9808	0.92	0.908	0.9714

Appendix E

E.1 Structural Regularity

E.1.1 Regularity Check in Plan

E.1.1.1 Slenderness Check for Regularity

The slenderness of the building is tested for all floors and the results can be seen as follows in Table 1 for each structural alternative. The slenderness of the building λ shall be not higher than 4, such that $\lambda = {}^{L_{max}}/_{L_{min}}$.

Table E. 1: Slenderness Check for Regularity in case of all Structural alternatives

Story	L_{max}	L_{min}	λ	Status
RF	24	15	1.600	Ok!
G+11	24	15	1.600	Ok!
G+10	24	15	1.600	Ok!
G+9	24	15	1.600	Ok!
G+8	24	15	1.600	Ok!
G+7	24	15	1.600	Ok!
G+6	24	15	1.600	Ok!
G+5	24	15	1.600	Ok!
G+4	24	15	1.600	Ok!
G+3	24	15	1.600	Ok!
G+2	24	15	1.600	Ok!
G+1	24	15	1.600	Ok!
Ground	24	15	1.600	Ok!

☞ From the result shown in Table E.1 the building satisfies regularity condition for slenderness ratio.

E.1.1.2 Eccentricity and Torsional Radius Check for Regularity

The structural eccentricity shall be smaller than 30% of the torsional radius, which is calculated using: $e_{ox} \leq 0.30r_x$ and $e_{oy} \leq 0.30r_y$. The eccentricity e_{ox} and e_{oy} is the difference between the center of mass and center of rigidity while the torsional radius r_x and r_y is defined as the square root of the ratio of the torsional stiffness (K_M) to the lateral stiffness in one direction K_{FY} and K_{FX}, where $r_{x,i} = \sqrt{\frac{K_{M,i}}{K_{FY,i}}}$ and $r_{y,i} = \sqrt{\frac{K_{M,i}}{K_{FX,i}}}$.

Table E. 2: Eccentricity and Torsional Radius Check for G+11 RCC Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	0.3*r_x	0.3*r_y	Status
	m	m	m	m	m	m	
RF	-0.006	0.003	239.822	235.938	71.947	70.781	Ok!
G+11	-0.0077	0.0043	241.215	238.534	72.365	71.560	Ok!
G+10	-0.0091	0.0051	242.756	241.926	72.827	72.578	Ok!
G+9	-0.0108	0.0061	244.227	244.88	73.268	73.464	Ok!
G+8	-0.0126	0.0072	245.716	247.041	73.715	74.112	Ok!
G+7	-0.0143	0.0082	246.185	247.744	73.856	74.323	Ok!
G+6	-0.014	0.0078	246.168	247.545	73.850	74.263	Ok!
G+5	0.6648	-0.387	247.423	247.382	74.227	74.215	Ok!
G+4	-0.001	0.0002	246.147	245.61	73.844	73.683	Ok!
G+3	-0.0002	-0.0003	246.316	244.139	73.895	73.242	Ok!
G+2	0.000	-0.0003	246.19	241.917	73.857	72.575	Ok!

G+1	0.000	-0.0002	247.222	240.339	74.167	72.102	Ok!
Ground	0.6176	0.4531	246.403	215.804	73.921	64.741	Ok!

Table E. 3: Eccentricity and Torsional Radius Check for G+11 Steel Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	0.3*r_x	0.3*r_y	Status
	m	m	m	m	m	m	
RF	0.00	0.00	252.694	238.404	75.808	71.521	Ok!
G+11	0.00	0.00	253.739	239.148	76.122	71.744	Ok!
G+10	0.00	0.00	255.17	240.464	76.551	72.139	Ok!
G+9	0.00	0.00	256.7	241.901	77.010	72.570	Ok!
G+8	0.00	0.00	258.179	243.19	77.454	72.957	Ok!
G+7	0.00	0.00	260.098	244.641	78.029	73.392	Ok!
G+6	0.00	0.00	262.032	245.919	78.610	73.776	Ok!
G+5	0.00	0.00	263.971	246.828	79.191	74.048	Ok!
G+4	0.00	0.00	266.428	247.522	79.928	74.257	Ok!
G+3	0.00	0.00	269.475	248.254	80.842	74.476	Ok!
G+2	0.00	0.00	272.747	248.406	81.824	74.522	Ok!
G+1	0.00	0.00	277.545	249.06	83.263	74.718	Ok!
Ground	0.00	0.00	303.109	274.089	90.933	82.227	Ok!

Table E. 4: Eccentricity and Torsional Radius Check for G+11 Composite Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	0.3*r_x	0.3*r_y	Status
	m	m	m	m	m	m	
RF	0.00	0.00	249.844	235.530	74.953	70.659	Ok!
G+11	0.00	0.00	250.826	236.455	75.248	70.936	Ok!
G+10	0.00	0.00	252.333	237.971	75.700	71.391	Ok!
G+9	0.00	0.00	254.22	239.998	76.266	71.999	Ok!
G+8	0.00	0.00	256.075	242.005	76.823	72.602	Ok!
G+7	0.00	0.00	257.806	244.016	77.342	73.205	Ok!
G+6	0.00	0.00	258.947	245.591	77.684	73.677	Ok!
G+5	0.00	0.00	259.671	246.869	77.901	74.061	Ok!
G+4	0.00	0.00	259.985	247.957	77.996	74.387	Ok!
G+3	0.00	0.00	259.696	248.727	77.909	74.618	Ok!
G+2	0.00	0.00	258.558	248.958	77.567	74.688	Ok!
G+1	0.00	0.00	257.337	249.383	77.201	74.815	Ok!
Ground	0.00	0.00	246.403	215.804	73.921	64.741	Ok!

Table E. 5: Eccentricity and Torsional Radius Check for G+8 RCC Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	$0.3*r_x$	$0.3*r_y$	Status
	m	m	m	m	m	m	
RF	0.000	-0.001	247.966	241.522	74.390	72.456	Ok!
G+8	0.000	-0.001	248.66	243.732	74.598	73.120	Ok!
G+7	0.000	-0.001	249.437	246.994	74.831	74.098	Ok!
G+6	0.000	-0.001	250.106	249.551	75.032	74.865	Ok!
G+5	0.000	-0.001	251.6	251.549	75.480	75.465	Ok!
G+4	0.000	0.000	252.676	252.282	75.803	75.685	Ok!
G+3	0.000	0.000	252.971	251.429	75.891	75.429	Ok!
G+2	0.000	0.000	253.299	249.565	75.990	74.870	Ok!
G+1	0.000	0.000	254.935	248.174	76.480	74.452	Ok!
Ground	0.000	0.000	246.403	215.804	73.921	64.741	Ok!

Table E. 6: Eccentricity and Torsional Radius Check for G+8 Steel Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	$0.3*r_x$	$0.3*r_y$	Status
	m	m	m	m	m	m	
RF	0.00	0.00	258.829	233.011	77.649	69.903	Ok!
G+8	0.00	0.00	260.377	233.807	78.113	70.142	Ok!
G+7	0.00	0.00	262.171	235.119	78.651	70.536	Ok!
G+6	0.00	0.00	263.987	236.453	79.196	70.936	Ok!
G+5	0.00	0.00	265.957	237.666	79.787	71.300	Ok!
G+4	0.00	0.00	268.565	238.864	80.570	71.659	Ok!
G+3	0.00	0.00	271.279	239.667	81.384	71.900	Ok!
G+2	0.00	0.00	273.813	239.851	82.144	71.955	Ok!
G+1	0.00	0.00	277.981	240.951	83.394	72.285	Ok!
Ground	0.00	0.00	246.403	215.804	73.921	64.741	Ok!

Table E. 7: Eccentricity and Torsional Radius Check for G+8 Composite Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	$0.3*r_x$	$0.3*r_y$	Status
	m	m	m	m	m	m	
RF	0.00	0.00	250.976	234.837	75.293	70.451	Ok!
G+8	0.00	0.00	252.094	235.822	75.628	70.747	Ok!
G+7	0.00	0.00	253.526	237.273	76.058	71.182	Ok!
G+6	0.00	0.00	255.745	239.673	76.723	71.902	Ok!
G+5	0.00	0.00	257.461	241.572	77.238	72.472	Ok!
G+4	0.00	0.00	259.43	243.842	77.829	73.153	Ok!
G+3	0.00	0.00	259.802	244.808	77.941	73.442	Ok!
G+2	0.00	0.00	259.436	245.304	77.831	73.591	Ok!

G+1	0.00	0.00	258.516	245.836	77.555	73.751	Ok!
Ground	0.00	0.00	795.074	689.306	238.522	206.792	Ok!

Table E. 8: Eccentricity and Torsional Radius Check for G+5 RCC Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	$0.3*r_x$	$0.3*r_y$	Status
	m	m	m	m	m	m	
RF	0.000	0.000	255.023	244.808	76.507	73.443	Ok!
G+5	0.000	-0.0005	255.424	247.019	76.627	74.106	Ok!
G+4	0.000	-0.0005	255.176	249.873	76.553	74.962	Ok!
G+3	0.000	-0.0005	254.229	250.658	76.269	75.197	Ok!
G+2	0.000	-0.0004	254.823	249.927	76.447	74.978	Ok!
G+1	0.000	-0.0002	254.292	245.747	76.288	73.724	Ok!
Ground	0.000	0.0000	246.403	215.804	73.921	64.741	Ok!

Table E. 9: Eccentricity and Torsional Radius Check for G+5 Steel Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	$0.3*r_x$	$0.3*r_y$	Status
	m	m	m	m	m	m	
RF	0.000	0.000	266.934	241.639	80.080	72.492	Ok!
G+5	0.000	0.000	269.198	242.206	80.759	72.662	Ok!
G+4	0.000	0.000	271.723	243.374	81.517	73.012	Ok!
G+3	0.000	0.000	274.311	244.258	82.293	73.278	Ok!
G+2	0.000	0.000	277.04	244.503	83.112	73.351	Ok!
G+1	0.000	0.000	281.567	244.906	84.470	73.472	Ok!
Ground	0.000	0.0000	280.86	249.706	84.258	74.912	Ok!

Table E. 10: Eccentricity and Torsional Radius Check for G+5 Composite Alternative

Story	e_{ox}	e_{oy}	r_x	r_y	$0.3*r_x$	$0.3*r_y$	Status
	m	m	m	m	m	m	
RF	0.000	0.000	256.474	240.359	76.942	72.108	Ok!
G+5	0.000	0.000	257.187	240.817	77.156	72.245	Ok!
G+4	0.000	0.000	257.695	241.38	77.308	72.414	Ok!
G+3	0.000	0.000	259.238	243.361	77.771	73.008	Ok!
G+2	0.000	0.000	259.618	244.312	77.885	73.293	Ok!
G+1	0.000	0.000	261.071	246.777	78.321	74.033	Ok!
Ground	0.000	0.000	246.403	215.804	73.921	64.741	Ok!

- From Table E.2 – Table E.10 the result shows the building satisfies regularity condition for eccentricity and torsional radius.
- Therefore, the test result shows the requirement is fulfilled according to recommendation of ES EN-8 section 4.2.3.2 for all stories and the overall buildings are regarded as regular in plan.

E.1.2 Regularity Check in Elevation

E.1.2.1 Stiffness Regularity Check

Table E. 11: Stiffness Regularity Check for G+11 RCC Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	$\Delta K<0.3*K_i$ X-Dirn	$\Delta K<0.3*K_i$ Y-Dirn
RF	62188.45	48079.98	Not Ok!	Not Ok!
G+11	90930.89	70456.81	Ok!	Ok!
G+10	108310.44	91988.64	Ok!	Ok!
G+9	123921.23	116172.86	Ok!	Ok!
G+8	133968.76	131282.23	Ok!	Ok!
G+7	140830.43	141565.56	Ok!	Ok!
G+6	147070.14	150395.21	Ok!	Ok!
G+5	154435.02	158322.77	Ok!	Ok!
G+4	164388.79	167200.93	Ok!	Ok!
G+3	182306.96	182516.56	Ok!	Ok!
G+2	215016.21	210005.06	Ok!	Ok!
G+1	301632.26	285197.93	Not Ok!	Not Ok!
Ground	884330.28	806409.51	Ok!	Ok!

Table E. 12: Stiffness Regularity Check for G+11 Steel Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	$\Delta K<0.3*K_i$ X-Dirn	$\Delta K<0.3*K_i$ Y-Dirn
RF	78247.87	57894.13	Not Ok!	Not Ok!
G+11	124171.10	84937.82	Ok!	Ok!
G+10	136087.08	91795.57	Ok!	Ok!
G+9	138682.94	93991.39	Ok!	Ok!
G+8	142503.53	97970.41	Ok!	Ok!
G+7	143918.16	99022.07	Ok!	Ok!
G+6	144605.79	99755.31	Ok!	Ok!
G+5	150029.85	104244.18	Ok!	Ok!
G+4	154238.32	105872.70	Ok!	Ok!

G+3	162907.13	110142.41	Ok!	Ok!
G+2	182104.69	119712.51	Ok!	Ok!
G+1	235563.90	148389.73	Not Ok!	Not Ok!
Ground	663271.77	400965.79	Ok!	Ok!

Table E. 13: Stiffness Regularity Check for G+11 Composite Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	ΔK<0.3*K_i X-Dirn	ΔK<0.3*K_i Y-Dirn
RF	80979.61	62193.85	Ok!	Ok!
G+11	104337.52	78540.45	Ok!	Ok!
G+10	125984.01	93243.49	Ok!	Ok!
G+9	130957.11	97776.47	Ok!	Ok!
G+8	135903.95	101465.87	Ok!	Ok!
G+7	142938.84	106882.46	Ok!	Ok!
G+6	144699.86	108749.41	Ok!	Ok!
G+5	149520.26	112774.48	Ok!	Ok!
G+4	157031.63	119288.15	Ok!	Ok!
G+3	171733.10	132189.34	Ok!	Ok!
G+2	202550.06	158834.34	Ok!	Not Ok!
G+1	288634.44	231839.68	Not Ok!	Not Ok!
Ground	927539.32	757546.14	Ok!	Ok!

Table E. 14: Stiffness Regularity Check for G+8 RCC Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	ΔK<0.3*K_i X-Dirn	ΔK<0.3*K_i Y-Dirn
RF	64123.02	50167.75	Ok!	Ok!
G+8	92726.22	72279.35	Ok!	Ok!
G+7	110278.40	94061.43	Ok!	Ok!
G+6	126529.40	118832.37	Ok!	Ok!
G+5	137981.40	134659.99	Ok!	Ok!
G+4	148325.96	146703.99	Ok!	Ok!
G+3	162633.67	160661.44	Ok!	Ok!
G+2	189726.32	183073.46	Ok!	Ok!
G+1	257295.64	238761.81	Not Ok!	Not Ok!
Ground	798402.32	714507.98	Ok!	Ok!

Table E. 15: Stiffness Regularity Check for G+8 Steel Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	ΔK<0.3*Ki X-Dirn	ΔK<0.3*Ki Y-Dirn
RF	83282.48	63301.34	Not Ok!	Ok!
G+8	129595.61	89752.11	Ok!	Ok!
G+7	140625.24	95437.37	Ok!	Ok!
G+6	142905.67	96968.55	Ok!	Ok!
G+5	147306.71	100832.94	Ok!	Ok!
G+4	150667.03	102208.90	Ok!	Ok!
G+3	157230.20	104824.21	Ok!	Ok!
G+2	177986.70	116243.56	Ok!	Ok!
G+1	228993.42	142902.03	Not Ok!	Not Ok!
Ground	633089.65	408663.21	Ok!	Ok!

Table E. 16: Stiffness Regularity Check for G+8 Composite Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	ΔK<0.3*Ki X-Dirn	ΔK<0.3*Ki Y-Dirn
RF	84912.62	66652.69	Ok!	Ok!
G+8	107283.38	81686.11	Ok!	Ok!
G+7	128916.51	96311.23	Ok!	Ok!
G+6	133715.74	100642.93	Ok!	Ok!
G+5	139100.53	104749.71	Ok!	Ok!
G+4	147633.87	111594.92	Ok!	Ok!
G+3	153565.05	117207.09	Ok!	Ok!
G+2	172039.21	132900.20	Ok!	Ok!
G+1	232500.29	181069.09	Not Ok!	Not Ok!
Ground	651597.63	539452.15	Ok!	Ok!

Table E. 17: Stiffness Regularity Check for G+5 RCC Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	ΔK<0.3*Ki X-Dirn	ΔK<0.3*Ki Y-Dirn
RF	66089.63	52079.28	Ok!	Ok!
G+5	94783.56	73978.57	Ok!	Ok!
G+4	113304.72	96245.86	Ok!	Ok!
G+3	133023.96	122781.06	Ok!	Ok!
G+2	155088.38	144663.64	Ok!	Ok!
G+1	209734.16	186968.53	Not Ok!	Not Ok!
Ground	567355.76	479226.43	Ok!	Ok!

Table E. 18: Stiffness Regularity Check for G+5 Steel Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	ΔK<0.3*K_i X-Dirn	ΔK<0.3*K_i Y-Dirn
RF	88987.12	68288.90	Ok!	Ok!
G+5	136523.48	94401.92	Ok!	Ok!
G+4	148314.74	99564.80	Ok!	Ok!
G+3	154303.63	102004.38	Ok!	Ok!
G+2	170062.17	110762.19	Ok!	Ok!
G+1	211233.21	131567.08	Not Ok!	Not Ok!
Ground	558678.83	337601.32	Ok!	Ok!

Table E. 19: Stiffness Regularity Check for G+5 Composite Alternatives

Story	Stiffness-X (kN/m)	Stiffness-Y (kN/m)	ΔK<0.3*K_i X-Dirn	ΔK<0.3*K_i Y-Dirn
RF	88686.52	70740.75	Ok!	Ok!
G+5	110360.78	84793.93	Ok!	Ok!
G+4	132795.97	100129.38	Ok!	Ok!
G+3	139386.81	106136.88	Ok!	Ok!
G+2	151974.64	116639.20	Ok!	Ok!
G+1	187347.11	147031.26	Not Ok!	Not Ok!
Ground	358405.61	283908.55	Ok!	Ok!

☞ From Table E.11 – E.19 the result shows the buildings has not been satisfying stiffness regularity condition for some storey.

E.1.2.2 Mass Regularity Check

Table E. 20: Mass Regularity Check in case of G+11 Buildings Alternative

Story	RCC Mass (kg)	Steel Mass (kg)	Composite Mass (kg)	$W_i<2W_{i+1}$	$W_i<2W_{i-1}$
RF	295450.02	249161.65	242682.6	Ok!	Ok!
G+11	410688.78	359251.97	333601.92	Ok!	Ok!
G+10	423017.79	371842.52	336235.89	Ok!	Ok!
G+9	428916.21	371842.52	339477.43	Ok!	Ok!
G+8	436051.04	372338.63	341039.72	Ok!	Ok!
G+7	442771.61	372868.5	344450.85	Ok!	Ok!
G+6	449629.2	372868.5	346847.29	Ok!	Ok!
G+5	437245.38	373839.73	347989.1	Ok!	Ok!
G+4	463379.43	374865.57	350939.87	Ok!	Ok!

G+3	470491.95	375202.8	354692.99	Ok!	Ok!
G+2	478018.73	375923.79	358061.78	Ok!	Ok!
G+1	484178.46	376312.36	362988.56	Ok!	Not Ok!
Ground	221205.12	145901.18	139303.53	Ok!	Ok!

Table E. 21: Mass Regularity Check in case of G+8 Buildings Alternative

Story	RCC Mass (kg)	Steel Mass (kg)	Composite Mass (kg)	$W_i<2W_{i+1}$	$W_i<2W_{i-1}$
RF	295450.02	249161.65	242682.6	Ok!	Ok!
G+8	410688.78	359251.97	333601.92	Ok!	Ok!
G+7	423017.79	371842.52	336235.89	Ok!	Ok!
G+6	428916.21	371842.52	339477.43	Ok!	Ok!
G+5	436051.04	372338.63	341039.72	Ok!	Ok!
G+4	442771.61	372868.5	344450.85	Ok!	Ok!
G+3	449629.2	372868.5	346847.29	Ok!	Ok!
G+2	457038.07	373839.73	347989.1	Ok!	Ok!
G+1	463379.43	374865.57	350939.87	Ok!	Not Ok!
Ground	224085.82	144856.38	125913.2	Ok!	Ok!

Table E. 22: Mass Regularity Check in case of G+5 Buildings Alternative

Story	RCC Mass (kg)	Steel Mass (kg)	Composite Mass (kg)	$W_i<2W_{i+1}$	$W_i<2W_{i-1}$
RF	295450.02	246293.7	242682.6	Ok!	Ok!
G+5	410688.78	356384.02	333601.92	Ok!	Ok!
G+4	423017.79	368974.57	336235.89	Ok!	Ok!
G+3	428916.21	368974.57	339477.43	Ok!	Ok!
G+2	436051.04	369470.68	341039.72	Ok!	Ok!
G+1	442771.61	370000.55	344450.85	Ok!	Not Ok!
Ground	214803.22	142751.93	121306.96	Ok!	Ok!

- From Table E.20 – Table E.22 the result shows the buildings has not been satisfying mass regularity condition for some storey.
- Therefore, the test result shows the requirement is not fulfilled according to recommendation of ES EN-8 section 4.2.3.3 for some storey and the overall buildings regarded as irregular in Elevation.