አዲስ አበባ ሳይንስና ቴክኖሎጂ ዩኒቨርስቲ
Addis Ababa Science & Technology University
University for the Industry

COST ANALYSIS OF COMPOSITE AND REINFORCED CONCRETE CONDOMINIUM BUILDING FRAME STRUCTURTES

IN ADDIS ABABA

IN PARTIAL FULLFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE (M.Sc.) IN STRUCTURAL ENGINEERING

BY

NEGASH TADESSE

ADDIS ABABA, ETHIOIPIA

June, 2016

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF ADDISABABA SCIENCE AND TECHNOLOGEY UNIVERSITY IN PARTIAL FULLFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE (M.Sc.) IN STRUCTURAL ENGINEERING

Approved by Board of Examiners

Dr. Suresh Borra Advisor	______________________ Signature	______________________ Date
Dr.Ing. Temesegne Wondimu External Examiner	______________________ Signature	______________________ Date
Dr.Ing. Habtemu Itefa Internal Examiner	______________________ Signature	______________________ Date
Ato: Yesuf Esleman Chairman	______________________ Signature	______________________ Date

Declaration

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

Name: Negash Tadesse

Signature: ____________________

Place: Addis Ababa Science and Technology University

Date of submission: June, 2016

Acknowledgements

First of all I would like to thank my adviser Dr. Suresh Borra for his motivation, advice and knowledge that he provide me during the preparation of my thesis

I also want to thank post graduate coordinators Dr Habtemu Itefa and Dr. Mesay for their unconditional support from the beginning up to completing master program course.

I want to express my gratitude to all Dr.who given courses

Finally, I want to thank my sponsor ship Ethiopia road authority for facilitating and attaining master program.

LIST OF ACRONYMS

A	Cross-sectional area of the effective composite section neglecting concrete in tension
A_a	Cross-sectional area of the structural steel section
A_b	Cross-sectional area of bottom transverse reinforcement
A_c	Cross-sectional area of concrete
A_{ct}	Cross-sectional area of the tensile zone of the concrete
A_{fc}	Cross-sectional area of the compression flange
A_p	Cross-sectional area of profiled steel sheeting
A_{pe}	Effective cross-sectional area of profiled steel sheeting
A_s	Cross-sectional area of reinforcement
A_{sf}	Cross-sectional area of transverse reinforcement
$A_{s,r}$	Cross-sectional area of reinforcement in row r
A_t	Cross-sectional area of top transverse reinforcement
A_v	Shear area of a structural steel section
E_a	Modulus of elasticity of structural steel
$E_{c,eff}$	Effective modulus of elasticity for concrete
E_{cm}	Secant modulus of elasticity of concrete
E_s	Design value of modulus of elasticity of reinforcing steel
$(EI)_{eff}$	Effective flexural stiffness for calculation of relative slenderness
$(EI)_{eff,II}$	Effective flexural stiffness for use in second-order analysis
$(EI)_2$	Cracked flexural stiffness per unit width of the concrete or composite slab
F_l	Design longitudinal force per stud
F_t	Design transverse force per stud
F_{ten}	Design tensile force per stud
G_a	Shear modulus of structural steel
G_c	Shear modulus of concrete

I Second moment of area of the effective composite section neglecting concrete in tension

I_a Second moment of area of the structural steel section

I_{at} St.Venant torsion constant of the structural steel section

I_c Second moment of area of the un-cracked concrete section

I_{ct} St.Venant torsion constant of the un-cracked concrete encasement

I_s Second moment of area of the steel reinforcment

I_1 Second moment of area of the effective equivalent steel section assuming that the concrete in tension is un-cracked

I_2 Second moment of area of the effective equivalent steel section neglecting concrete in tension but including reinforcement

K_e ,$K_{e,II}$ Correction factors to be used in the design of composite columns

K_{sc} Stiffness related to the shear connection

K_β Parameter

K_0 Calibration factor to be used in the design of composite columns

L Length; span; effective span

L_e Equivalent span

L_i Span

L_0 Length of overhang

L_p Distance from centre of a concentrated load to the nearest support

L_s Shear span

L_x Distance from a cross-section to the nearest support

M Bending moment

M_a Contribution of the structural steel section to the design plastic resistance moment of the composite section

$M_{a,Ed}$ Design bending moment applied to the structural steel section

$M_{b,Rd}$ Design value of the buckling resistance moment of a composite beam

$M_{c,Ed}$ The part of the design bending moment applied to the composite section

M_{cr} Elastic critical moment for lateral-torsional buckling of a composite beam

M_{Ed} Design bending moment

$M_{Ed,i}$ Design bending moment applied to a composite joint *i*

$M_{el,Rd}$ Design value of the elastic resistance moment of the composite section

$M_{max,Rd}$ Maximum design value of the resistance moment in the presence of a compressive normal force

M_{pa} Design value of the plastic resistance moment of the effective cross-section of the profiled steel sheeting

M_{perm} Most adverse bending moment for the characteristic combination

$M_{pl,a,Rd}$ Design value of the plastic resistance moment of the structural steel section

$M_{pl,N,Rd}$ Design value of the plastic resistance moment of the composite section taking into account the compressive normal force

$M_{pl,Rd}$ Design value of the plastic resistance moment of the composite section with full shear connection

$M_{pl,y,Rd}$ Design value of the plastic resistance moment about the *y*-*y* axis of the composite section with full shear connection

$M_{pl,z,Rd}$ Design value of the plastic resistance moment about the *z*-*z* axis of the composite section with full shear connection

M_{pr} Reduced plastic resistance moment of the profiled steel sheeting

M_{Rd} Design value of the resistance moment of a composite section or joint

M_{Rk} Characteristic value of the resistance moment of a composite section or joint

$M_{y,Ed}$ Design bending moment applied to the composite section about the *y*-*y* axis

$M_{z,Ed}$ Design bending moment applied to the composite section about the *z*-*z* axis

N Compressive normal force; number of stress range cycles; number of shear connectors

N_a Design value of the normal force in the structural steel section of a composite beam

N_c Design value of the compressive normal force in the concrete flange

$N_{c,f}$ Design value of the compressive normal force in the concrete flange with full shear connection

$N_{c,el}$ Compressive normal force in the concrete flange corresponding to $M_{el,Rd}$

$N_{cr,eff}$ Elastic critical load of a composite column corresponding to an effective flexural stiffness

N_{cr} Elastic critical normal force

N_{c1} Design value of normal force calculated for load introduction

N_{Ed} Design value of the compressive normal force

$N_{G,Ed}$ Design value of the part of the compressive normal force that is permanent

N_p Design value of the plastic resistance of the profiled steel sheeting to normal force

$N_{pl,a}$ Design value of the plastic resistance of the structural steel section to normal force

$N_{pl,1Rd}$ Design value of the plastic resistance of the composite section to compressive normal force

$N_{pl,Rk}$ Characteristic value of the plastic resistance of the composite section to compressive normal force

$N_{pm,Rd}$ Design value of the resistance of the concrete to compressive normal force

N_s Design value of the plastic resistance of the steel reinforcement to normal force

N_{sd} Design value of the plastic resistance of the reinforcing steel to tensile normal force

$P_{l,Rd}$ Design value of the shear resistance of a single stud connector corresponding to Fl

$P_{pb,Rd}$ Design value of the bearing resistance of a stud

P_{Rd} Design value of the shear resistance of a single connector

P_{Rk} Characteristic value of the shear resistance of a single connector

$P_{t,Rd}$ Design value of the shear resistance of a single stud connector corresponding to Ft

R_{Ed} Design value of a support reaction

S_j Rotational stiffness of a joint

$S_{j,ini}$ Initial rotational stiffness of a joint

$V_{a,Ed}$ Design value of the shear force acting on the structural steel section

$V_{b,Rd}$ Design value of the shear buckling resistance of a steel web

$V_{c,Ed}$ Design value of the shear force acting on the reinforced concrete web encasement

V_{Ed} Design value of the shear force acting on the composite section

V_{ld} Design value of the resistance of the end anchorage

$V_{l,Rd}$ Design value of the resistance to shear

$V_{pl,Rd}$ Design value of the plastic resistance of the composite section to vertical shear

$V_{pl,a,Rd}$ Design value of the plastic resistance of the structural steel section to vertical shear

$V_{p,Rd}$ Design value of the resistance of a composite slab to punching shear

V_{Rd} Design value of the resistance of the composite section to vertical shear

V_t Support reaction

$V_{v,Rd}$ Design value of the resistance of a composite slab to vertical shear

$V_{wp,c,Rd}$ Design value of the shear resistance of the concrete encasement to a column web panel

Latin lower case letters

a Spacing between parallel beams; diameter or width; distance

b Width of the flange of a steel section; width of slab

b_b Width of the bottom of the concrete rib

b_c Width of the concrete encasement to a steel section

b_{eff} Total effective width

$b_{eff,1}$ Effective width at mid-span for a span supported at both ends

$b_{eff,2}$ Effective width at an internal support

$b_{eff,c,wc}$ Effective width of the column web in compression

b_{ei} Effective width of the concrete flange on each side of the web

b_{em} Effective width of a composite slab

b_f Width of the flange of a steel section

b_i Geometric width of the concrete flange on each side of the web

b_m Width of a composite slab over which a load is distributed

b_p Length of concentrated line load

b_r Width of rib of profiled steel sheeting

b_s Distance between centers of adjacent ribs of profiled steel sheeting

b_0 Distance between the centers of the outstand shear connectors; mean width of a concrete rib (minimum width for re-entrant sheeting profiles); width of haunch

c Width of the outstand of a steel flange; effective perimeter of reinforcing bar

c_y, c_z Thickness of concrete cover

d Clear depth of the web of the structural steel section; diameter of the shank of a stud connector; overall diameter of circular hollow steel section; minimum transverse dimension of a column

d_p Distance between the centroidal axis of the profiled steel sheeting and the extreme fibre of the composite slab in compression

d_s Distance between the steel reinforcement in tension to the extreme fibre of the composite slab in compression; distance between the longitudinal reinforcement in tension and the centroid of the beam's steel section

e	Eccentricity of loading; distance from the centroidal axis of profiled steel sheeting to the extreme fibre of the composite slab in tension
e_D	Edge distance
e_g	Gap between the reinforcement and the end plate in a composite column
e_p	Distance from the plastic neutral axis of profiled steel sheeting to the extreme fibre of the composite slab in tension
e_s	Distance from the steel reinforcement in tension to the extreme fibre of the composite slab in tension
f	Natural frequency
f_{cd}	Design value of the cylinder compressive strength of concrete
f_{ck}	Characteristic value of the cylinder compressive strength of concrete at 28 days
f_{cm}	Mean value of the measured cylinder compressive strength of concrete
$f_{ct,eff}$	Mean value of the effective tensile strength of the concrete
f_{ctm}	Mean value of the axial tensile strength of concrete
$f_{ct,0}$	Reference strength for concrete in tension
f_{lctm}	Mean value of the axial tensile strength of lightweight concrete
f_{sd}	Design value of the yield strength of reinforcing steel
f_{sk}	Characteristic value of the yield strength of reinforcing steel
f_u	Specified ultimate tensile strength
f_{ut}	Actual ultimate tensile strength in a test specimen
f_y	Nominal value of the yield strength of structural steel
f_{yd}	Design value of the yield strength of structural steel
$f_{yp,d}$	Design value of the yield strength of profiled steel sheeting
f_{ypm}	Mean value of the measured yield strength of profiled steel sheeting
f_1 ,f_2	Reduction factors for bending moments at supports
h	Overall depth; thickness
h_a	Depth of the structural steel section

h_c Depth of the concrete encasement to a steel section; thickness of the concrete flange; thickness of concrete above the main flat surface of the top of the ribs of the sheeting

h_f Thickness of concrete flange; thickness of finishes

h_n Position of neutral axis

h_p Overall depth of the profiled steel sheeting excluding embossments

h_s Depth between the centroids of the flanges of the structural steel section; distance between the longitudinal reinforcement in tension and the centre of compression

h_{sc} Overall nominal height of a stud connector

h_t Overall thickness of test specimen

k Amplification factor for second-order effects; coefficient; empirical factor for design shear resistance

k_c Coefficient

k_i Stiffness coefficient

$k_{i,c}$ Addition to the stiffness coefficient k_i due to concrete encasement

k_l Reduction factor for resistance of a headed stud used with profiled steel sheeting parallel to the beam

k_s Rotational stiffness; coefficient

k_{sc} Stiffness of a shear connector

k_{slip} Stiffness reduction factor due to deformation of the shear connection

$k_{s,r}$ Stiffness coefficient for a row r of longitudinal reinforcement in tension

k_t Reduction factor for resistance of a headed stud used with profiled steel sheeting transverse to the beam

$k_{wc,c}$ Factor for the effect of longitudinal compressive stress on transverse resistance of a column web

k_{ϕ} Parameter

k_1 Flexural stiffness of the cracked concrete or composite slab

k_2 Flexural stiffness of the web

l_{bc} ,l_{bs}	Bearing lengths
n	Modular ratio; number of shear connectors
n_f	Number of connectors for full shear connection
n_L	Modular ratio depending on the type of loading
n_r	Number of stud connectors in one rib
n_0	Modular ratio for short-term loading
r	Ratio of end moments
s	Transverse spacing centre-to-centre of the stud shear connectors
t	Age; thickness
t_e	Thickness of end plate
$t_{eff,c}$	Effective length of concrete
t_f	Thickness of a flange of the structural steel section
t_s	Thickness of a stiffener
t_w	Thickness of the web of the structural steel section
t_{wc}	Thickness of the web of the structural steel column section
t_0	Age at loading
$v_{L,Ed}$	Design longitudinal shear stress
w_k	Design value of crack width
x_{pl}	Distance between the plastic neutral axis and the extreme fibre of the concrete slab in compression
y	Cross-section axis parallel to the flanges
z	Cross-section axis perpendicular to the flanges; lever arm
z_0	Vertical distance

Greek lower case letters

α	Factor; parameter

α_{cr} Factor by which the design loads would have to be increased to cause elastic instability

α_M Coefficient related to bending of a composite column

$\alpha_{M,y}$, α_{Mz} Coefficient related to bending of a composite column about the *y*-y axis and the *z-z* axis respectively

α_{st} Ratio

β Factor; transformation parameter

β_c, β_i Parameters

γ_C Partial factor for concrete

γ_F Partial factor for actions, also accounting for model uncertainties and dimensional variations

γ_{Ff} Partial factor for equivalent constant amplitude stress range

γ_M Partial factor for a material property, also accounting for model uncertainties and dimensional variations

γ_{M0} Partial factor for structural steel applied to resistance of cross-sections, see EBCS EN 1993-1-1: 2013, 6.1(1)

γ_{M1} Partial factor for structural steel applied to resistance of members to instability assessed by member checks, see EBCS EN 1993-1-1: 2013, 6.1(1)

γ_{Mf} Partial factor for fatigue strength

γ_S Partial factor for reinforcing steel

γ_V Partial factor for design shear resistance of a headed stud

γ_{VS} Partial factor for design shear resistance of a composite slab

δ Factor; steel contribution ratio; central deflection

δ_{max} Sagging vertical deflection

δ_s Deflection of steel sheeting under its own weight plus the weight of wet concrete

$\delta_{s,max}$ Limiting value of δ_s

δ_u Maximum slip measured in a test at the characteristic load level

δ_{uk} Characteristic value of slip capacity

□□ $\sqrt{235 / f_y}$, where f_y is in N/mm^2

η Degree of shear connection; coefficient

η_a,□η_{ao} Factors related to the confinement of concrete

η_c, η_{co}, η_{cL} Factors related to the confinement of concrete

θ Angle

λ,λ_v Damage equivalent factors

λ_{glob}, λ_{loc} Damage equivalent factors for global effects and local effects, respectively

λ□ Relative slenderness

μ □ Coefficient of friction; nominal factor

μ_d Factor related to design for compression and uniaxial bending

μ_{dy} ,μ_{dz} Factor μ_d related to plane of bending

ν_a Poisson’s ratio for structural steel

ξ Parameter related to deformation of the shear connection

ρ Parameter related to reduced design bending resistance accounting for vertical shear

ρ_s Parameter; reinforcement ratio

$\sigma_{com,c,Ed}$ Longitudinal compressive stress in the encasement due to the design normal force

$\sigma_{c,Rd}$ Local design strength of concrete

σ_{ct} Extreme fibre tensile stress in the concrete

$\sigma_{s,max,f}$ Stress in the reinforcement due to the bending moment $M_{Ed,max,f}$

$\sigma_{s,min,f}$, Stress in the reinforcement due to the bending moment $M_{Ed,min,f}$

σ_s Stress in the tension reinforcement

$\sigma_{s,max}$ Stress in the reinforcement due to the bending moment M_{max}

$\sigma_{s,max,0}$ Stress in the reinforcement due to the bending moment M_{max}, neglecting concrete in tension

$\sigma_{s,0}$ Stress in the tension reinforcement neglecting tension stiffening of concrete

τ_{Rd} Design shear strength

τ_u Value of longitudinal shear strength of a composite slab determined from testing

$\tau_{u,Rd}$ Design value of longitudinal shear strength of a composite slab

$\tau_{u,Rk}$ Characteristic value of longitudinal shear strength of a composite slab

$\emptyset^*$ Diameter (size) of a steel reinforcing bar

φt Creep coefficient

χ Reduction factor for flexural buckling

χ_{LT} Reduction factor for lateral-torsional buckling

Ψ_L Creep multiplier

LIST OF TABLES

LIST OF FIGURES

TABLE OF CONTENTS

ABSTRACT

The steel and concrete composite construction is a method of construction of the structures for buildings and bridges, though it was introduced and practiced since so many years in various parts of the world , it is new to the Ethiopia. Because the methods for analysis and design are not popular as that of the R.C structures analysis and design. Apart these the availability of the components for the steel and concrete composite structures such as decking and shear connectors are not abundant in the market. Still it's usage is negligible quantity in building the structures. In relation to this, this is an attempt to show that how the steel and concrete composite structures are advantageous over the conventional structures in terms of the structural behavior and economical point view. In cost comparison the cost of the elements of the structures such as column and beam are higher than the cost of the R.C elements. Even though finally the less duration for completion of the composite structures and structural behavior point of the steel and concrete structures are more economical than R.C structures.

Key words: Composite, decking, shear connectors.

CHAPTER ONE

INTRODUCTION

1.0 Introduction

It is to be explained that the history of structural design may be in terms of continuous progress for obtaining optimal constructional systems with respect to engineering aesthetic and economic parameters. If our attention is mainly focused on the structure optimality it is sought through improvement of the materials and the form. Among all use of hybrid or composite material solution is of particular interest because of significant latent in overall performance improvement which can be obtained through modest changes in manufacturing and constructional technologies. By having Successful groupings of materials may even generate a innovative material, as seen in the event of reinforced concrete or more recently fiber reinforced plastics, however most repeatedly the synergy between structural components made of diverse materials to be fairly efficient choice. The most significant example in this field is characterized by the steel concrete composite construction, the vast potential which is not hitherto fully exploited after more than one hundred years since its first advent.

Composite buildings and composite bridges appeared in the U.S in the same year in 1894.

1) Concrete encased floor beams were used in the Methodist building in Pittsburgh.
2) Use of curved steel I beams embedded in concrete in the rock rapids bridge in rock rapids, Iowa

In these cases the composite action is relied on interfacial connection (bond) between concrete and steel. Reliability and efficiency of the bond was rather limited. Since the very beginning of the century attempts which were made to improve concrete and steel joining systems. But the breakthrough had occurred when the use of welded headed stud connectors in the year in 1956. By concurrence welded studs were used in the same year in a building (IBM Education building in Poughkeepsie, New York) and a bridge (Bad river bridge in Pierre, south Dakota) . From that time the metal studs have been by far the most popular shear transferring device in composite (steel concrete) systems for both building structure and bridge structures.

The noteworthy interest elevated by this new material stimulated a number of studies, both in North America and Europe on composite members (columns and beams) and connecting devices. Consequent to it and the increased knowledge empowered development of code provisions, which first seemed for buildings (The New York city buildings code in 1930) and later for bridges (the AASHO specifications in 1944). In effect a number of beneficial with respect to structural steel and reinforced were acknowledged and confirmed as:

1) High constructability (eg:- tubular in filled columns, floor decks, moment connections)
2) Reasonably satisfactory performance under fire conditions (all members and the whole system)
3) High strength and stiffness (columns, beams girders and moment connections)
4) Intrinsic toughness and ductility and satisfactory damping properties (eg:-encased columns, beam to column connections)

In relation to continuous development on the way to composite action was first focused on structural elements and members and was based mainly on technical innovation as in the case of steel concrete slabs with the use of profiled steel sheeting and of headed studs joined through the metal decking which fruitfully spread composite slab systems in the construction market since the 1960s. Invention of types of structural forms is the second important factor on which more latest developments (in the 1980s) were founded.

In case of tall buildings design philosophy a very recent trend considers the entire structural system as a body where diverse materials can live together in a fairly beneficial way. Steel, reinforced concrete, and composite steel concrete members and subsystems are used in a synergy way. These all together forms mixed systems often unite composite super frames whose columns handily built up by taking advantage of the steel erection columns have a tendency to become more and more similar to highly reinforced concrete columns. The growth of such systems stresses again the vitality of the composite construction which appears to increase rather than decay. Therefore, many researchers propose that for the construction of 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, less project period etc. Therefore from the researches Steel-concrete composite frame system can be an economically practical answer for high-rise in their countries perspectives. So It may be reflected that they are appropriate if these new technology is also applied to Addis Ababa buildings too.

So this thesis will likely to cover the cost implication of condominium buildings using the steel concrete composite materials. Researches and practical experience in the developed countries has shown that steel concrete composite system has several advantages over traditional reinforced concrete building structures. These include high strength-to-weight ratios, structural integrity, durable finishes, and dimensional stability, very much less project period and sound absorption. These advantages have led to a substantial increase in the use of composite materials for building constructions in the developed countries in recent years. As can be seen in these countries high-rise buildings currently are done by the use of composite buildings rather than concrete buildings. And researches in these developed countries suggest that for medium to high-rise buildings using composite material buildings are cost effective. However, in a developing country like Ethiopia/Addis Ababa this innovative technology is not yet practiced widely. The reason could be researches about the use of composite buildings are not widely done in our countries

1.1. Back ground

In pace with the economic development of the country the construction industry must also act properly to fulfill the needs of the economic development and aspirations of the people of the country. The developing country like Ethiopia is experiencing the fast economic development along with rapid urbanization. Particularly the housing sector there is a need of building structures like medium and high rise. By this time the cost of construction of these structures has raised to alarming levels because various reasons such as cost of land, construction materials cost, labor cost etc. So it is necessary to search for alternatives for reducing the cost of construction. In such one of the alternative is introducing the composite steel and concrete structures construction. This method of construction is a common practice in developed countries. And also they claimed for medium to high rise structures the composite steel and concrete has economical benefits than reinforced concrete structures.

But, as per the country Ethiopia researches to be conducted to compare the cost analysis of the composite steel concrete building structures and the Reinforced concrete structure. How dose going to affect the construction market of Ethiopia using this new innovative technology will cover my research? To which type of buildings is preferable, is it the traditional concrete building cost effective or Composite steel building structures cost effective? So in my research work an attempt will be made in this study to explore the cost analysis and feasibility of composite construction of condominium in Addis Ababa for medium to high-rise buildings.

1.2 Statement of the Problem

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

Use of composite or hybrid material is of particular interest, due to its significant potential in improving the overall performance through rather modest changes in manufacturing and constructional technologies. Thus, it is important to make a cost comparative study of steel-concrete composite structures with its equivalent normal reinforced concrete structure.

1.3 Objective

1.3.1 General objective

- ❖ To investigate the cost effectiveness of steel-concrete composite structure over reinforced concrete building structures.

1.3.2 Specific objectives

- ❖ Understand the various concepts for the analysis and design of the steel concrete composite structures according to the EBCS-4, reproduce them in thesis with background information.
- ❖ Analyze the manual design of the various composite elements.
- ❖ Produce the comparison tables for cost between the reinforced and composite steel and concrete structures.

1.4 Research Question.

Does the construction of high-rise building structures using composite materials is cost effective rather than using Reinforced Concrete?

1.5 Methodology

- ❖ A typical condominium building has been chosen for the feasibility study of steel concrete composite construction at Addis Ababa.
- ❖ Analysis is done by the finite element software ‘ETABS’ /or SAP for both RC framed and composite framed buildings.

- ❖ Understand the steel concrete composite system is which is analyzed and designed using EBCS 1995(4) , Draft EBCS code and Euro code 2002.
- ❖ Produce Cost comparison tables will be developed and also graphical representation is done wherever required like height Vs cost.
- ❖ Comparing the results of cost of the conventional reinforced concrete condominium building frames and steel-concrete composite condominium building frames.

1.6 Scope

The scope of this paper is limited to

- ❖ Since composite construction is a relatively new concept for Ethiopia, a brief introduction to composite building system will certainly help the design engineers to familiarize themselves with the components of this system.
- ❖ To fulfill the second objective of a typical condominium building with a floor area of 600m^2 to700 m^2 is selected for this study. Design and estimation of the cost of the building super structure only is conducted with similar floor pattern 7 storied buildings. Structural analysis for the buildings with reinforced concrete framing system as well as with steel-concrete composite framing system is performed using ETABS finite element/or SAP software.
- ❖ Cost of the building superstructure is estimated for the two framing systems in the context of Ethiopia

1.7 Significance of the study

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

- ❖ It will solve the existing high cost problem associated with the current condominium building in Addis Ababa by introducing the construction of composite structures in Ethiopia country with effective cost.
- ❖ It will help us to practice the use of light weighted structures that minimizes the cost consumption of the concrete and reinforcements well.
- ❖ And also because of these composite construction the construction time will come down , where the country like Ethiopia having high scarcity of housing can be fulfilled with the pace of the demand of the people by using these.
- ❖ It will also enable to advertise the use and advantage of composite structures to the contractors and stockholder of the condominium buildings by exploring its cost effective nesses.

1.8 .Literature review

- ❖ Up to now many researchers had been contributed for the re from various parts of the world. Therefore the published journals of them are now as references.

D. R. Panchal and P. M. Marathe. (2011) on Comparative Study of R.C., Steel and Composite (G+30 Storey) Building found that

- ❖ Steel option is better than R.C But the Composite option for high rise building is best option.
- ❖ The reduction in the dead weight of the Steel framed structure is 32 % with respect to R.C. frame Structure and Composite framed structure is 30 % with respect to R.C.framed structure.
- ❖ Steel and composite structure gives more ductility to the structure as compared to the R.C.C. which is best suited under the effect of lateral forces.

Mahbuba Begum1, Md. Serajus Salekin1, N.M. TauhidBelal Khan 1 and W. Ahmed has also conducted a research on cost analysis of steel concrete composite structures in Bangladesh ,the results show that RC construction is better for low rise building. For medium to high rise buildings steel concrete composite frame system is a better choice over reinforced concrete frame system from both economy & serviceability point of view.

CHAPTER TWO
MATERIALS AND LOADS

2.0 Introduction

Composite action infers that forces are transmitted between concrete and steel components. That is the concrete strength considerably affects the total and overall performance of the shear connection due to inverse relation among the resistance and the strain capacity of the material. Therefore the ability of redistribution of forces within the shear connection is restricted by the use high strength of concretes and as a result the design methods based on full redistribution of the shear forces supported by the connectors and use of plastic analysis is also limited.

For composite flexural elements the LRFD specifications(AISC 1993) prescribe that quality requirements of ACI(1989) mode concrete meet with rotary kiln produced C330 aggregates or ASTM C33 with concrete unit weight not less than 14.4 N/m^3 (90 pcf). This specification would allow for the development of the full flexural capacity according to test results by olgaard et al.

A restriction on the concrete strength is also imposed in composite compression members to safeguard consistency of the stipulations with available investigational data. The upper limit of the strength is 55 N /mm^2 (8 ksi) and the lower limit is 20 N/mm^2(3 ksi) for normal weight concrete and 27 N/mm^2(4 ksi) for light weight concrete.

But Euro code-4 recommendations are applicable for concrete strength classes up to C 50/60 ie, to concrete with cylinder characteristic strength up to 50 N/mm^2. The test data should justify for higher classes for light weight concretes with unit weight not less than 16 kN/m^3 is used.

Immediate concrete strength f_c is determined by the compression tests. But the strength under sustained loads is attained by applying a reduction factor 0.85 to f_c. Time dependence of concrete properties i.e, creep and shrinkage should be controlled when determining the response of the composite structures under sustained loads with particular reference to member stiffness. In order to treat them simple design methods can be adopted.

In composite beams strength and stiffness calculations of may be based on the converted cross section approach which is first developed for reinforced concrete structures which uses the modular ratio n=$\frac{E_s}{E_c}$in order to reduce the concrete area component to an equivalent steel area. Creep effect may be suitably accounted by defining the modular ratio in the analysis$\eta_{ef} = \frac{E_s}{E_{cef}} = \frac{E_s}{\{{}^{E_c}/_{1+\phi}\}}$ (1)

E_{cef} - An effective modules of elasticity of the concrete, $\phi - a\ creep\ coefficent$, approximating the ratio of creep strain to elastic strain for sustained compressive stress. For short term loading this coefficient may generally be assumed as "1" leading to reduction by half of the modular ratio, when significant portion of the live loads is likely to be on the structure quasi permanently a value of " $\phi = 2$" (i.e, a reduction by a factor 3) is recommended by Eurocode-4.In building design the effects of shrinkage are rarely critical except when slender beams are used (with span to depth ratio greater than 20.)

The total long term drying shrinkage strains ε_{sh} varies quite significantly depending on the amount of restraint from steel reinforcement, concrete and environmental characteristics.

According to the Eurocode-4 the following design values are provided for ordinary cases.

1) Dry environments
   $325X\ 10^{-6}$for normal weight concrete
   $500\ X\ 10^{-6}$for light weight concrete
2) Other environments and in filled members
   200 X$10^{-6}$for normal weight concrete
   $300\ X\ 10^{-6}$for light weight concrete

Finally the same value of the coefficient of thermal expansion may be conveniently assumed as for the steel components (i.e, 10 X10 $^{-6}$ $^{O}/_{C}$) even for light weight concrete.

Eurocode-4 proposed values of characteristic strength (f_c) , characteristic tensile strength (f_{ct}) and secant modulus of elasticity (E_c)

Table 1.0 Values for concrete strength

f_c n/mm^2	C20/25	C25/30	C30/35	C35/45	C40/50	C45/55	C50/60
f_{ct}	2.2	2.6	2.9	3.2	3.5	3.8	4.1
E_c n/mm^2	29	30.5	32	33.5	35	36	37

2.1Reinforcing steel

In most instances rebar's yield strength up to 500 N/mm^2(72 Ksi) are acceptable when plastic analysis is adopted for continuous beams the reinforcing steel should have adequate ductility. In the selection of the steel grade in particular when high strength steels are used this factor should hence be carefully considered. AISC specified a different requirement in encased composite columns is implied by the limitation of 380 N/mm^2 (55 Ksi) for the yield strength of reinforcement this is aimed at safeguarding that buckling of the reinforcement does not occur before complete yielding of the steel components.

2.2 Structural steel

For composite members Structural steel alloys with yield strength up to 355 N/mm^2 (50 ksi) for American grades used and high strength steel are available covering a yield strength range up to 780 N/mm^2 (113 ksi) for joints. . However substantial research is needed to cover the range of structural steels up to such levels of strength. Recently included in the Euro code -4 as Annex-H the rules applicable to steel grades Fe 420and Fe 460(with fy 420 N/mm^2 and 460 N/mm^2 respectively). Account is taken of the impact of the higher strain at yielding on the possibility to advance the full plastic sagging moment of the cross section and the greater significance of buckling of the steel components

As per the reinforcement the AISC specification applies the same limitation to the yield strength of structural steel.

2.3 Steel decking

The increased acceptance of the concrete decking associated with the trend towards higher flexural stiffness's allowing possibility of large run shored spans is clearly validated by the remarkable variety of products at present available. A wide range of forms , depths from 38 to 200mm(15 to 79 in.) thickness (0.76 to 1.52 mm)(5/24 to 5/12 in)and steel grades (with yield strength from 235 to 460 n/mm2(36 to 67 ksi) may be accepted.

Mild steels are generally used to ensure reasonable ductility. Protection requirements against corrosion determines the minimum thickness of the sheeting by Zinc coating should be selected, on the level of aggressiveness of the environment the total mass of which should depend. For internal floors in a non-aggressive environment a coating of total mass 275 g/m^{2}may be considered adequate

2.4 Shear connectors

The steel class of the connectors should be selected according to the method of fixing (Usually welding or screwing). The welding technique also should be considered for welded connectors (studs , anchors , hoops etc.)

Connectors do possess adequate deformation capability because design methods implying redistribution of shear forces among connectors. A problem arises regarding the mechanical properties to be essential to the stud connectors. Standards for material testing of welded studs are not available. The connectors are attained by cold working the bar material which is then exposed to localized plastic straining during the heading procedure. The Euro code hereafter specifies requirements for ultimate to yield strength ratio($\frac{f_U}{f_y} \geq 1.2$) and to the elongation at failure (not less than 12 % on a gauge length of 5.65$\sqrt{A_0}$ with , A_0- tensile specimen cross sectional area) to be fulfilled by the finished (cold drawn)product. Such a trouble in setting a suitable definition of necessities in terms of material properties leads many codes to advice, for studs cold bending tests after welding as a means to check the ductility.

CHAPTER THREE

BEAMS

3.0 Introduction

Flexural members are the one's first exploited composite action for which it represents a "natural" way to augment the response of the structural steel. Many varieties of composite beams are presently used in building and bridge construction. In building systems with reference to the steel members, either rolled or welded I- sections are the favorite solution. Particularly hollow sections are selected when torsional stiffness is a serious design factor. The trend on the way to larger spans (higher than 10m) and the necessity of freedom in accommodating services made the composite truss turn into more prevalent.

The main features of composite beam actions are briefly presented with reference to design. Due to the different behavioral aspects and the different level of complexity involved in the analysis and design of simply supported and continuous composite beams separate chapters are dedicated to these two cases.

3.1 Simply supported composite beam

Simply supported beams are subjected to shear and positive (sagging) moment. Composite steel concrete systems are advantageous in comparison with both structural steel members and reinforced concrete

1) In reinforced concrete beams, concrete is utilized in a more efficient way i.e, the concrete which lies in compression. But the concrete in tension which may be substantial portion of the member in reinforced concrete beams does not contribute to the resistance, whereas it increases the dead load. Moreover to avoid durability problems as reinforcement corrosion cracking of concrete in tension has to be controlled. Lastly construction methods can be chosen so that form work is not needed.
2) From the view of structural steel beams, a great part of the steel section or even the entire steel section is stressed in tension. The significance of local and flexural torsional buckling significantly reduced, if not excluded and plastic resistance can be

accomplished in most instances. Furthermore the sectional stiffness is extensively increased due to the role of the concrete flange deformability problems are consequently reduced and tend not to be critical.

Therefore it can be summarized by stating that simply supported composite beams are characterized by

1) An efficient use of both steel and concrete materials.

2) Has low sensitivity to local and flexural torsional buckling and high stiffness

The design and analysis may focus on few critical phenomenon's and the associated limit states .For the case of usual uniform loading pattern typical failure modes are schematically indicated below.

1) Mode -I, In the mid span cross section fails by attainment of the ultimate moment of resistance.
2) Mode- II , at the supports shear failure is dominant
3) Mode- III ,In the vicinity of the supports is by achievement of the maximum strength of the shear connection between concrete and steel

In order to avoid local failures a careful design of the structural details is necessary as the longitudinal shear failure of the slab along the planes shown in below figure, where the collapse under longitudinal shear does not involve the concrete flange or connectors

The analysis of composite simply supported beams is completed under the assumption that boundary (interface) slip can be omitted and the strength of the shear connections not critical.

Now the behavior of the elements is examined in detail. During construction the members can have shored construction (i.e., be temporarily supported) at intermediate points in order to reduce deformation and stresses of the steel section in the course of concrete casting. The construction processes can affect the structural behavior of the composite beam. In the case of the un shored construction the constructional loads and the weight of the fresh concrete are supported by the steel member alone till concrete has achieved at least seventy five percentage (75%) of its strength and the composite action can develop and the steel section has to be checked for all possible loading condition arising during construction. In particular the confirmation against lateral torsional buckling can become important because there is no benefit of the restraint provided by concrete slab and the steel section has to be properly braced horizontally.

In the case of shored constructions the overall load comprising self-weight is resisted by the composite member. This method of construction is advantageous from a statically point of view, but it may lead to significant increase of cost. The props are usually placed at the quartets and half of the span. So that results in full shoring. The effect of the construction method on the deformation and stress state of the members generally has to be accounted for in design calculations. It is exciting to observe that if composite section does possess required ductility the method of construction does not impact the ultimate capacity of the structure. The different responses of shored and unshored under service loading is very different but, if the elements are ductile enough the two structures attain the same ultimate capacity. In general the composite member ductility permits a number of phenomena, such as shrinkage of concrete, residual stresses in the steel sections and settlement of supports to be neglected at ultimate. On the other hand, all these actions can substantially influence the performance in service and ultimate capacity of the member in the case of slender cross sections susceptible to local buckling in the elastic range.

3.1.1 The effective width of concrete flange

The traditional form of composite beam can be modeled as a T-beam, the flange of which is the concrete slab. Despite the inherent in plane stiffness , the geometry characterized by a significant width for which the shear lag effect is non-negligible and the particular r loading condition(through concentrated load sat the steel- concrete interface) make the response of the concrete “flange” truly bi-dimensional in terms of distribution of strains and stresses. However it is possible to define a suitable breadth of the concrete flange permitting analysis of a composite beam as a mono dimensional member by means of the usual beam theory. The definition of such an effective width may be seen s the very first problem in the analysis of composite members in bending. The width can be determined by the equivalence between the responses of the beam computed via the beam theory and via refined model accounting for the actual bi-dimensional behavior of the slab. In principle the equivalence should be made with reference to the different parameters characterizing the member performance (i.e. elastic limit moment, the ultimate moment of resistance, the maximum deflections) and to different loading patterns.

A number of numerical studies of this problem are available in the literature base on equivalence of the elastic or inelastic response and rather refined approaches were developed to permit determination of elastic effective widths depending on the various design situations and related limit states. Some codes provided detailed and quite complex rules based on these studies. However recent parametric numerical analysis the findings of which were validated by experimental results indicated that simple expressions for effective width calculations can be adopted, if the effect of the non linear behavior of concrete and steel is taken into account.

Moreover the assumption in design global analysis of a constant value for the effective width "b_{eff}" leads to satisfactorily accurate results. These outcomes are reflected by recent design codes. In particular both the Euro Code -4 and AISC specifications assume in the analysis of simply supported beams , a constant effective width " b_{eff}" obtained as the sum of the effective widths "b_{ei}" at each side of the beam web determined via the following expression

$$b_{ei} = \frac{l_o}{8}$$

Where "l_o" is the beam span. The values of the beam span "b_{ei}" should be lower than one half the distances between center lines of adjacent beams or the distance to the slab free edge as shown in figure below.

3.1.2 Elastic analysis

When the interface slip can be neglected as assumed here a similar procedure for the analysis of the reinforced concrete sections can be adopted for composite members subject to bending. In fact the cross sections remain plane and the strains vary linearly along the section depth

The stress diagram is also linear if the concrete stress is multiplied by the modular ratio $n = \frac{E_s}{E_c}$ between the elastic modulus $E_s$and E_c of steel and concrete respectively. As further assumptions the concrete tensile strength is neglected as it is the presence of reinforcement placed in the concrete compressive area in view of it's modest contribution. The theory of the transformed sections can be used i.e, the composite section is replaced by an equivalent all steel section the flange of which has a breadth equal to $\frac{b_{eff}}{n}$, the translational equilibrium f the section requires the centroidal axis to be coincident with the neutral axis . Therefore the position of the neutral axis can

determined by imposing that the first moment of effective area of the cross section is equal to zero.

In the case of a solid concrete slab, and if the elastic neutral axis lies in the slab this condition leads to the equation

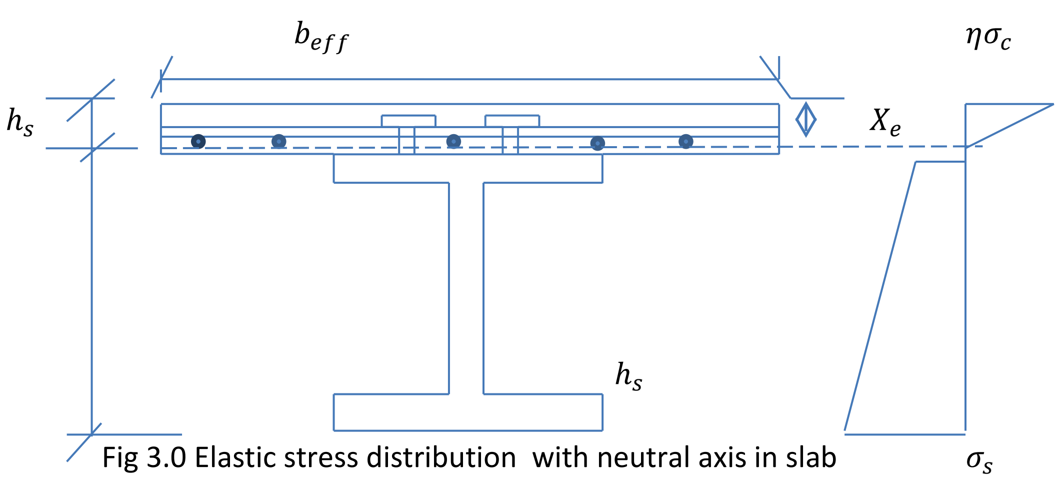


Fig 3.0 Elastic stress distribution with neutral axis in slab

$$s = \frac{1}{n}\frac{b_{eff}{X_e}^2}{2} - A_s\left[\frac{h_s}{2} + h_c - X_e\right] = 0 \qquad (2)$$

This is quadratic in terms of the unknown”X_e “(which is the distance of elastic neutral axis to the top fiber of the concrete slab) Once the value of”X_e “is calculated the second moment of area of the transformed cross section can be evaluated by the following expression.

$$I = \frac{1}{n}\frac{b_{eff}{X_e}^3}{3} + I_s + A_s\left(\frac{h_s}{2} + h_c - X_e\right)^2 \qquad (3)$$

The same procedure is used if the whole cross section is effective that is, if the elastic neutral axis lies in the steel profile .In this case it results.

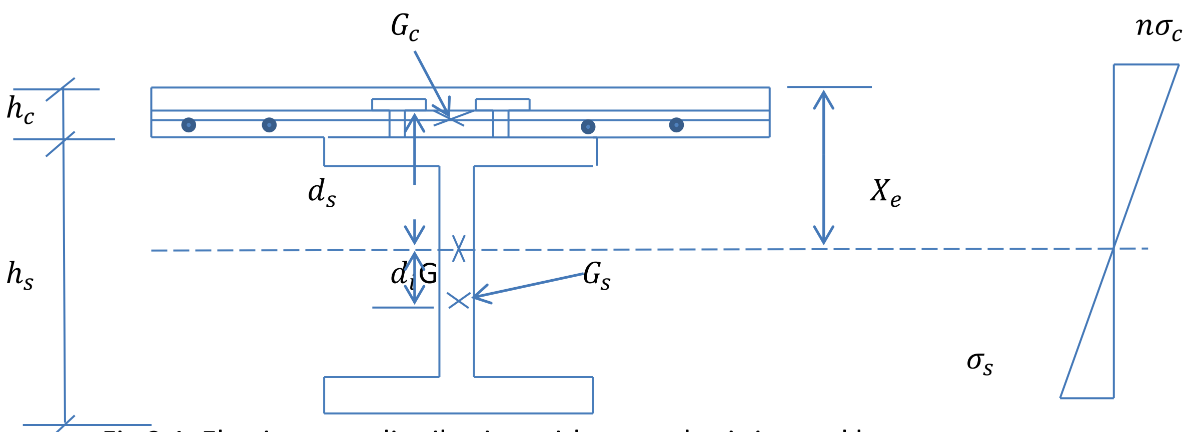


Fig 3.1 Elastic stress distribution with neutral axis in steel beam.

$$X_e = d_s + \frac{h_c}{2} \qquad (4)$$

Where $d_s = \frac{A_s}{A_s + b_{eff}\frac{h_c}{n}}\frac{h_c + h_s}{2}$ (5)

Where "d_s" is the distance between the centroid of the slab and the centroid of the transformed section.

$$I = I_s + \frac{1}{n}\frac{b_{eff}{h_c}^3}{12} + A^{\star}\frac{(h_s + h_c)^2}{4} \qquad (6)$$

$$A^{\star} = \frac{A_s b_{eff}\frac{h_c}{n}}{A_s + b_{eff}\frac{h_c}{n}} \qquad (7)$$

When the neutral axis depth and the second moment of area of the composite section are known the maximum stress of concrete in compression and structural steel in tension associated with a bending moment “M” are evaluated by the following expressions.

$$\sigma_c = \frac{1}{n}\frac{M}{I}X_e \qquad (8)$$

$$\sigma_s = \frac{M}{I}(h_s + h_c - X_e) \qquad (9)$$

These stresses must be lower than the relevant maximum design stresses allowed at the elastic limit condition.

In the case of Un shored construction determination of the elastic distribution determination of the elastic stress distribution should take into account that the steel alone resists al the permanent loads acting on the steel work before composite action can develop.

In many instances it is convenient to refer in cross sectional verifications to the applied moment rather than to the stress distribution. Therefore it is useful to define an elastic moment of resistance, as the moment at which the strength of either structural steel or concrete is achieved. This elastic limit moment can be determined as the lowest of the moments associated with the attainment of the elastic limit condition and obtained from the above two equations by imposing the maximum stress equal to the design limit stress values of the relevant material (i.e, $\sigma_c = f_{cd}$, $\sigma_s = f_{ysd}$)

As the nominal resistances are assumed as in the AISC specifications.

$$M_{el} = \min\{ f_{cd}\frac{nI}{X_e} f_{ysd}\frac{I}{h_s+h_c-X_e}\} \qquad (10)$$

The stress check is then indirectly satisfied if (and only if) it results

$$M \leq M_{el}$$

Where “M” is the maximum value of the bending moment for the load combination considered.

The elastic analysis approach based on the transformed section concept requires the evaluation of the modular coefficient “n” . Through an appropriate definition of this coefficient it is possible to complete the stress distribution under sustained loads as influenced by creep of concrete. In particular the reduction of the effective stiffness of the concrete due to creep is reflected by a decrease of the modular ratio and consequently the stress in the concrete slab decreases while the stress in the steel section increases – values can be obtained for the reduced effective modulus of elasticity E_{cef} of concrete, accounting for the relative proportion of long to short term loads. Codes may suggest values of "E_{cef}" defined accordingly to common load proportions in practice

Selection of the appropriate modular ratio “n” would permit in principle the variation of the stress distribution in the cross section to be checked at different during the life of the structure.

3.1.3 Plastic analysis

Refined non linear analysis of the composite beam can be carried out accounting for yielding of the steel section and inelasticity of the concrete slab. However the stress state typical of composite beams under sagging moments usually permits the plastic moment of the composite section to be achieved. In most instances the plastic neutral axis lies in the slab and the whole of the steel section in tension which results in

1) Local buckling not being a critical phenomenon
2) Concrete strains being limited, even when the full yielding condition of the steel beam is achieved.

Therefore the plastic method of analysis is applicable to most simply supported composite beams. Such a tool is so practically advantageous that it is the nonlinear design method for these members. In particular this approach is based on equilibrium equations at ultimate and does not depend on the constitutive relations of the materials and on the construction method.

The plastic moment can be computed by application of the rectangular stress block theory. Moreover the concrete may be assumed in composite beams to be stressed uniformly over the full depth X_{pl} of the compression side of the plastic neutral axis while the reinforced concrete sections usually the stress block depth is limited to$0.8X_{pl}$. The evaluation of plastic moment requires calculation of the following quantities.

3.1.4 Vertical shear

In composite elements shear is carried mostly by the web of the steel profile contribution of the concrete slab and steel flanges can be neglected in the design due to their width. The design shear strength can be determined by the same expression as for steel profiles

$$V_{pl} = A_v f_{ys,V}$$

A_v- The shear area of the steel section

$f_{ys,V}$ −The shear strength of the structural steel

With reference to the usual case of "I" steel sections and considering the different values assumed for $f_{ys,V}$ the AISC and Euro Code specifications provide some shear resistance in fact:

$V_{pl} = h_s t_w \left(0.6 f_{ys}\right) - - - - AISC$--- (11)

$V_{pl} = 1.04 h_s t_w \frac{f_{ys}}{\sqrt{3}} - - Euro\ Code -$ (12)

The design value of the plastic shear capacity is obtained either by multiplying the value of V_{pl} from equation (1) by a $\emptyset_v$ factor equal to 0.90 (AISC) or by using in equation (2) the design value of f_{ysd} (euro code). For slender beam webs (i.e, when their depth to thickness ratio is lower than $\frac{69}{\sqrt{\frac{235}{f_{ys}}}}$ (withf_{ys} $n/mm2$) . The shear resistance is suitable determined by taking into account web buckling in shear. The shear moment interaction is not important in simply supported beams (in fact for usual loading conditions where the moment is maximum the shear is zero. where the shear is maximum the moment is zero. But the situation in continuous beam is different.

3.1.5 Serviceability limits states

The adequacy of the performance under service loads requires that the use, efficiency or appearances of the structure are not impaired. Besides the stress state in concrete also needs to be limited due to the possible associated durability problems. Micro cracking of concrete (when stressed over "0.5f_c") may allow development of rebar's corrosion in aggressive environments. This aspect has to be addressed with reference to specific design conditions.

As to the member deformability the stiffness of composite beams in sagging bending is far higher than in the case of steel members of equal depth due to the significant contribution of the concrete flange. Therefore deflection limitation is less critical than in steel systems. However the effect of concrete creep and shrinkage has to be evaluated which may significantly increase the beam deformation as computed for short term loads. In service the beam should behave elastically.

Under the assumption of full interaction the usual formulae for beam deflection calculation can be used.

As an example the deflection under a U.D.L "P" is obtained as

$$\delta = \frac{5}{384}\frac{P}{E_s}\frac{l^4}{I} \qquad (13)$$

For un shored beams the construction sequence and the deflection of the steel section under the permanent loads has to be taken into account before development of composite action is added to the deflections of the composite beam under the relevant applied loads.

The value of the moment of inertia "I" of the transformed section and hence the value of "δ" depends on the modular ratio"n" . Therefore effective modulus (EM) theory enables the effect of concrete creep to be incorporated in design calculations without any additional complexity. Determination of the deflection under sustained loads simply requires that an effective modular ratio $n_{ef} = \frac{E_s}{E_{cef}}$ is used when computing “I”

The effect of the shrinkage ε_{sh} would be evaluated considering that the compatibility of the composite beam requires a tension force N_{sh} to develop in the slab equal to

$$N_{sh} = \varepsilon_{sh}\ E_{cef}\ b\ h_c \qquad (14)$$

This force is applied in the centeroid of the slab and due to equilibrium produces momentM_{sh} equal to

$$M_{sh} = N_{sh} d_s \qquad (15)$$

Where

$$d_s = \frac{A_s}{A_s + b_{eff}\ \frac{h_c}{n}} \frac{h_{c+h_s}}{2} \qquad (16)$$

This moment is constant along the beam. The additional deflection can be determined as$\delta_{sh} = \frac{M_{sh}\, l^2}{8E_s\, I} = 0.125\ \varepsilon_{sh} \frac{b h_c d_s}{n_{ef}\ I} l^2$ (17)

Typical ε_{sh} (shrinkage) values are given below

The influence of shrinkage on the deflection is usually important in dry environment and span to beam depth ratios greater than 20.

In partially composite beams the deflection associated with interface slip has also to be accounted for. The total deflection should be lower than limit values compatible with the serviceability requirements specific to the building system considered. Reference values by the Euro code-4 are presented below.

Euro code- 4 limiting values for vertical deflections.

Table 3.0 Limiting values for vertical deflection

Conditions	δ_{max}	δ_2
Roofs generally	$\frac{L}{200}$	$\frac{L}{250}$
Roofs frequently carrying personnel other than for maintenance	$\frac{L}{250}$	$\frac{L}{300}$
Floors generally	$\frac{L}{250}$	$\frac{L}{300}$
Floors and roofs supporting brittle finish on non flexible partitions	$\frac{L}{250}$	$\frac{L}{350}$
Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	$\frac{L}{400}$	$\frac{L}{500}$

Where δ_{max} can impair the appearance of the buildings

$$\frac{L}{250}$$

For cantilever "L" = twice the cantilever span.

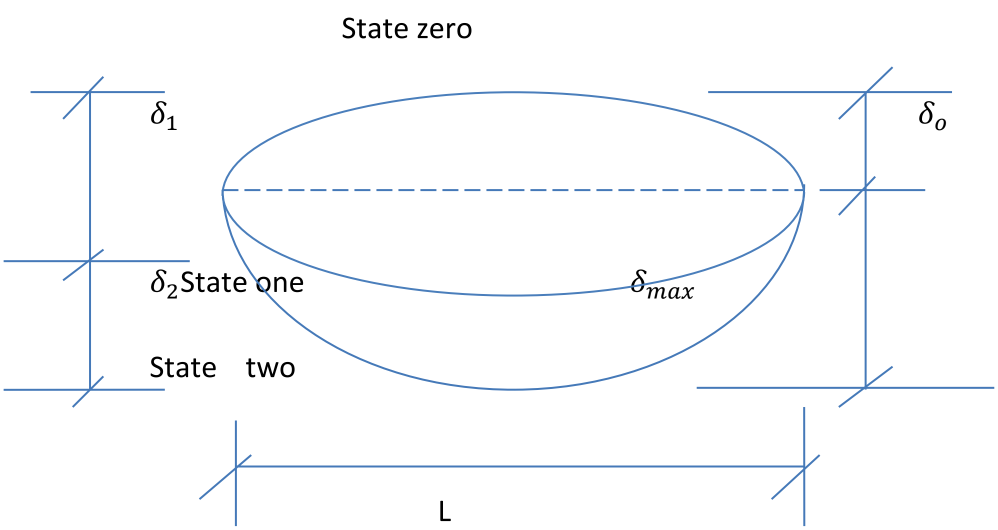


Fig 3.2 Deflection diagram

δ_{max} –Sagging in the final state relative to the straight line joining the supports.

δ_o – Pre camber (hogging) of the beam in the un loaded state (state zero)

δ_1 – Due to G (variation of the deflection of the beam due to permanent loads ((state one)

δ_2 – Due to Q (variation of the deflection of the beam due to the variable loading (State two)

3.2 Continuous beams

Beam continuity may represent an efficient statical solution with reference to both load capacity and stiffness. In composite buildings different kinds of continuity may in principle be achieved between beams and columns and possibly between adjacent beams. Furthermore the degree of continuity can vary significantly in relation to the performance of joints as to both strength and stiffness: Joints can be designed to be full or partial strength (strength) and rigid, semi rigid or pinned (stiffness). Despite the growing popularity of semi rigid partial strength joints rigid joints may still be considered the solution most used in building frames. Structural solutions for the flooring system were also proposed which allow an efficient use of beam continuity without burden of costly joints.

In bridge structures the use of continuous beams is very advantageous for it enables joints along the beams to be substantially reduced or even eliminated. This results in a remarkable reduction in design work load, fabrication and construction problems and structural cost.

From the structural point of view the main benefits of continuous beams are the following.

1) At the serviceability limit state: Deformability is lower than that of simply supported beams, providing a reduction of deflections and vibrations problems.
2) At the ultimate limit state: Moment redistribution may allow an efficient use of resistance capacity of the sections under positive and negative moment.

However the continuous beam is subjected to hogging (negative) bending moment at intermediate supports and it's response in these regions is not efficient as under sagging moments for the slab in tension and the lower part of the steel section is in compression. The first practical consequence is the necessity of adequate reinforcement in the slab besides the following problems arises:

1) At the serviceability limit state
   Concrete in tension cracks and the related problems such as control of crack width. The need of a minimum reinforcement etc. have to be accounted for in the design. Moreover deformability increases reducing the beneficial effect of the beam continuity.
2) At the ultimate limit state: Compression in steel could cause buckling problems either locally (in the bottom flange in compression and/or in the web) or globally(distortional lateral – torsional buckling)

Other problems can arise as well i.e., in simply supported beams the shear moment interaction is usually negligible, while at the intermediate supports of continuous beams both shear and bending can simultaneously attain high values and shear moment interaction becomes critical.

In this section the assumption of full shear concrete interaction is still maintained i.e., the shear connection is assumed to be a "full" shear connection.

3.2.1 Effective width

The general definition of the effective width "b_{eff}" is the same for the simply supported beam. The determination of the effective width along a continuous beam is certainly a more complex problem. Besides the type of loading and geometrical characteristics, several other parameters are involved which govern the strain (stress) state in the slab in the hogging moment regions. This complexity results in different provisions in the various national codes. However it should be noted that the variability of "b_{eff}" along the beam would imply if accounted for a substantial burden for design analysis.

For a continuous composite beam it was shown that the selection in the global analysis of a suitable effective width constant within each span allows us to obtain internal forces with satisfactory accuracy . On the other hand sectional verification should be performed with reference to the “local” value of “b_{eff}", The effective width in the moment negative zone allows evaluation of the reinforcement area that is effective in the section. The AISC provisions suggest use of equation $b_{ei} = \frac{l_0}{8}$ considering the full span length and center to center support for the analysis of the continuous beams. No recommendations are provided for sectional verification. Euro code-4 also recommends that in the global analysis “b_{eff}" is assumed to be constant over the whole length of each span and equal to the value at the mid span. The resistance of the critical cross sections is determined using the values of “b_{eff}" compute via $b_{ei} = \frac{l_0}{8}$, where the length “L” is replaced by the "l_0" defined in the below figure.. The effective width depends on the type of applied moment (hogging or sagging) and span(external, internal, cantilever). The value of the "b_{eff}" in the hogging moment enables determination of the effective area of steel reinforcement to be considered in design calculations.

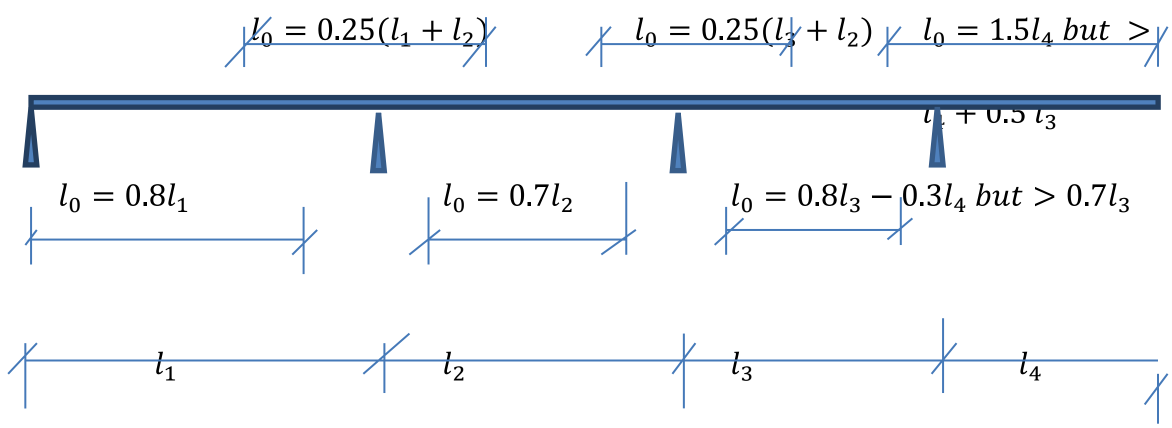


Fig 3.3 Effective length calculation

3.2.2 Local buckling and classification of cross section

Local buckling has to be accounted for in the very preliminary phase of design due to the occurrence of local buckling sections subjected to negative moment may not attain their plastic moment of resistance or develop the plastic rotation required for the full moment redistribution associated with the formation of a beam plastic mechanism.

In order to enable a preliminary assessment of strength and rotation capacity steel sections can be classified according to slenderness of the flanges and of the web. Four different member behaviors could be identified according to the importance of local buckling effects.

1) Members that can develop their full plastic moment capacity and also posses a rotation capacity sufficient to make in most practical cases, a beam plastic mechanism.
2) Members that can develop their plastic moment of resistance but then have limited rotation capacity.
3) Members that achieve the elastic moment of resistance associated with yielding of steel in the more stressed fiber, but not the plastic moment of resistance.
4) Members for which local buckling occurs still in the elastic range, so that even the elastic limit moment cannot be developed and elastic local buckling govern resistance.

3.2.3 Elastic Analysis of the cross section

In the negative moment regions where the concrete slab is subject to tensile stresses two main states of the composite beam can be identified with reference to the value of moment M_{cr} at which cracks start to develop when the bending moment is lower than M_{cr} the cross section is in the "state 1 un cracked" and it's un cracked moment of inertia I_1 can be evaluated by the same procedure of the section subjected to positive moment , when $M > M_{cr}$ the cross section enters the "state 2 cracked" characterized by the moment of inertia I_2 in this phase the elastic neutral axis X_e usually lies between the steel section , so that concrete does not collaborate to the stiffness and strength of the composite section. As a consequence the effective cross section of the composite beams consists only of steel (reinforcement bars and steel section). The moment of inertia I_2 and the stresses can be computed straight forwardly.

3.2.4 Plastic resistance of the cross section

In most cases as already discussed , sections in positive bending have the neutral axis within the slab, the steel section is hence fully(or predominantly) in tension and plastic analysis can be applied i.e., sections are I class-1 or class-2 (compact). The stress block model presented previously may be adopted for determining the plastic moment of resistance of the cross section.

Plastic analysis under hogging moment requires a preliminary classification of the cross section a plastic or compact. The fully plastic stress distribution of the composite cross section under hogging moments is shown below. The location of plastic neutral axis (i.e., the depth X_{pl}) is determined by imposing the equilibrium to the translation in the direction of the beam axis. Usually the neutral axis lies in the steel web and the value of X_{pl} is given by the following expression.

$$x_{pl} = \frac{h_s}{2} + h_c - \frac{F_{sr}/_2}{t_w f_{y.s}} \qquad (18)$$

F_{sr} − Where it is plastic strength of the reinforcement

The evaluation of the plastic moment is then carried out by imposing the equilibrium of the cross section to the rotation respect to the neutral axis.

$$M_{pl} = M_{pl.s} + F_{sr}\left(\frac{h_s}{2} + h_c - c\right) - \frac{{F_{sr}}^2}{4t_w f_{y.s}} \qquad (19)$$

Where “C” is the concrete cover.

CHAPTER FOUR

SLABS

4.0 Introduction

A slab in which profiled steel sheets are used initially as permanent shuttering and subsequently combine structurally with the hardened concrete and act as tensile reinforcement in the finished floor.

Since its development in North American the late 1950's composite floor systems using light gauge metal sheeting proved to be efficient solution, which became increasingly popular worldwide.

The steel deck serves:

First as working plat form and safety nesting system. Then as shuttering for the in-situ casting of concrete and finally as the bottom tensile reinforcement of the composite slab.

This capability of efficiently fulfilling different roles during construction and in service conditions is certainly one of the main factors of the success of composite flooring. A second key factor was related to the technological breakthrough provided by the possibility of welding stud connectors through the sheeting by means of convenient and reliable process.

The steel deck

A profiled steel sheeting may be seen as a mono directional; structural system. Its geometry-depth and thickness are dictated by the types of loads imposed during construction and by the economical requirement of maximizing the span without need for shoring. This purpose led to an increase of the deck depth from values lower than 50mm (2") to 70/75 (3") or more.(Even sheeting's 200mm (8") deep are presently available. The rib width is also important in relation to the composite beam performance. When the composite slab acts together with the steel beam and the ribs are transverse to the beam axis, narrow deck flutes would penalize the stud resistance often resulting in use of more studs, wider flutes tend to characterize present deck profiles. Wide rib profiles have a ratio $\frac{h_r}{b_r}$ between the rib height and average width greater than 2. Sheeting thickness range from 0.76 to 1.52mm (0.3 to 06. In). Deeper deck requires greater thickness in order not to have significant out of plane deformability which would reduce the shear transfer capacity in the composite slab. The necessary protection against corrosion is provided by zinc coating. In many instances the deflection under fresh is the parameter governing deck selection and design. Therefore steel requirements not fully exploited and use of high strength steels would generally not be advantageous.

In the construction stage the sheeting acts as working platform and shuttering system for the fresh concrete. The concrete is in liquid state and applies a load normal to each of the plate components. As a result the sheeting bends transversally due to the variation in lateral restraint from center to the edge. However it is acceptable for design purposes to model the wet concrete load as a uniform load. Besides the "Ponding" effect of concrete due to the deck deflection imposes an additional load. As a working platform the deck supports different construction loads including the ones related to concreting (heaping, Pipelines and pumping). Local vibration and impact effects may e significant and should be considered depending on site equipment and operation. Codes specify minimum construction loads to be used in addition to the weight of the fresh concrete for the design checking of the steel deck. In several cases the designer should assess the construction loads in order to better approximate the actual conditions uniform and concentrated live loads are given simulate the overall and local effects . Differences in value and distribution also reflect the different constructional practices.

Euro code-4 allows for the local nature of the construction loads and applies a characteristic load of 1.5 kN/m^2(30 Psf) distributed on any area 3m x 3m , while the remaining area should be subject to a load of 0.75 kN/m^2(15 psf). Furthermore the sheeting should be able resist in absence of concrete a concentrated load of 1 kN(0.22 kip) on a square area side 300mm (11,8 in) , so that a sufficient resistance against crushing of the profile is ensured. Partial safety factors should be applied to the characteristic load values given by Euro Code-4 in order to obtain the design load combination.

Elastic methods of analysis should be used to compute the internal forces. The slenderness ratios of the component plates are usually so high (typically about 50) that local buckling governs the resistance of the deck even when flats are stiffened. However the effect of the load buckling may be neglected in many instances and the design analysis performed assuming uniform stiffness. A rough account for the loss of effectiveness of some parts of continuous sheeting may then be obtained via partial moment redistribution. This approach is rather conservative. However more accurate calculations should involve iterative procedures to determine the effective cross section properties

In consideration of the fairly complex response and the many parameters involved(some of which as the variation of the yield strength in the cross section due to the forming process and the presence of embossments are difficult to be accounted for in a simple yet reliable way). A number of design aids were developed and are available to practitioners mainly providing values of stiffness and resistance based on tests commissioned by the manufacturers. The verification in service is based on a check of mid span deflection "δ" under the wet concrete weight. The deflection limit is assumed as $\frac{l}{180}$ or 20mm whichever is minimum. TheEuro Code prescribes that when this limit is exceeded the effect of concrete ponding should be allowed for in the design. A uniform load corresponding to an additional concrete thickness of 0.7 δ may be assumed for that purpose.

4.1. Composite slab

When concrete has achieved its full strength the deck acts as composite slab and the steel sheeting serves as the bottom reinforcement under sagging moments. The concrete is continuous over the whole floor span. However the amount of bottom tensile reinforcement provided by the sheeting is sufficient to make it advantageous to consider and design the slab as simply supported. A design method based on elastic un cracked analysis maximum allowable loads for a continuous slab lower those determined assuming the slab as simply supported. Top reinforcement is present anyway for shrinkage and temperature effects as well as for crack control over the intermediate supports.

1) A minimum amount of reinforcement is specified by Euro code-4 as 0.2% of the cross section of the concrete above the steel ribs for un shored construction and 0.4% for shored construction

Further requirements relate to the minimum values of the total depth h_t of the slab and of the thickness of the concrete coverh_c. These values are similar in the Euro code and ASCE specifications. For they basically reflect past satisfactory performances and satisfy the need for consistency with other detailing rules. The analysis usually considers a slab strip of unit width which depends on the system of units adopted, 1m (S.I units), 1ft (American Unit system)

4.1.1. Minimum values of slab depth

Depth	Euro Code-4	ASCE
h_t	80^{*}(90)	90
h_c	40^{*}(50)	50

⋆ —For slabs acting comparatively with the beam

$Table$ 4.0 Minimum values of slab depth

4.1.2 Simply supported composite slabs

The same modes of failures already identified for composite beams may be associated with ultimate conditions of a composite slab

1) Flexural resistance
2) Longitudinal shear
3) Vertical shear

The type of mechanism for longitudinal shear transfer which involves bond and frictional interlock. The condition of complete shear connection difficult to achieve for the slab geometrics and spans typical of current design practice. Therefore collapse is primarily due to the loss of shear transfer capacity at the steel deck concrete interface (failure mode III). However the bending capacity may become the critical parameter for the slab with full shear connection.(either long slabs or slabs with efficient end anchorage) and vertical shear may govern design of slabs with fairly low span to depth ratios.

Longitudinal shear capacity

The shear transfer mechanism in concrete slabs is fairly complex. Besides material properties its effiecnecy depends upon many parameters in particular to those related to the sheeting and to its deformations such as the geometry(height shape and orientation) and spacing of embossments and the out of plane flexibility of the sheeting component plates. In 1976porter and Ekberg proposed the empirical method on which most design code recommendations are based. The so called M-K method this approach conveniently relates the vertical shear resistance"V_u" at the shear bond failure and the shear span "l_s" in which that failure occurs.

The method requires that the minimum vertical shear not exceed the longitudinal shear bond capacity. The un factored shear bond capacity may be expressed as.

$$V_{lu} = bd_p(\frac{mA_P\sqrt{f_c}}{bl_s}) + k\sqrt{f_c} \qquad (20)$$

A_P –The cross sectional area of the steel deck for unit width
b- The specimen width
d_p –The distance of the top fiber of the composite slab to the centroid of the steel deck.
The factors m and k are the slope and the ordinate intercept of the shear bond line.
Test results by Evans and Wright showed that the influence of the concrete strength is modest and may be neglected. Based on this outcome the above formula in the Euro Code-4 becomes

$$V_{lu} = bd_p(\frac{mA_P}{bl_s}) + k \qquad (21)$$

4.1.2.1 The flexural capacity

Traditional approaches to the analysis and design of composite slabs adopted and adapted the methods and design criteria developed for reinforced concrete elements. The steel sheeting is hence considered and modeled as the tensile reinforcement and limitations are imposed to ensure that the failure is associated with a “ductile” mode that is the crushing of concrete in compression is avoided while the steel reinforcement may achieve its full plasticity strength. Restrictions are made on the depth of the concrete in compression or a range is defined as in the ASCE standards (1991) with in which only the plastic stress block analysis can be applied. Consistently with the reinforced concrete analogy the parameter adopted to define the upper boundary of such a range is a suitable “reinforcement ratio” obtained by modifying the relevant expression in the ACI standards and hence assumed as the ratio of the steel deck area to the effective concrete area in the unit slab width” b”

$\rho = \frac{A_p}{bd_p}$ The balanced value of it ρ_b also defined to the ACI standards(1989) is

$$\rho_b = \left(\frac{0.85\beta f_c}{f_{y.p}}\right)\frac{\varepsilon_c E_s(h_t - h_p)}{(\varepsilon_c E_s + f_{y.p})D_p} \qquad (22)$$

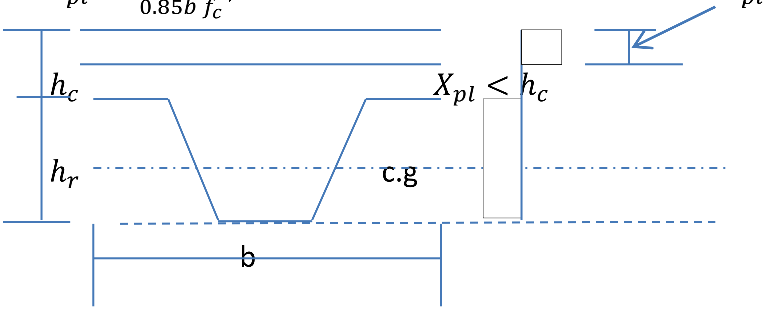
Where β is stress block depth factor depending on concrete strength($\beta = 0.85\ for\ f_c \leq 28\,{}^{n}/_{mm^2}$) and decreases by 0.05 for each increase of f_c equal to 7 n/mm2 down to a minimum value of 0.65)ε_c maximum allowable compression strain ($\varepsilon_c = 0.003\ mm/mm$ in the ASCE stanadards) and h_t is the overall thickness of the slab.

The ratio ρ_b defined in the above equation refers to the balanced cross section strain condition involving simultaneous achievement of the maximum concrete strain” ε_c” and full plastification of the steel sheeting . A slab with reinforcement ratio lower than ρ_b is under reinforced and the nominal moment resistance can be determined via a stress block analysis (below figure) then

$$M_{pl.cs} = A_p\, f_{y.p}(d_p - \frac{x_{pl}}{2}) \qquad (23)$$

x_{pl} −the depth of concrete stress block

$x_{pl} = \frac{A_p f_{y.p}}{0.85b\ f_c}$, b −unit slab width



Simplified calculation method for plastic analysis of the slab

The above two equations (X) and (Y) assume that the plastic neutral axis is in the slab and the whole steel deck yields in tension. Therefore it is not applicable to deep decks for which the plastic neutral axis would lie within the deck profiles and for decks made of steel grades with low ductility. The latter aspect is covered in the ASEC standards by imposing that the ratio $\frac{f_u}{f_y}$ for the steel deck shall not be lower than 1.08.

Determination of the flexural resistance of over reinforced slabs (for which $\rho > \rho_b$)requires use of general strain analysis in order to take account of all the various phenomena that possibly affect the slab performance . Besides including material nonlinearity, a refined analytical model should enable simulation of several events such as fracture of the deck in tension, buckling of deck parts in compression. Presence of additional reinforcing bars, crushing of concrete and interface slip between steel and concrete. Furthermore the influence of shoring on strains and stresses should also be accounted for

The value of strength reduction factor "Ø" for the different types of slab recognizes the characteristic of mode of failure "Ø" is assumed equal to 0.85 for under reinforced slabs and decreased to 0.7 for over reinforced slabs and 0.65 for under reinforced slabs for which $\frac{f_u}{f_y} \leq 1.08$ in consideration of the possibility of brittle failure. Recent research studies pointed out that several features are peculiar of the steel deck acting as "reinforcement" which cause composite slabs to perform differently from reinforced concrete members. In particular they are less sensitive to concrete failure.

These features are

1) The bending stiffness and strength of the steel deck which becomes significant for deep decks.
2) The yield strength of the sheeting usually substantially lower than that of the reinforcing bars.
3) The fact that the self-weight of the slab is resisted by the sheeting alone which is subject to important stresses before acting compositely with the concrete.

As a consequence traditional approaches based on the behavioral analogy with reinforced concrete members were found to be rather conservative, in particular with respect to the range of application of plastic analysis.
Amore general procedure for determining the ultimate flexure resistance of the slab was then proposed.

Equations (X) and (Y) implicitly assume that the shear bond is sufficient to cause the full flexural capacity of the composite slab to develop (i.e., the case of a fully composite slab) and that the Neutral axis lies in the concrete ($X_{pl} \leq h_c$). For usually deep decks the neutral axis lies within the steel section ($X_{pl} > h_c$). In this case a simplified approach can be used to compute$M_{pl,cs}$ which neglects the concrete in the rib. The tensile force in the sheeting can be decomposed in two forces 1) One at the bottom $N_{pl,pr}$ and 2) $N_{pl,pt} \cong compression\ force = 0.85\ f_c\ b\ h_c$

The contribution of forces $N_{pl,pc}$ defines a moment $M_{pl,pr}$ that may be considered as the plastic moment of the steel deck $M_{pl.p}$ reduced by the presence of the axial force$N_{pl,pt}$. The reduced plastic moment $M_{pl.pr}$ can be obtained from interaction diagram of the sheeting. A good approximation is provided by the following expression

$M_{pl.pr} = 1.25\ M_{pl.p} \left(1 - \frac{N_{pl.c}}{N_{pl.p}}\right) \leq M_{pl.p}$--------Z1

Where $M_{pl.p}$and $N_{pl.p}$ are the plastic moment and the full plastic axial resistance of the sheeting respectively

The plastic moment of resistance of the composite slab is then obtained as

$M_{pl.cs} = M_{pl.pr} + 0.85 f_c b h_c h^{\star} = M_{pl.pc} + N_{pl.c}\, h^{\star}$--------Z2

The lever arm $h^{\star}$can satisfactorily approximated by the relationship

$h^{\star} = h_t - 0.5h_c - e_p + [(e_p - e)\frac{N_{pl.c}}{N_{pl.p}}]$----------Z3

Where "e" and "e_p"are the distances from the bottom of the slab to the centroid and the plastic neutral axis of steel sheet respectively.

The extension of the method to the case of parallel composite slabs is straightforward. The compression force in the concrete"F_c" is lower than the value "F_{cf}" associated with the condition of full shear connection and two neutral axes are present in the cross section. The first lying in the concrete and the second within the steel sheeting

The depth of the concrete stress block is given by

$$X_{pl} = \frac{F_c}{0.85 f_c b} \le h_c$$

By replacing the equations Z1 to Z3, h_c with the X_{pl} and $N_{pl.pt}$ with F_c the moment of resistance can be computed as

$$h^{\star} = h_t - 0.5x_{pl} - e_p + [(e_p - e)\frac{F_c}{N_{pl.p}}]$$

$$M_{pl.pr} = 1.25 M_{pl.p}[1 - \frac{F_c}{N_{pl.p}}] \le M_{pl.p}$$

$$M_{w.cs} = M_{pl.pr} + F_c h^{\star}$$

4.1.3 Continuous composite slabs

Elastic analysis with limited redistribution and plastic analysis can both be adopted. In most instances the later approach is advantageous. However it imposes that rotation capacities are checked in the hogging movement regions which are substantially affected by the ductility of reinforcing bars. If high ductility rebar's are selected the method can be applied to commonly used slabs for spans up to 5m.

The hogging ultimate moment of resistance can be computed by the stress block theory, also accounting for the contribution of the sheeting when continuous over the support. The possible buckling of plate components should be considered. The restraint offered by the concrete allows for relation of the related rules. The Euro Code recommends that the effective widths be taken as twice the values given for class-1 steel webs. When the deck is not continuous over the support the slab in the hogging moment region should be modeled as a reinforced concrete element.

If elastic design analysis is adopted the large available sagging moment of resistance makes the modeling the slab as independent simple spans the most convenient approach. The elastic analysis with limited redistribution moments (up to 30% is allowed by the Euro Code) is less advantageous in terms of load carrying capacity on the other hand continuity may become beneficial to reduce deflections and meet serviceability requirements. When checking the shear bond resistance of the slab portions in sagging moment, an effective simple span equal to the distance between points of contra flexure may be assumed for internal spans, while for the end spans the full exterior span length has to be used. The regions in the hogging moment provide a constraint shear slippage which is modest for end spans and should be neglected.

4.1.3.1 Vertical shear

The resistance to vertical shear is mainly provided by the ribs and formulae for reinforced concrete T-beams can be applied , if suitably adjusted .Moreover the shear stresses in the sheeting consequent to it functioning as shuttering during concrete casting can be neglected and the total shear force can be considered as resisted by the composite cross section.

Reference can be made to a slab width equal to the distance between the centers of adjacent ribs and the un factored vertical shear expressed as (Euro Code-4)

$$V_{\vartheta} = b_r d_p \tau_u k_v (1.2 + 40\rho) \qquad (24)$$

Where

τ_u –The shear strength of concrete

b_r – Mean width of concrete rib

$$k_v = (1.6 - d_p) \geq 1 \; (with\; d_p\; in\; \mathrm{m})$$

$$\rho = \frac{A_p}{b_r d_p}$$

A_p –The effective area in tension within width b_o

The shear strength for Euro Code should be taken equal to $0.25 f_{ct}$

4.1.3.2 Punching shear and two way action

Heavy concreted loads may be applied to the slab (i.e, by the wheels of the fork lift trucks) which make the slab subject to two way action and may cause failure by punching shear.

The limited experimental knowledge available is not sufficient to allow for an appraisal of the sheeting contribution to the resistance to punching shear. This resistance is hence generally determined as for reinforced concrete sections. An effective area can be defined accounting for the different stiffness's of the slab in the two directions. The critical perimeter "C_p" can be obtained by a 45^0 dispersion of the load down to the centroidal axis of the sheeting in the longitudinal direction and the top of the sheeting in the transverse direction.

$$C_p = 2\pi h_c + 2(2d_p + a_p - 2h_c) + 2b_p + 8h_f \qquad (25)$$

Where $a_p and\; b_p$ define the loaded area and h_f is the height of the finishes.

In analogy with vertical shear and considering the height the deck $h_c$as the effective depth, the punching shear can be written as

$$V_p = C_p d_p \tau_u k_v (1.2 + 40\rho) \quad (26)$$

The load distribution requires the slab to possess adequate flexural strength in the transverse direction and suitable transverse reinforcement should be placed I consideration of negligible bending strength of the steel sheeting transverse to the deck ribs.

4.1.3.3. Serviceability limits state

The performance in service is verified mainly with reference to cracking of concrete and to the flexural stiffness (Through a limitation of the mid span deflections)

Cracking of concrete:

Cracking of concrete may occur in the regions over the supports where some degree of continuity develops due to intrinsic continuity of the concrete slab, also when the concrete slab is conceived and designed as a series of simply supported elements control of the crack width would require that the criteria are used which were developed and codified for reinforced concrete members when the environment is not aggressive and the width of the cracks not critical for the functioning of the structure , placement of nominal anti crack reinforcement would be sufficient to satisfy serviceability requirements . The Euro code specifies a minimum amount of reinforcement equal to 0.2% of the concrete area over the deck for unsupported slabs and 0.4% for the propped slabs.

Deflections

The floor deflection has to be limited to the values that ensue that no damage is induced by floor deformation in partitions and other non-structural elements. Values of maximum deflections are provided in codes which can usually be allowed in buildings. Below table presents the limit values given in the ASCE standards while the one's in Euro code-4 are given in below table limiting values for vertical deflections .Besides both codes provide limitations to the span to depth ratios(table below for , recommended Limiting values for Span – to -Depth) which are related though in a different way to in-service conditions. The ASCE span to depth ratios refer to the total depth"h_t" of the slab and intend to provide guidance to obtain satisfactory in service deflections, slab deformation should be computed and checked. On the contrary fulfillment of the Euro Code limitation which refers to the effective depth of the slab "d_p" allows for deflection calculations to be omitted at least when slip does not significantly affect the slab response. In some instances a more accurate assessment is necessary accounting for the expected type of behavior of the non-structural elements chosen in the specific design project.

Limiting values for vertical deflections, Euro code -4 and EBCS-4 :

Condition	Limits	
	δ_{max}	δ_1
Roofs generally	$L/_{200}$	$L/_{250}$
Roofs frequently carrying personnel other than for maintenance	$L/_{250}$	$L/_{300}$
Floors generally	$L/_{250}$	$L/_{300}$
Floors and roofs supporting brittle finish on non-flexible partitions	$L/_{250}$	$L/_{350}$
Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	$L/_{400}$	$L/_{500}$
Where δ_{max} can impair the appearance of the buildings	$L/_{250}$	

Recommended Limiting values for span to depth

Ratios	ASCE :$l/_{h_t}$	Euoroced-4/EBCS-4-$l/_{d_p}$
Simply supported slabs	22	25
External span of continuous slabs	27	32
Internal spans of continuous slabs	32	35

In the deflection calculation different components have to be taken into accounts that are associated with various facets of the response. In particular the deformation under short and long term loading and the effect of interface slip need to be considered. Immediate slab deformation "δ_{st}" can be determined via a linear elastic analysis. In continuous slabs calculations can assume that the slab has a uniform stiffness characterized by a moment of inertia equal to the average of those of the cracked and un cracked section.

Limiting values for vertical deflections recommended by the ASCE standards

The effect of the slip may be important in external spans and it should accounted for with reference to the results of the performance tests. It generally may be neglected when experimental data indicate that end slip greater than 0.5mm does not occur loads equal to 1.2 times the service loads. In this condition is not fulfilled there are two possible alternatives to the calculation of the deflection including slip.

1) Suitable end anchorages can be provided
2) The design service loads are reduced so that the previous limit on end slip is met.

Additional deflections under long term loading "δ_{lt}" may be approximated as for reinforced concrete members as a quota of the elastic deflection under short term loads.

i.e,$\delta_{lt} = k_{\delta}\delta_{st}$

The ASCE standards specify as k_{δ} as the same factor as in the ACI 318 code.

$$k_{\delta} = [2 - 1.2\left(\frac{A_{s.c}}{A_{s.t}}\right)] \leq 0.6 \qquad (27)$$

Where $A_{s.c}A_{s.t}$ the areas of steel in compression and tension under service loads respectively.

The area $A_{s.c}$ includes reinforcement and the possible portion of the steel deck in the compression.

The Euro code enables a simplified appraisal of the total deflection under sustained loads to be obtained for slabs with normal density concrete via a linear elastic analysis, performed with a slab stiffness based on an average modular ratio for long and short term effects. In the calculations of total deflections for serviceability checks, loads should be carefully selected. The immediate deformations induced by all the dead loads applied before placement of the relevant non-structural elements can be neglected. However the applied forces simulating the effect of shore removal, which contributes to long term deflections, should be accounted for as well. Finally in some instances the shear bond slip may also cause significant additional deflections under sustained loads. Test data are needed in these cases to provide approximate input to design calculations.

CHAPTER FIVE

COLUMNS

5.0 Introduction

The most common types of steel concrete composite columns are shown below

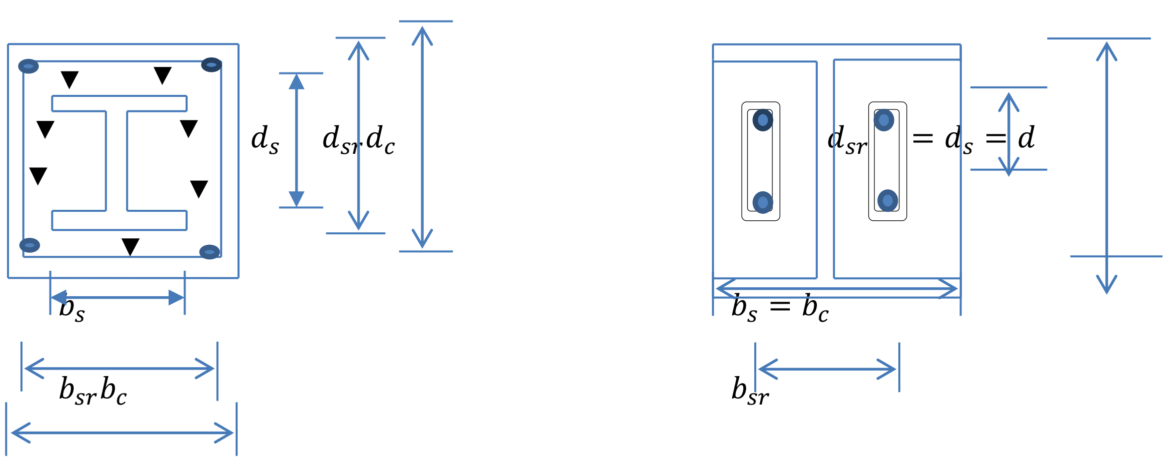
The cross sections are classified in 3-groups:

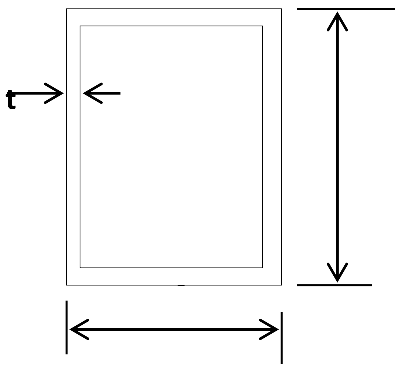
1) Fully encased
2) Partially encased
3) Concrete filled.

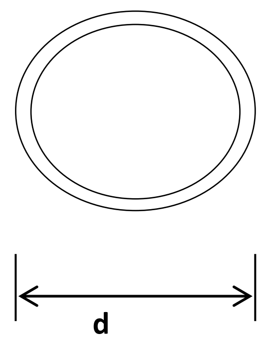
Fully encased: Steel profiles are fully encased in concrete and have good fire resistance owing to the protection offered by concrete

Partially encased: Steel profiles are partially encased in concrete while the external surface of the steel flanges is uncovered and good fire resistance owing to the protection offered by the concrete.

Concrete filled: The concrete completely fills a steel hollow section. In this case the column behavior is different when rectangular steel sections or circular steel sections are considered. The advantage of these types of columns are steel also serves as form work for concrete (Additives have to e used for reducing the concrete separation from the steel)







Hollow steel section
Filled of concrete

Circular steel section
Filled of concrete

Fig 5.0 various types of steel concrete composite columns

5.1 Composite columns have several advantageous

In general the two limit cases of a composite section are the reinforced concrete section when the steel area is small and the steel section when the concrete is not introduced. Thereby the composite system is a more complete system than simple reinforced concrete steel elements.

When adopting a composite section the amount of structural steel reinforcing steel and concrete area and the geometry as well as the position of three materials represent relevant design parameters. Indeed a number of different combinations is possible thus leading to flexible design.

Other advantageous are associated with constructional techniques. It is possible to set up entirely the steel part of the structure and then to complete it with concrete at alternate levels reducing erection time. It is also possible a convenient precast of partially reducing columns. In particular the steel profile can be filled with concrete in a horizontal position and then the column can be turned$180^0$and completed with the remaining concrete.

Structural Benefits: One important aspect is that concrete prevents local buckling more effectively in fully encased sections but also in partially encased ones. Also for concrete filled sections this problem is reduced. Indeed concrete represents an effective bound for steel in order to prevent or delay the critical warping there by the elements are generally characterized by a compact behavior while the section reaches full plastic state.

In the concrete filled type the steel provides benefits to concrete. In detail the confinement effect due to steel is high for the rectangular sections and very high for circular sections resulting in the increasing of the strength with a great enhancement ductility (The advantage appears to be more relevant to seismic countries in which composite columns are largely used)

Failure mechanism

Composite columns are characterized by several typical failure mechanisms. Collapse due to combined compression and bending could occur together with the phenomena that characterizes the behavior of slender beam columns (i.e, geometrical imperfections, erection imperfections and residual stresses). The shear interaction mechanism could also present, especially for stocky elements, local buckling is usually prevented. A problem relevant to composite systems is represented by the force transfer mechanism between the two components.

5.2 Elastic behavior of the section

All the sections reported in the above figure are characterized by the centroid of steel profile, reinforcement and concrete that are coincident owing to the symmetry about both axes, other cases are more complex.

Due to the aforementioned symmetry, the geometrical characteristics can be evaluated in simple manner. If concrete is cracked the overall full section must be considered and as a result the area A, and the moment of inertia “I” of the composite section can be evaluated as the sum of the area and the inertia of the two components by introducing the modular ration “n”

$A = A_s + A_{sr} + \frac{A_c}{n}$ (28)

$I = I_s + I_{sr} + \frac{I_c}{n}$ (29)

The long term effects can be taken into account by the "EM"(Effective Modulus)method. In detail the above equation can be adopted to introduce the creep effects caused by dead loads. Both the maximum and minimum stress in concrete, steel profile and steel rebar's can be combined by means of the following expressions

$$\sigma_{c,\max(\min)} = \frac{N}{nA} \pm \frac{M}{nI}\frac{d_c}{2}$$

$$\sigma_{s,\max(\min)} = \frac{N}{A} \pm \frac{M}{I}\frac{d_s}{2}$$

$$\sigma_{sr,\max(\min)} = \frac{N}{A} \pm \frac{M}{I}\frac{d_{sr}}{2}$$

Where M and N represent the design values. If the section is in cracked condition only the concrete in compression has to be considered and the approach is similar to the one used for reinforced concrete sections. The elastic neutral axis "X_e" can be evaluated by means of the following equation.

$I - S\left(\frac{M}{N} - \frac{d}{2} + X_e\right) = 0$ (30)

Where

S- The first moment of the cross section effective area (steel and concrete assumed to be under compression) with respect to the elastic neutral axis

I- The inertia of the effective section with respect to the same line

d- The overall dimension of the cross section.

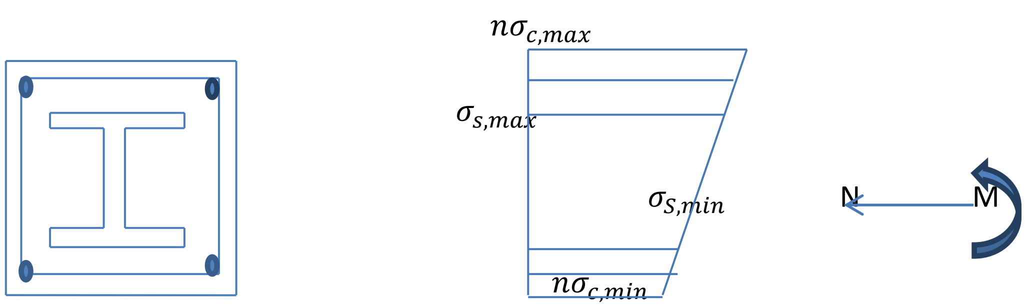
Finally concrete contributions are divide by the modular ratio"n". Moreover stresses can be computed by means of similar expressions that are typical of reinforced concrete sections.

$$\sigma_c = \frac{N}{n\,s} X_e$$

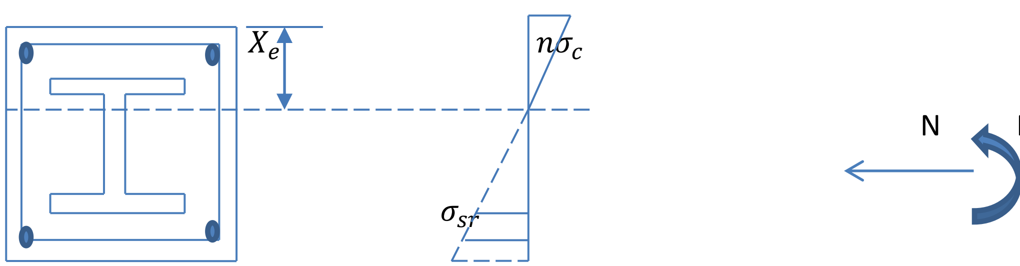
$$\sigma_s = \frac{N}{s} \left(\frac{d + d_s}{2} - X_e\right)$$

$$\sigma_{sr} = \frac{N}{s} \left(\frac{d + d_{sr}}{2} - X_e\right)$$

The stress control follows the same indications as the ones adopted for composite beams. Composite section fully effective



Composite section fully effective.



Cracked composite section

Fig 5.1 the plastic behavior of the section:

5.3 Resistance of the section under compression

First the uniaxial compression case is considered in order to highlight the main aspects of the problem. Experimental results showed that the resistance of the section can be evaluated as the sum of the strength of three components (i.e., concrete, structural steel and reinforcing steel). This approach is allowed if the maximum stress in concrete is reached when steel is yielded. Since the concrete strain at the maximum stress is about 0.2% and by assuming elastic modulus of steel is about 210000 N/mm2,maximum stress is reached simultaneously in the two materials, if the steel stress at yielding is lower than $\frac{2}{1000}210000 = 420\frac{N}{mm2}$, on the basis of aforementioned consideration AISC provisions specify maximum yield stress of 55Ksi(about 380 N/mm^2. Therefore high strength steel is excluded as steel yield strain could be higher than the peak strain of the concrete. As a result the yield stress of steel could be reached when concrete behaves in the softening range so as to get section resistance lower than sum of resistance of the two components. However the sum of two resistances can still be obtained only if concrete is well confined in such conditions concrete is ductile and stress remains practically constant even for high strain values.

According to the Euro code-4 provisions the “design” plastic axial resistance of the section N_{pl} is evaluated by dividing the “characteristic “strength of material f_{ys} ,f_{ysr} $and f_c$ by means of the partial safety factors as follows.

Encased sections

$$N_{pl} = \frac{f_{ys}A_s}{1.1.0} + \frac{f_{ysr}\,A_{sr}}{1.15} + \frac{0.85 f_c A_c}{1.50} \qquad (31)$$

Rectangular concrete filled sections

$$N_{pl} = \frac{f_{ys}A_s}{1.1.0} + \frac{f_{ysr}\,A_{sr}}{1.15} + \frac{f_c A_c}{1.50} \qquad (32)$$

Additional corrective factors are introduced for circular concrete filled sections to take into account the confinement action, which is both very effective and beneficial in concrete and reduces the normal bearing capacity of steel that is subject to biaxial tension and compression.

According to AISC provisions the resistance of the section is computed as the product of the nominal strength of the materials $f_{ys},f_{ysr},$ $and\ f_c$times the resistance factor $\emptyset_c = 0.85.$ As a result$N_{pl} = 0.85(\,f_{ys}A_s +\ c_1 f_{ysr}A_{sr} + c_2 f_c A_c)$. In which the strength of materials has to be considered as a “nominal value “while the numerical factors "c_i" assume the following values.

Encased sections $c_1 = 0.70 \ \ and \ \ c_2 = 0.60$

Concrete filled sections, $c_1 = 1.00 \ \ and \ c_2 = 0.85$

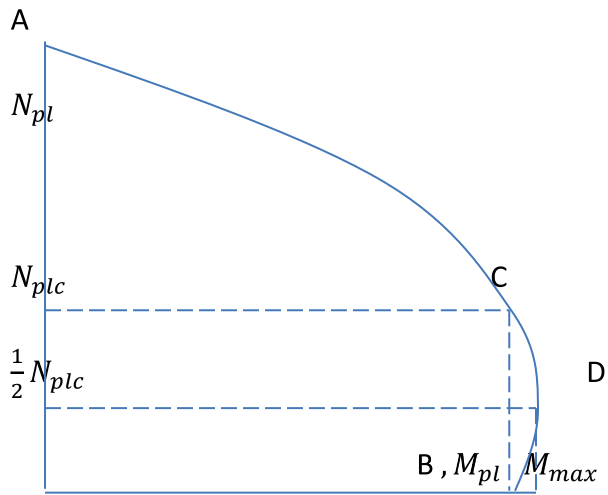
Resistance of a section to combined compression and bending

The behavior of the composite cross section under compression and bending actions is determined by the interaction curves M-N. The relevant codes provide methods quite different for defining the interaction curves. For example the Japanese provisions based on the method of (wakabaiashi) are reported in (AIJ 1987) and the AISC provisions are reported in (AISC 1994). The Euro Code approach is based on the study of Roik and Bergman.

The general procedure consists of defining some points of the interaction curve by means of the solution of both translational and rotational equilibrium equations for the section. These are based on the Bernoulli's hypothesis and by introducing the constitutive relationships of materials. The method is theoretically simple but it requires considerable effort and it is not used in practice . Thereby simplified methods are required.

The ductility of the materials allows a full plastic analysis to be used with reference to the interaction curve of the below figure. Point "A" defines the uniaxial plastic resistance ($N = N_{pl,} \, M = 0$) and it can be determined by means of the formulation reported in the previous paragraph. Point "B" represents the plastic moment resistance ($M = M_{pl,} N = 0$)

The studies of Roik and Bergman suggest a piecewise linear curve on the safe side with respect to the actual interaction curve to be drawn. The minimum number of points necessary to assuming a full plastic stress distribution(i.e, stress block) in all the materials(concrete , structural steel and reinforcing steel.). The result of the method is schematically shown in below figure. It can be observed that Point "C" which can be readily defined without any additional evaluation is determined by the same moment M_{pl} that characterizes the condition N=0

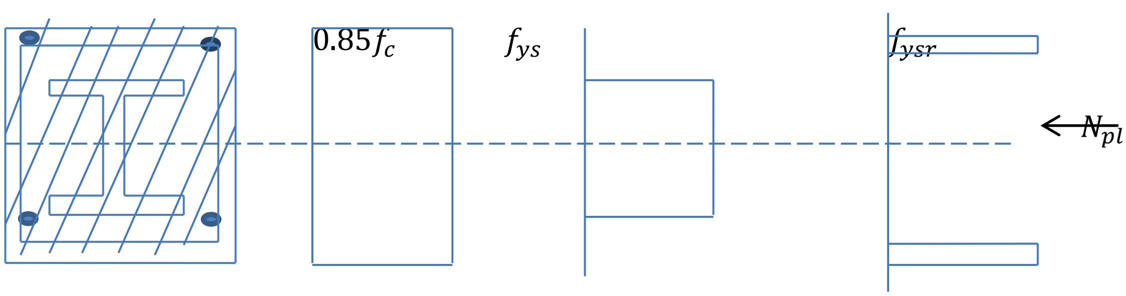


M-N interaction curve

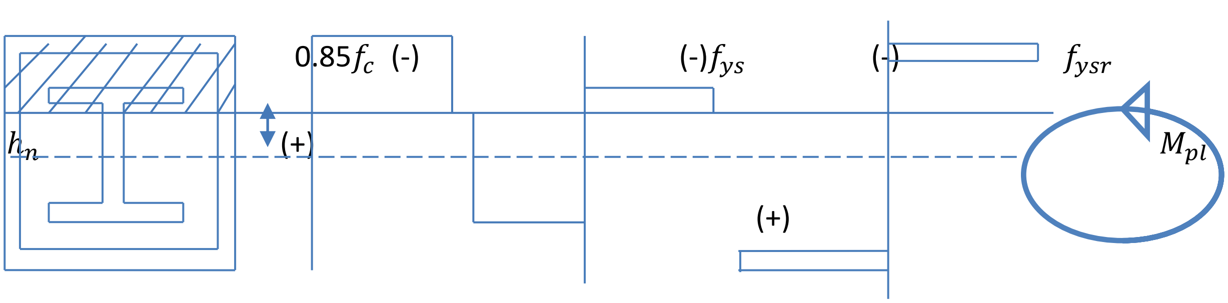
Fig 5.2 M-N interaction curve

Below figure shows the stress distributions corresponding to an interaction curve.

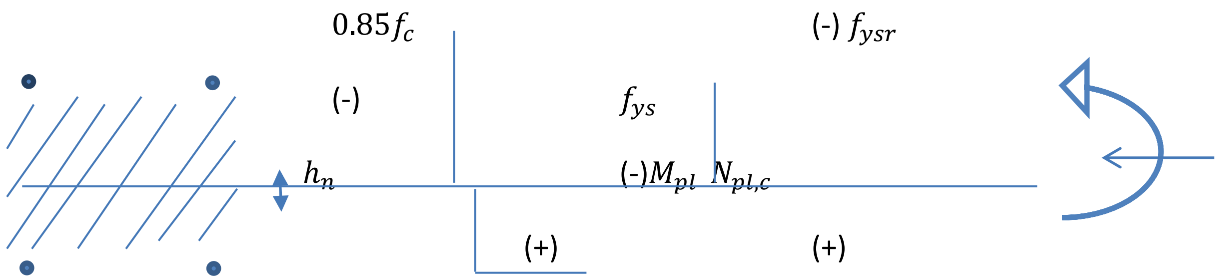
Point A:



Point B:



Point c:



Point D:

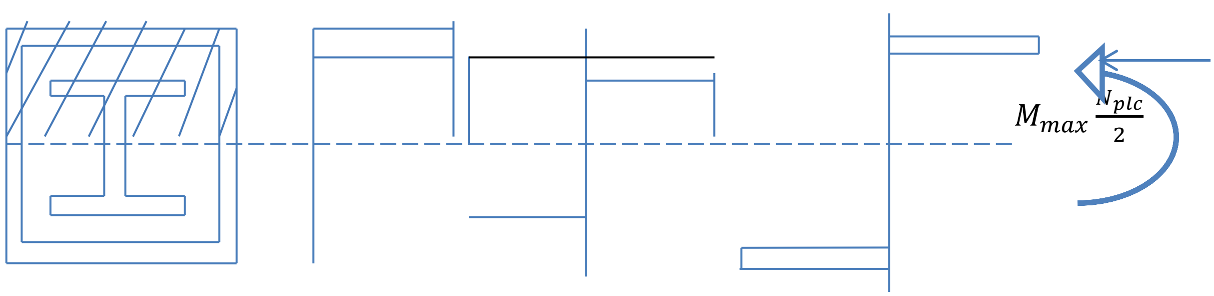


Fig 5.3 Stress distribution corresponding to M-N interaction curve

This consideration allows $N_{pl,c}$ to be easily identified as the resistance of the concrete area by comparing the stress pattern to the one of point "B":

$$N_{pl,c} = \frac{c_3 f_c A_c}{1.5} \quad (33)$$

Where $c_3 = 1.0$ for filled and c_3=0.85 for encased sections respectively.

Point "D" is identified through the co-ordinates (N $=\frac{N_{plc}}{2}$ $and\ M = M_{max}$). Point "D"can be safely neglected leading to slightly conservative interaction curve. In general the interaction curve A-C-D is convex and as result on the safe aide.

In order to evaluate M_{max} and M_{pl} by means of the stress pattern of the figures above showed (point "A" "B" , "C" and Point "D") the following equations can be identified.

$M_{max} = w_s f_{ys} + w_{sr} f_{ysr} + \frac{w_c}{2}\, 0.85\, f_c$ (34)

$M_{pl} = (w_s - w_{s.n}) f_{y.s} + (w_{sr} - w_{sr.n}) f_{ysr} + \frac{1}{2}(w_c - w_{c.n}) 0.85 f_c$ (35)

$w_c, w_{sr}, w_s -$ The plastic moduli of the concrete, reinforcement and steel profile

$w_{c.n}, w_{sr.n}, w_{s.n} -$ The plastic moduli of the part of the concrete, reinforcement and steel profile in the height of the section $\pm h_n$, where $h_n -$ is the distance between the plastic neutral axis line and the centroid line in correspondence of the $M = M_{pl}$

In the above two equations the strength of materials should be considered as "Un factored "strength. In order to obtain the design values of Euro Code-4 it is necessary to introduce the partial safety factors. Clearly the interaction curve characterizes the sole section behavior or the behavior of the stocky element.

The behavior of the members

5.4 Resistance of the members to compression

As far as buckling problems are concerned both AISC and Euro Code-4 extend the approach of the steel columns to the composite ones:
According to Euro Code-4 the design ultimate bearing capacity of the composite column "N_u"has to be determined by considering imperfection effects and residual stresses. The influence of these effects on the axial resistance of the section N_{pl} evaluated by means of below equations are introduced by means of the factor χ of the buckling curves in order to evaluate the ultimate axial load of the column N_u.

Encased sections:

$N_{pl} = \frac{f_{ys} A_s}{1.1.0} + \frac{f_{ysr}\, A_{sr}}{1.15} + \frac{0.85 f_c A_c}{1.50}$----------- (A)

Rectangular concrete filled sections:

$N_{pl} = \frac{f_{ys} A_s}{1.1.0} + \frac{f_{ysr}\, A_{sr}}{1.15} + \frac{f_c A_c}{1.50}$------------ (B)

$$N_u = \chi\, N_{pl}$$

$$\chi = \frac{1}{\phi + \sqrt{\emptyset^2 - \lambda^2}}$$

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$$

Where $\lambda -$ the relative slenderness of the column that shall be defined the following

$\alpha -$ The imperfection factor that is equal to 0.21, 0.34 and 0.49 for three different buckling curves named a, b, c respectively. In particular these three buckling curves a, b and c refers to concrete filled cross section's , encased cross section's loaded along the strong axis and encased sections loaded along weak axis respectively.

Euro code- 4 defines the relative slenderness by means of the following expression

$$\lambda = \sqrt{\frac{N_{pl}}{N_{cr}}} = \frac{kl}{\pi}\sqrt{\frac{f_{ys}A_s+f_{ysr}A_{sr}+0.85f_cA_c}{E_sI_s+E_{sr}I_{sr}+\frac{0.85E_cI_c}{1.35}}} \qquad (36)$$

Since N_{pl} is expressed by means of equation (A) and (B) by assuming characteristic values of the strength without the partial safety factors.

$$N_{pl} = f_{ys}A_s + f_{ysr}A_{sr} + c_3 f_c A_c \qquad (37)$$

$$c_3 = 1.0 \; for \; filled \; and \; c_3 = 0.85 \; encased.$$

The critical bearing capacity N_{cr} is calculated as follows

$$N_{cr} = \frac{\pi^2 (EI)_e}{(kl)^2} \qquad (38)$$

Where $kl -$ the effective length.

$(EI)_e -$ The effective flexural stiffness of the composite section

$(EI)_e = E_sI_s + E_{sr}I_{sr} + 0.59E_cI_c$ The factor $\frac{0.8}{1.35} = 0.59$ that reduces the elastic modulus owing to the non linear behavior of concrete and it allows a good agreement of the theoretical and experimental results to be achieved. The creep influence can be remarkable for slender columns therefore in such a case an additional reduction of the elasticity modulus of concrete that leads to an effective elasticity modulus.

$$E_{cef} = E_c(1 - 0.5\frac{N_p}{N}) \qquad (39)$$

Where the ratio between the axial loads owing to the dead load N_p and the total axial load “N” is considered to weight the creep effect. As an example ifthe dead load is $\frac{2}{3}$ and the live load is $\frac{1}{3}$ of the total axial load the effective modulus of concrete is obtained by a reduction factor equal to$\left(1 - 0.5\frac{2}{3}\right) = 0.67.$

Resistance of members to combined compression and bending:

The check of column subject to combined compression and bending has to be carried out by means of the following steps.

1) The interaction curve of the section has to be evaluated
2) The interaction curve of the member shall be drawn reducing the interaction curve of the section to take into account the geometrical non linearity related both to the axial compression (axial buckling) and to bending (flexural torsional buckling). Moreover it has to be considered the influence of the geometrical imperfections (by the fabrication of the elements as well as the erection procedure) mechanical imperfections (residual stresses) Bending moment pattern along the element, the presence of lateral restraints.

   The external actions have to be increased in a simplified way (in particular the B.M owing to the load has to be increased) to evaluate the stress introduced by the geometrical non- linearity that can influence the slender elements.

   By considering the procedure of Euro Code-4 one assumes that no B.M “M” can be applied in correspondence of $N_u = \chi N_{pl}$. However when the axial compression is null the moment M_{pl} can be applied entirely. As a result for a generic value of “N” Euro code-4 suggests referral to the line between these two limiting values. For the design value of the axial force "N_d" the plastic moment M_{pl}reported in below figure can be carried by the column. Moreover what follows can be observed

   The use of stress blocks for the materials is unsafe to a certain extent because it implies infinite ductility of concrete. As a result Euro Code-4 suggests reduction of M_{pl} by a factor of 0.9

   The geometrical imperfections of the column which are taken into account in the buckling curve are considered in most unfavorable combination: Constant along al the element thus the method previously explained is referred to as a constant distribution of the B.M. If the moment is variable along the column the procedure is much safer. It is possible to consider the interaction curved marked by the dotted line in below figure. That intersects the points M=0, N=$\chi_n N_{pl}$ where

$$\chi_n = \chi\frac{1-r}{4}$$

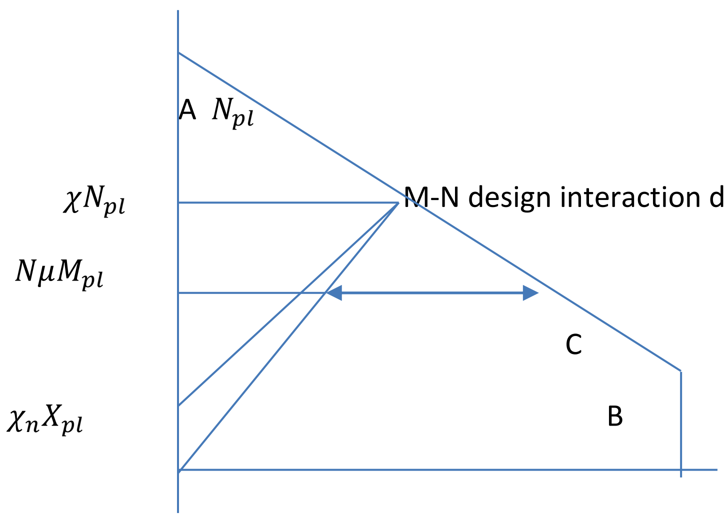


Fig 5.4 M-N Design interaction Diagram

Where "r" is limited between "-1" and "1" and represents the ratio between the values of bending moments at the member ends. The design bending moment "M" that has to be considered in the check is an equivalent moment owing to the variability along element. A number of studies dealt with this aspect, Euro code-4 take into account the second order moment multiplying the first order bending moment by means of a factor "a"

$$a = \frac{c}{1-\frac{N}{N_{cr}}} \geq 1.0$$

C= 0.66+0.44, $r \geq 0.44$

In the factor "a" the term above takes into account the moment distribution long the column. In detail "C" equal to "1" if the moment is constant the term below is lower than "1" and increase the equivalent moment introducing the effect of the second order moment(this term approaches “1” if “N” is much lower than N_{cr} while it approaches “0” if “N” tends to “N_{cr}"). Finally the check of the column requires that the moment “M” is lower than the ultimate value $M_u = 0.9\mu M_{pl}$

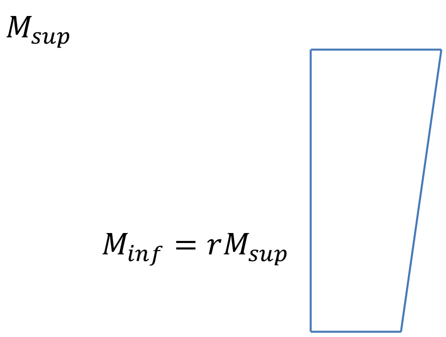


Fig 5.5 Bending moments at the member ends

Definition of “r” factor in accordance with Euro Code-4

The aforementioned numerical procedure is developed by means of the examples to follow regarding fully encased columns, partially encased columns and concrete filled columns

5.5. Influence of local buckling

The elastic approach has to be applied to evaluate the stress condition under the serviceability loading. Conversely the full plastic bearing capacity of the section can be reached almost always at failure. In fact as mentioned previously the presence of the concrete reduces the local buckling phenomena, in particular in fully encased sections local buckling is definitely prevent if a certain minimum concrete cover is used. In the other cases local buckling can be excluded if limit ratios of the depth to thickness for the steel section are respected.

By considering Euro Code-4 it follows that

Partially encased”I” sections

$$\frac{b_s}{t_f} \leq 44\sqrt{\frac{235}{f_{ys}}}$$

Circular hollow steel sections:

$$\frac{d_s}{t} \leq 90\sqrt{\frac{235}{f_{ys}}}$$

Rectangular hollow steel sections

$$\frac{b_s}{t} \leq 52\sqrt{\frac{235}{f_{ys}}}$$

Where"f_{ys}" is the characteristic strength in N/mm^2 . These restrictions are loose with respect to the case of simple profiles.

Shear effects

In stocky members or in the case of high horizontal loads the shear influence is remarkable .In this case the interaction curve N-M-V has to be drawn both for the section and for the member. To analyze this problem in a general manner the constitutive relationships of materials and appropriate resistance criteria are required. Moreover all the geometrical and mechanical non linearity's and imperfections of the element have to be considered in the computation.

Usually the provisions suggested for the continuous beam where interaction between shear and bending takes place apply to columns. In detail the approximate procedure assumes that shear is carried out by means of the web of the steel profile and only a part of the web is considered able to resist the combined compression and bending.

5.6 Restrictions for the application of the design methods

The rules explained in the previous paragraphs according to both Euro Code-4 and AISC are effective if some restrictions are fulfilled. Indeed the simplified procedure is based on large experimental and numerical analysis that, however cannot take into account all design conditions. Thereby though it is always possible to apply a general procedure introducing constitutive relationships of materials the rules suggested by Euro Code-4 can be used only if the following restrictions are satisfied.

1) The cross section are symmetric about the two axes and the cross section is constant along the member
2) The factor$\delta_s = \frac{A_s f_{ys}/_{1.1}}{N_{pl}}$

   That represents the contribution of the structural steel in the plastic axial load capacity varies between 0.2 and 0.9 otherwise the member has to designed as a reinforced concrete element corresponding to a lower restriction or as a steel element corresponding to a higher restriction.

3) The relative slenderness λ shall be lower than “2”
4) If the longitudinal reinforcement is considered in the design the minimum share of 4% of the concrete area shall be provided
5) In the fully encased cross section a minimum concrete clear cover of 40mmshall be provided.

CHAPTER SIX

SHEAR CONNECTION

6.0 Introduction

The mutual transfer of forces between the steel and concrete components is the key mechanism, which makes composite action possible. The mechanism generally involves a complex combination of forces (or stresses) acting at the steel concrete interface. I design the main attention is focused on the forces (or stresses) parallel to interface i.e., on the longitudinal shear forces (and stresses). The components of the interface forces perpendicular to the interface, which may play a significant role in the transfer mechanism, are principally considered through the selection of suitable detailing.

It can be stated that the sear connection is a factor of substantial importance and , in many instances, it permits achievement of the required performance. Therefore a substantial research effort has been devoted to the development of the fundamental knowledge of the response and performance of the different shear connections available or proposed for practical use.

Preliminary to any treatment of the behavioral features and design criteria of the shear connection, it seems convenient to give some useful definitions and classifications based on the key behavioral parameters of stiffness, strength and ductility.

1) Stiffness: A shear connection realizes either full interaction (the connection is “rigid” and no slip occurs under stress at the steel concrete interface) or partial interaction(the connection is flexible and interface slip occurs)
2) Resistance: When the overall resistance of the connection can be conveniently considered as in plastic design, a full connection has the shear strength sufficient to make the composite structural element (beam or slab) to develop its ultimate flexural resistance before collapse is achieved. If this condition is not fulfilled the connection is partial connection. A structural element with full shear connection is a fully composite structural element. A structural element with partial shear connection is a partially composite structural element. The ratio $\frac{F_c}{F_{c.f}}$ between the resistance of the shear connection and the minimum resistance required by the full connection condition defines the degree of shear connection.
3) Finally, a connection is ductile if its deformation (slip) capacity is adequate for a complete redistribution of the forces acting on the individual connectors. The ductility demand depends on the span and the degree of the shear connection.

The behavioral parameters relevant to the type of analysis adopted in design (i.e, elastic, inelastic or plastic analysis) have to be considered. In particular connection flexibility should be accounted for in elastic and plastic analysis, which would make design rather complex. However the simplified assumption of full interaction is satisfactory for most shear connections used in practice: the effect of slip is in fact negligible.

6.1 The shear transfer mechanisms

Various forms of shear transfer can be identified for nature and effectiveness, namely:

1) Adhesion and chemical bond
2) Interface friction
3) Mechanical interlock
4) Dowel action

6.1.1 Adhesion and chemical bond

Shear transfer via adhesion and bond has the non-negligible advantage of being associated with no steel –concrete slip. However, tests show rather low maximum shear resistance, which decreases rapidly and remarkably, in the post ultimate range of response. Moreover this form of the shear strength is highly dependent on factors such as the quality of the steel surface and the concrete shrinkage, the control and quantification of which is difficult. Therefore, low values have to be assumed in design for bond strength. Nevertheless, bond might be sufficient when the demand of interface shear capacity is limited s in composite columns or in fully encase beams at least in the elastic range.

Euro Code-4 Specifies the following values of the bond stress (including the effect of friction) to be considered when checking the connection effectiveness of composite columns:

Completely encased sections	0.6 N/mm^2
Concrete filled hollow sections	0.4 N/mm^2
Flanges in partially encased sections	0.2 N/mm^2
Webs in partially encased sections	Zero

6.1.2 Interface Friction

Friction is often associated to bond in resisting shear. In flexural members the tendency of the steel and concrete elements to separate usually makes friction action rapidly deteriorate. A suitable geometry of the composite element, as in the composite slab with “dove-tailed” profiled sheeting prevents separation and allows friction interlock to develop throughout the response.

6.1.3 Mechanical interlock

It is obtained by embossing the metal decking so that slip at the interface is resisted by bearing between the steel ribs and the concrete indentations. The effectiveness of the embossments depends on their geometrical dimensions (mainly the height and depth) and shape. Enhancement of the shear transfer capacity in the composite slabs is achieved if frictional and mechanical interlock are combined. The complexity of these interlock shear transfer mechanisms dictates that the response of the shear connection is determined by the appropriate tests. Mentioned in the codes.

The transfer of large shear forces dictates that suitable mechanical connectors are used. New types of connectors have been continuously developing since the early stages of composite construction. Therefore an increasing variety of forms of shear connectors is available for practical use. Despite possible significant differences they all act as steel dowels embedded in concrete and hence apply a concentrated load to the concrete slab, the diffusion of which requires careful consideration in the design of the slab detailing. The headed stud is by far the most popular connector.

The behavior and modes of failure of each type of connector highly dependent on the local interaction with the concrete, and can only be determined via ad-hoc tests: the so called push-out tests. All types of mechanical shear connectors possess a limited deformation capacity. However in many instances, the associated slip is sufficient to make the design flexural resistance and rotation capacity of the composite section to be developed. If this condition is fulfilled, the connectors (and the connection) can be classified as ductile. As mentioned in the introduction to this section, the ductility requirements depend on the span and the degree of the shear connection: the classification of a connector as ductile should hence be associated with a definition of a combination of:

1) A range for these parameters
2) A characteristic value of slip capacity

Euro code-4 assumes a characteristic slip capacity of 6mm as the reference parameter in the calibration of the recommendations related to partial composite beams with ductile connectors. On the basis of an assessment of the available experimental data it then classifies as ductile for the given ranges of span and the degree of the shear connection, only the friction grip bolts and the welded stud connectors meeting the following requirements:

- Overall length H_{sc} after welding not less than 4 times the diameter d_{sc}
- $16mm \leq d_{sc} \leq 22mm$

Despite the assumed limitations appearing to the rather strict, most of the shear connections in buildings fall into the ductile category. Therefore the current practice should not be largely affected. Stud shear connectors may also be used to provide end anchorage in composite slabs: their effect is significant on both the resistance and the ductility of the shear connection as a whole.

6.2 Studs connectors used with profiled steel decking

The most popular solution for floor systems in composite framed construction makes use of decks where the profiled steel sheeting acts compositely with a concrete "ribbed" slab. The studs are placed within a rib

The prime parameters affecting the stud behavior are:

1) The orientation of the ribs relative to the beam span
2) The rib geometry as characterized by the $\frac{b_r}{h_r}$ ratio
3) The stud height H_{sc} relative to the rib height h_r

Proposals have been made to account for the influence of the relevant parameters on stud ultimate resistance. However the available data do not enable a comprehensive design method to be developed. In codes the effects of the main factors are accounted for via a suitable reduction factor.

6.2.1 Deck ribs oriented parallel to steel beams

The shear resistance q_u of an individual stud in solid slab is lesser of

$$q_{u.c} = k_c A_{sc} (f_c E_c)^{0.5}$$

$$q_{u.s} = k_s A_{sc} f_{u.sc}$$

Where A_{sc} −represents the cross sectional area of a stud shear connector. The Euro code-4 specifies $k_c = 0.36$, $k_s = 0.8$, more over the Euro code limits to 500 N/mm^2 the value of the tensile strength "$f_{u.sc}$" and it restricts the application of the resistance equations(the above two equations) to studs with diameter not greater than 22mm(7/8 in).

Experimental results had proven that the height to diameter ratio for the stud ($\frac{H_{sc}}{d_{sc}}$) affects the resistance$q_{u.c}$: the full resistance is developed only ($\frac{H_{sc}}{d_{sc}}$) ≥ 4. The European code aims at permitting use of a wider range of studs with (studs with lower height may be conveniently used in shallow floor systems). Therefore it specifies a reduction coefficient of the resistance $q_{u.c}$ expressed as $\alpha = 0.2[\left(\frac{H_{sc}}{d_{sc}}\right) + 1] \leq 1$. In any case studs with $\frac{H_{sc}}{d_{sc}} < 3$ cannot be used.

Studies conducted at Lehigh University in the 1970s provide the sole background to this problem. The results indicated that the resistance of the stud in a rib parallel to the supporting beam can be determined by reducing resistance in a solid slab equations (above two $q_{u.c}$,$q_{u.s}$) by the factor

$$k_{rp} = 0.6\frac{b_r}{h_r}[\frac{H_{sc}}{h_r} - 1.0] \leq 1.0$$

Which mainly accounts for the limited restraint provided to the concrete by the sheeting side walls? The restraint is even negligible when the deck is split longitudinally at the beam. Good practice in this case would suggest meeting the requirements set for concrete haunches. Euro code limits $H_{sc} < h_r + 75\ mm$

6.2.2Deck ribs oriented perpendicular to steel beams

The efficiency of the floor system may require the steel sheeting to be placed with the ribs transverse to the supporting beam. This deck arrangement apparently involves a concrete rib that is significantly stressed, as it acts the transfer medium of the longitudinal shear between the concrete slab above the sheeting and the base of the stud. Moreover the stud connector subject to a highly eccentric load tends to be less effective than in solid slabs. Its performance in terms of the strength and ductility may be adversely affected by the interaction with other connectors in the same rib(i.e, when the number of studs in a rib ,$N_{r,}$increases) and/or the reduced efficiency of longitudinal restraint offered by the concrete(i.e, when the studs are placed off center)

A comprehensive investigation carried out at Lehigh university which also accounted for the results of the previous studies permitted definition of the general form of the relationship between the main parameters governing the shear connection performance and the reduction factor k_{rt} to be applied to stud resistance in solid concrete slabs:

$$k_{rt} = \frac{c}{\sqrt{N_r}}\frac{b_r}{h_r}[\frac{H_{sc}}{h_r} - 1.0] \leq 1.0$$

Therefore Euro Code-4 assumed c=0.7 and limited the application of the above equation

1) Stud diameter $d_{sc} \leq 20mm$
2) Studs welded through deck
3) Studs with ultimate tensile strength not greater than 450 n/mm2
4) Ribs with $\frac{b_r}{h_r} \geq 1\ and\ h_r \leq 85\ mm$
5) $k_{rt} \leq 0.8\ when\ N_r \geq 2$

The limited knowledge still available does not allow coverage of the practice common in a few countries of welding studs through holes cut in the sheeting. Neither code takes into account the effect of off center placement of the studs, resulting from the presence of stiffening ribs in the selecting .This effect may be significant. In the case of symmetrically loaded simply supported beams the “strong” side is the one nearest to the closest support.

6.3Other types of connectors

The knowledge of the performance of other types of connectors is far more restricted and it is mainly based on research work conducted in the 1970s. This reflects the rather limited use of connectors other than studs both in building and bridge practice. Some new types of shear connector such as the cold formed” seat element” connected to the steel beam by means of shot fried pins are increasingly employed. Their response was found satisfactory in a few studies of composite beams and joints and comparable to equivalent welded studs. However no specific rules are currently provided in codes and their use requires suitable testing to be carried out.

6.3.1 Channel connectors

The Euro code does not cover channel connectors while channel connectors are the only connectors other than studs included in the AISC specifications, which are based on the work by slutter and driscoll. The strength equation proposed there is modified in order to cover the case of light weight concrete. Their capacity is then determined as

$$q_u = 0.3(t_f + 0.5t_w)b_c\sqrt{f_c E_c}$$

Where $t_f, t_w \ and \ b_c$ are the flange thickness, the web thickness and the length of the channel respectively.

6.3.2 Angle connectors

Angle connectors were investigated mainly in France and the Euro code design formula is based on the French studies.

$$q_u = 10b_a {h_{ac}}^{3/4} {f_c}^{2/3}$$

Where b_a, h_{ac} are the length and the height of the outstanding leg of the connector. The design resistance $q_{u.d}$ is then obtained by applying to q_u a partial safety factor equal to 1.25.

CHAPTER SEVEN

7.1 MODELING, ANALYSIS AND DESIGN OF B+G+7 RCC BUILDING

Introduction

This design report explains the assumptions, the analysis, the design process and the results associated with the structural design of B+G+7 Building.

Purpose –Condominium Building

Approach – Serviceability limit states

Design specifications and coefficients

(A) Concrete

- Unit weight of concrete $\gamma = 24\ KN/m^3$ EBCS 1 table 2.1

- Partial safety factors, $\gamma c = 1.5$ for class I work EBCS 2 table 3.1

-Characteristic compressive strength

C-25: $fcu = 25Mpa$

$fck = fcu/1.25 = 25/1.25 = 20$ Mpa EBCS 2 table 2.3

-Characteristic tensile strength

$fctk = 0.21 \times fck^{2/3} = 0.21 \times 20^{2/3} = 1.55$ Mpa EBCS 2 table 2.4

Design strength

I. $fcd = 0.85*fck/\gamma c = 11.33MPa$ (In compression) EBCS 2 eq(3.4)
II. $fctd = fctk/\gamma c = 1.032MPa$ (In tension) EBCS 2 eq(3.5)

Modulus of elasticity:

$Ec = 9.5(fck + 8)^{1/3} = 9.5 * (20 + 8)^{1/3}$ EBCS 2 eq(2.3)

(B) Steel

-Steel grade = S-400

-Partial safety factors, γs = 1.15 for class I work EBCS 2 table 3.1

-Yield strength, fyk = 400 Mpa

Design strength

$fyd = fyk/ \gamma s$ = 400/1.15 = 347.83 Mpa EBCS 2 eq(3.6)

Design loads

Fd= γf *Fk EBCS 1 eq(1.1)

Where, Fk = characteristics loads

γf = partial safety factor for loads

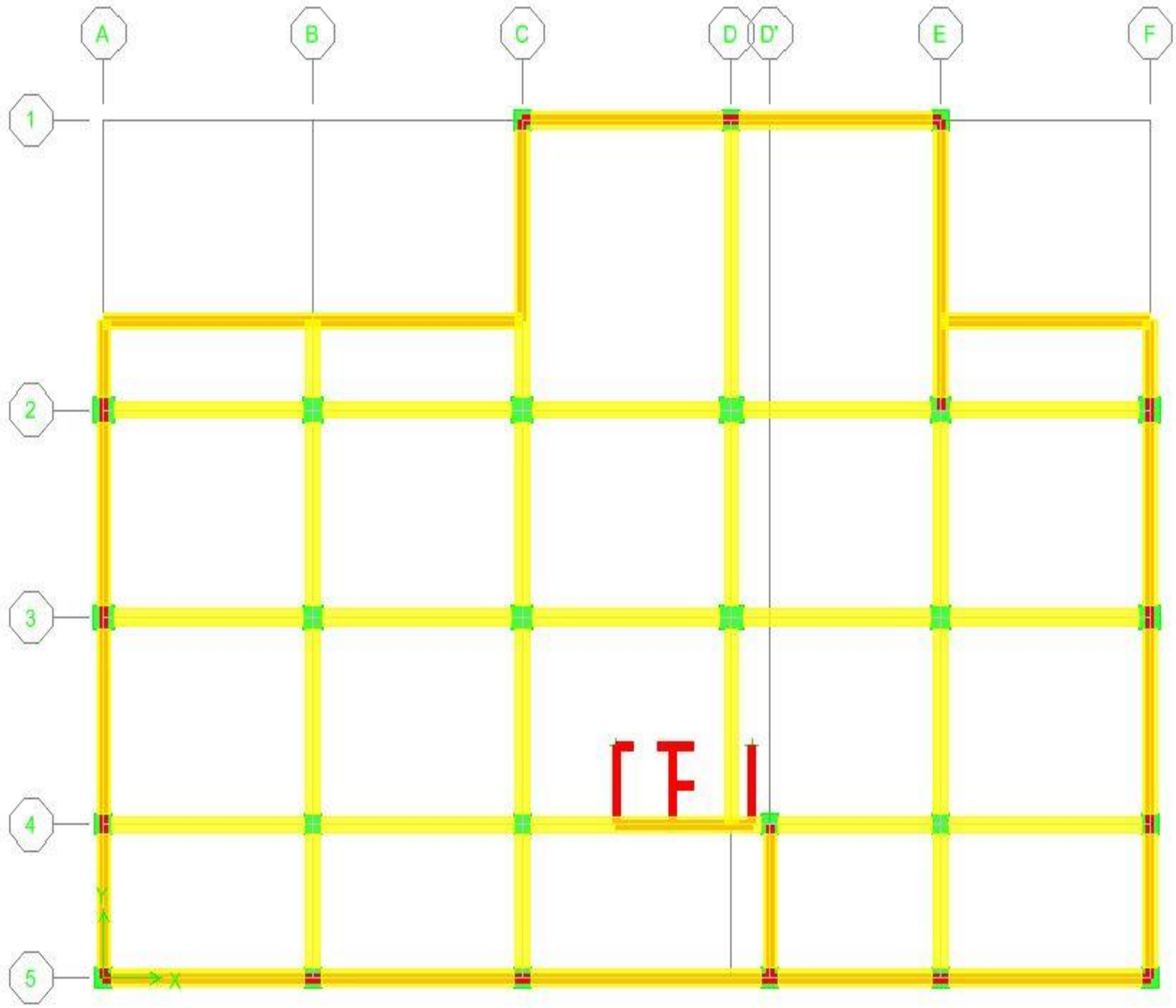
= 1.3 for dead loads EBCS 1 table 1.2

= 1.6 for live loads

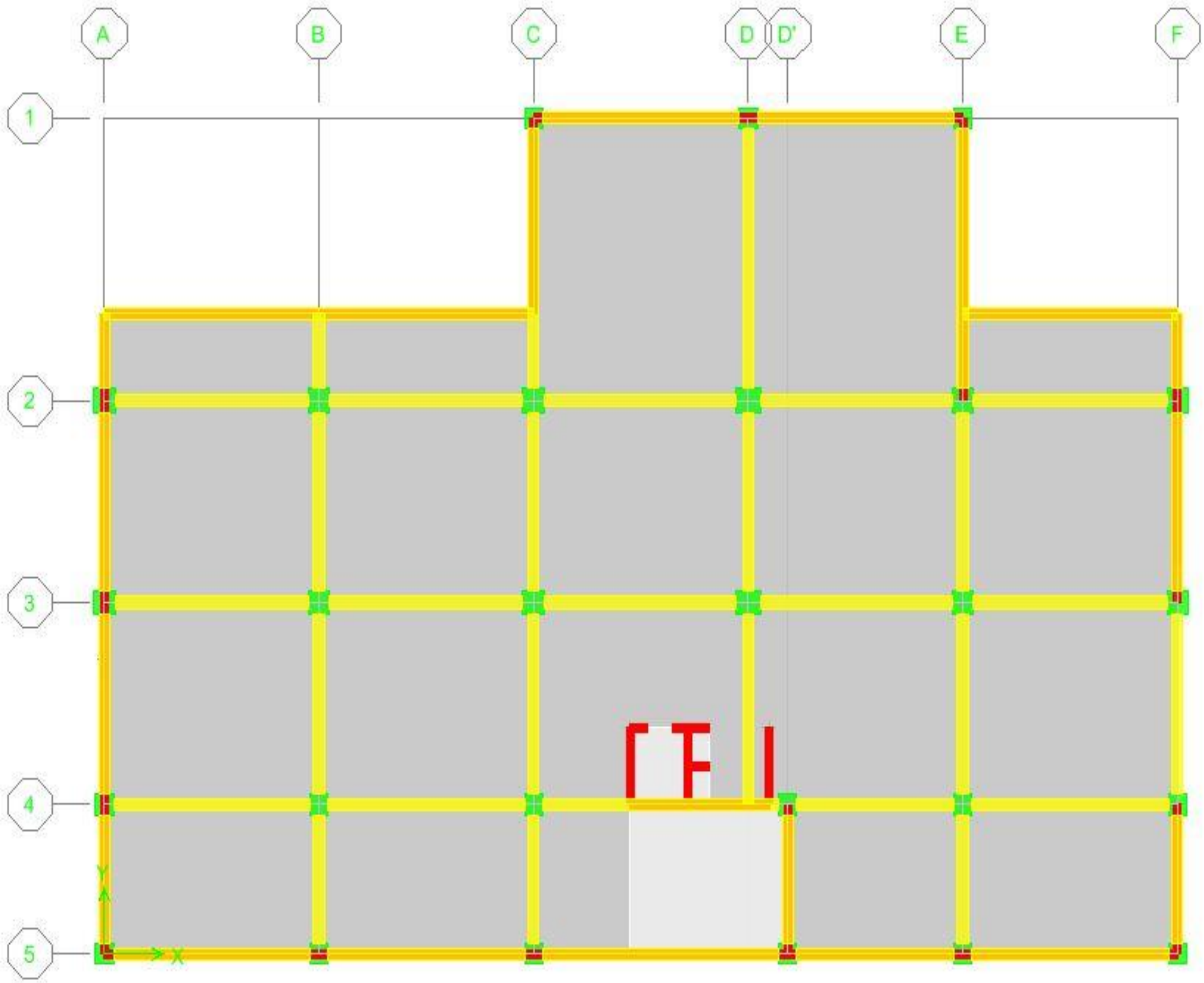
Seismic condition

Addis Ababa– Which is Zone 2 EBCS 8 table 1.3

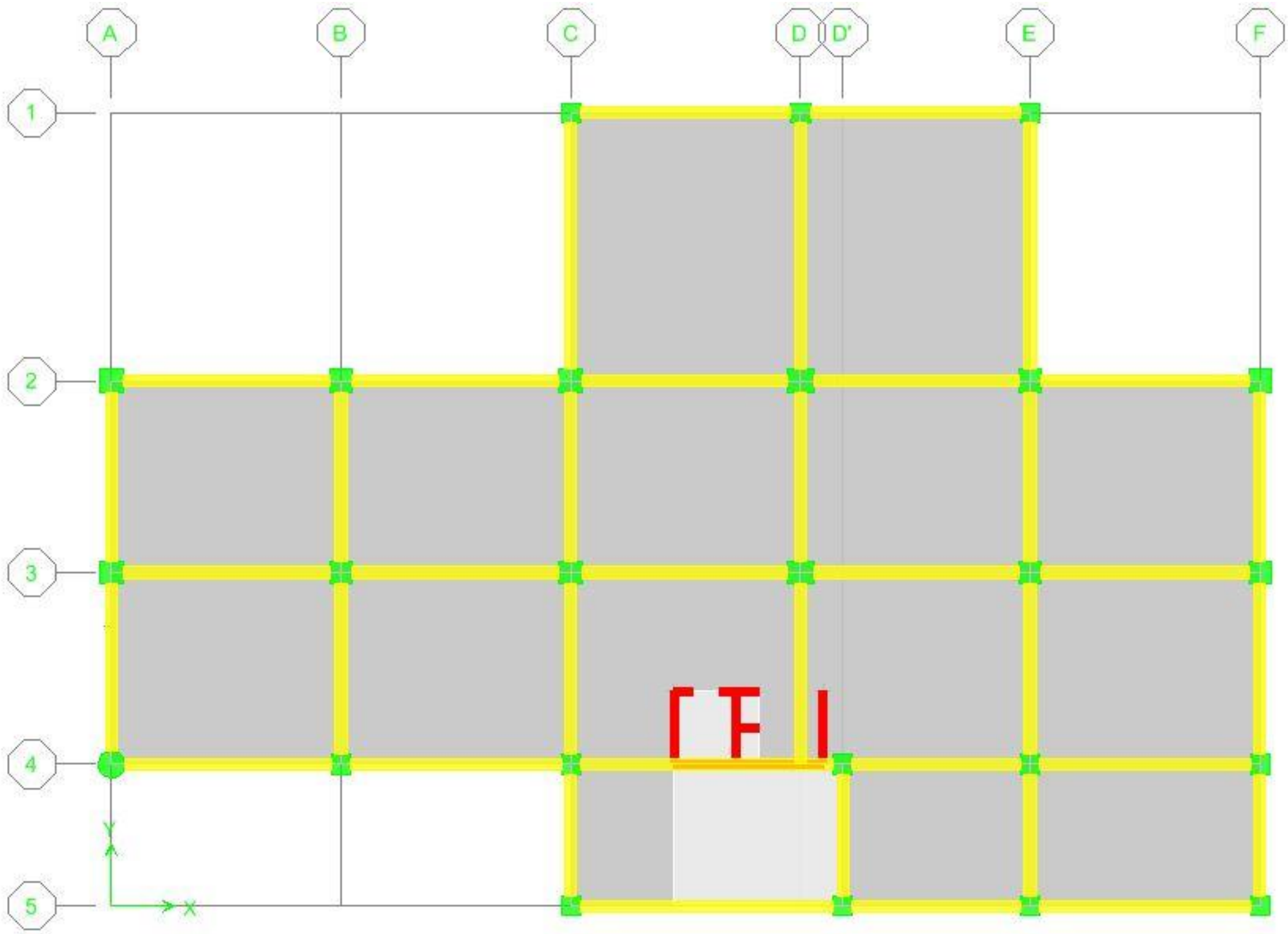
Characteristics



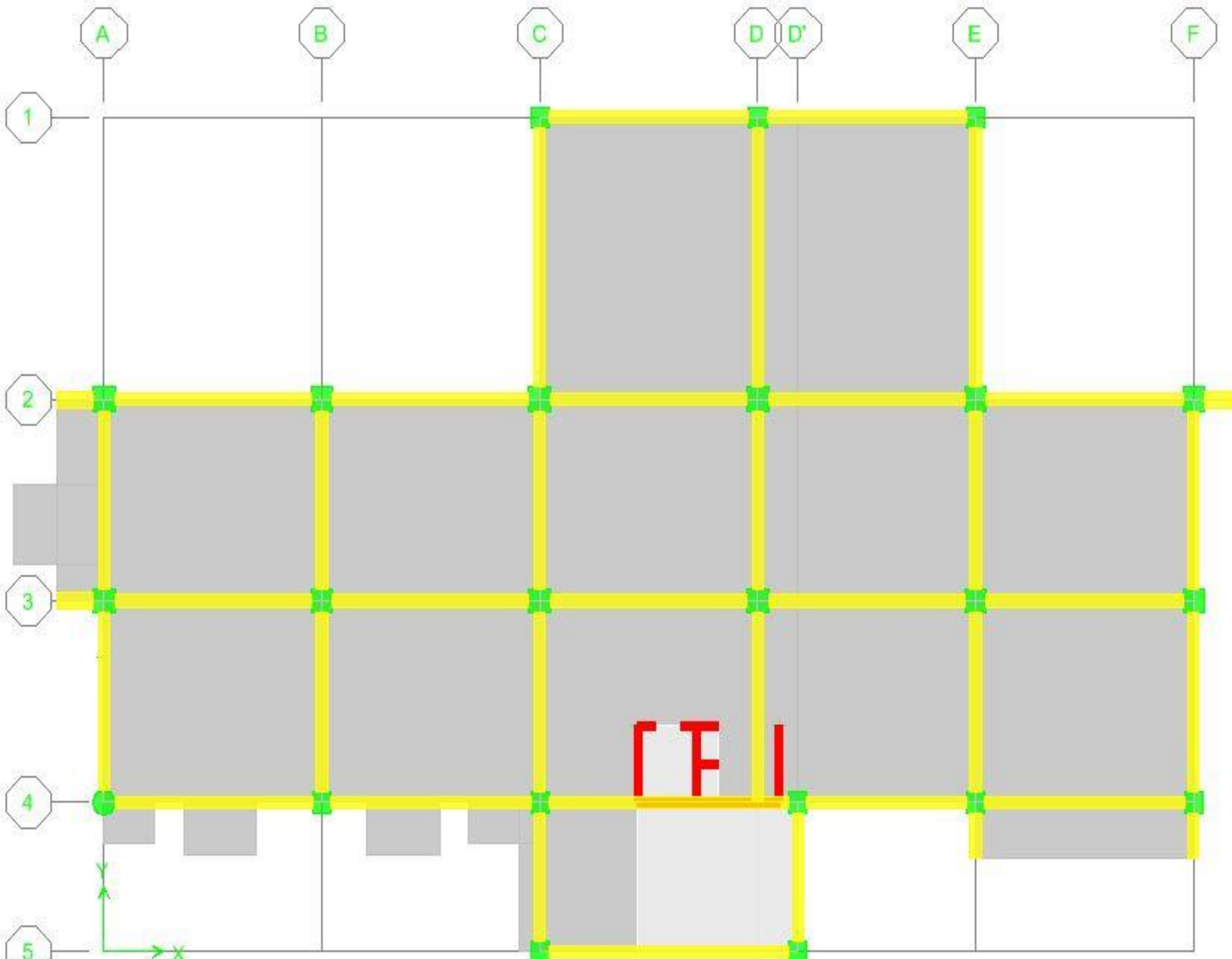
Basement Floor @ -2.50



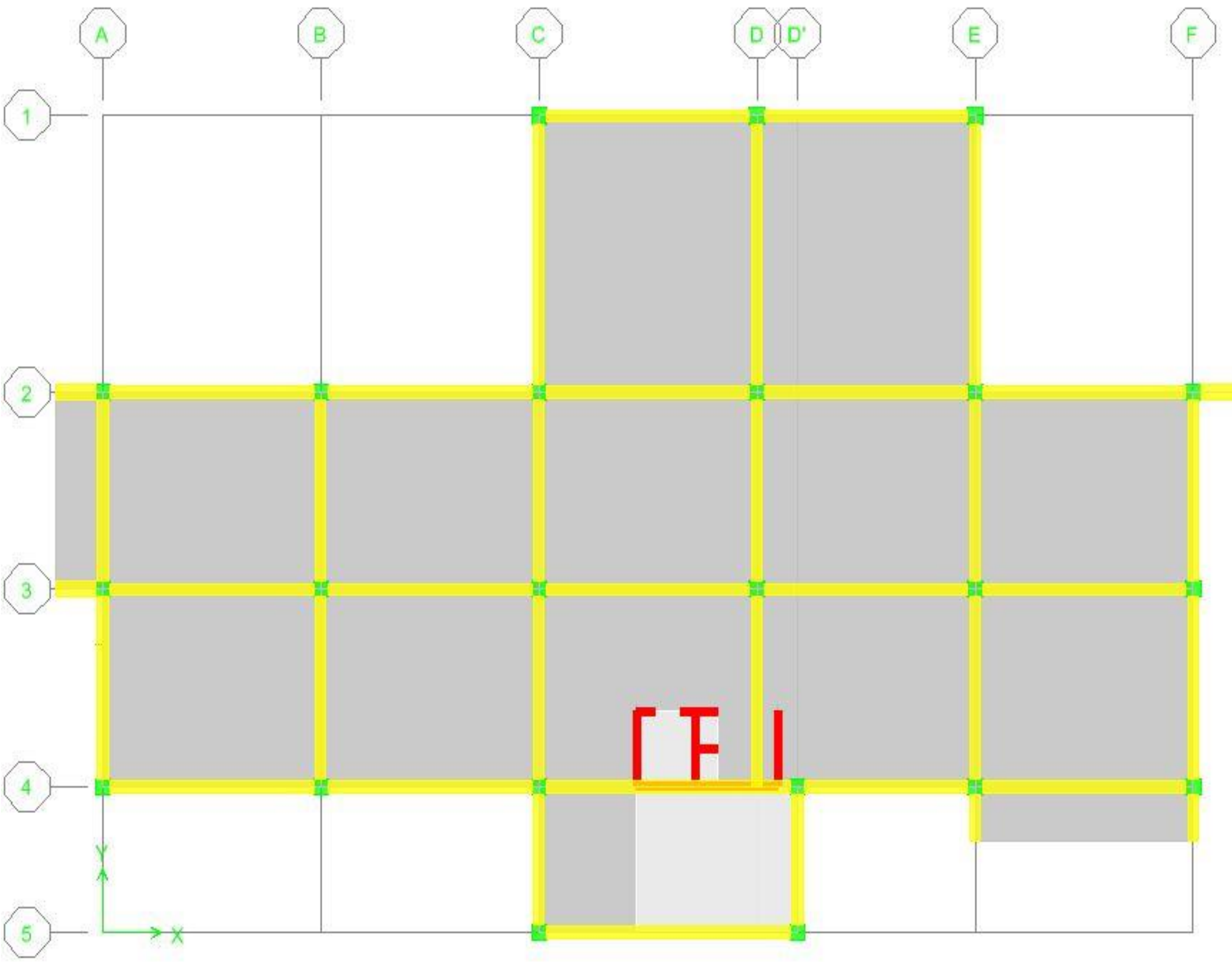
Ground Floor @ ± 0.00



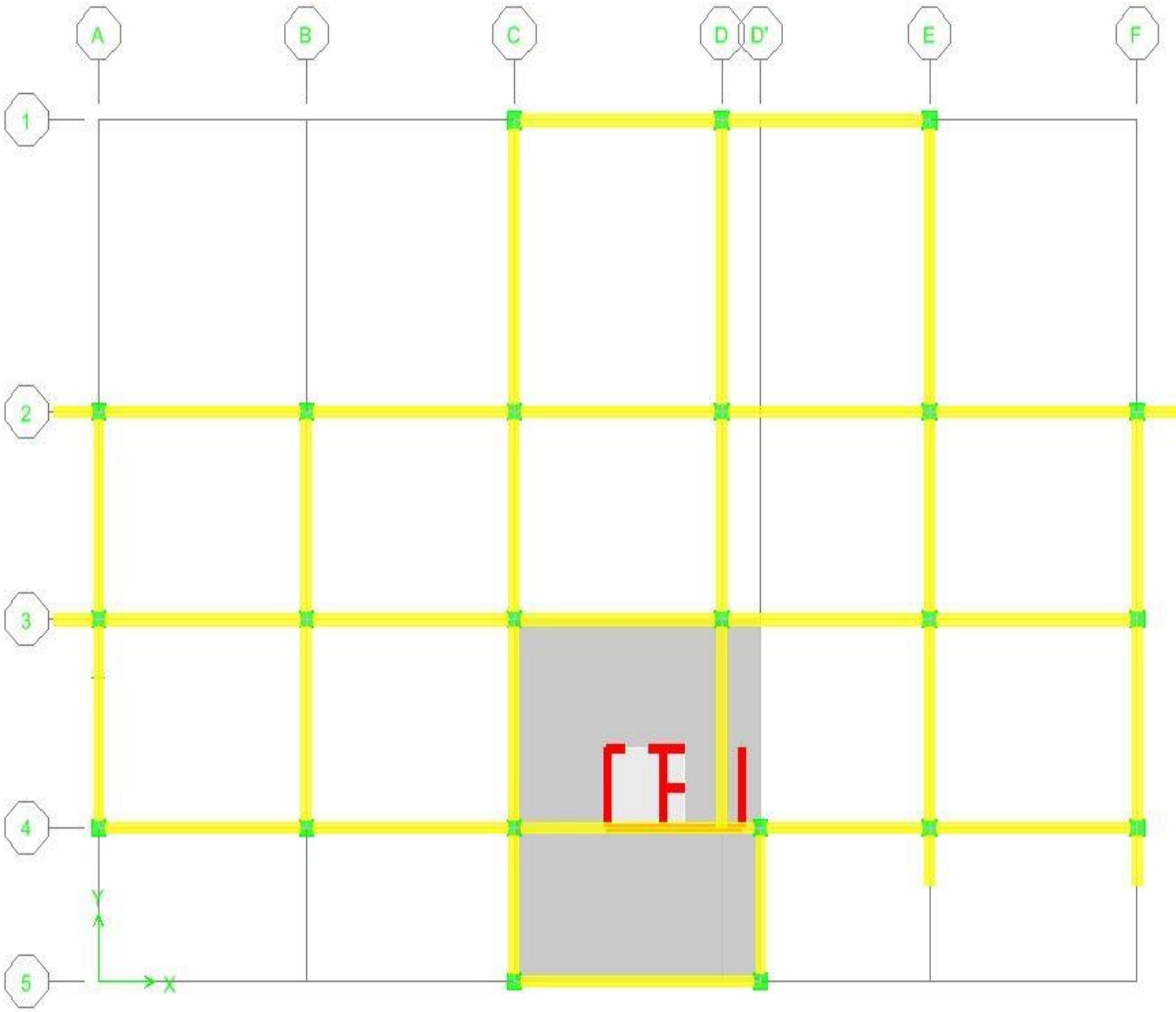
1st Floor @ +3.23



2^{nd} Floor to 7^{th} Floor



Terrace Floor



Roof Level

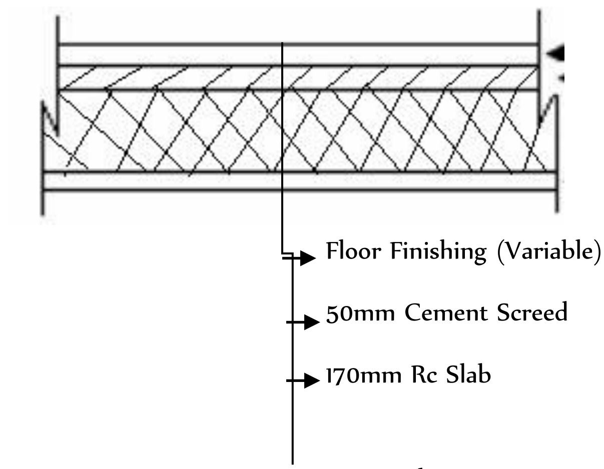
The building has staircase that provide vertical communication between floors. Solid slab has been adopted for the slab system. The columns are considered to be the main lateral force resisting system.

Material Properties

The material properties as well as the material partial safety factor dictated by the Ethiopian Building Code and Standards, EBCS, are clearly identified in the “Material Properties” section of the Structural Calculations.

Loading

Sectional (detail) elevation of floor slabs



Dead Load

Concrete	24 KN/m^3
Screed	23KN/m^3
Marble /Granite	27KN/m^3
Steel Structure	78.50 KN/m^3
PVC/Carpet	16KN/M^3
Terrazzo	23KN/m^3
Brick	22KN/m^3
Aluminum Curtain wall	1KN/m^2
Glass	0.027 KN/m^2/mm
Acoustic Ceiling	10KN/m^3
HCB	14KN/m^3

Partial Safety Factor: 1.3

Live Load

Function	Category		Live Load
Kitchen	A	General	2 KN/m^2
Cafeteria	C	C1	3 KN/m^2
Shop	D	D1	5 KN/m^2
Corridor	C	C3	5 KN/m^2
Retail	D	D1	5 KN/m^2
Balcony	A	balconies	4 KN/m^2
Toilet	A	general	2 KN/m^2
Shower	A	general	2 KN/m^2
Landing	A	satires	3 KN/m^2
Internet Cafe	C	C1	3 KN/m^2
Bed Room	A	general	2 KN/m^2
Janitor Room	D	D1	5 KN/m^2

Partial Safety Factor: 1.6

Earthquake

Loading for the different occupancies are clearly identified in the "Loading" section of the Structural Calculations. The above loadings make up a total of nine different combinations.

No.	Combination Name	Factored Loading Combination	
		Vertical	Lateral
1	Gravity	1.3*DL +1.6*LL	-
2	CEQXT	0.75(1.3*DL +1.6*LL)	+EQx
3	CEQXTN	0.75(1.3*DL +1.6*LL)	-EQx
4	CEQXB	0.75(1.3*DL +1.6*LL)	+EQy
5	CEQXBN	0.75(1.3*DL +1.6*LL)	-EQy
6	CEQXL	0.75(1.3*DL +1.6*LL)	+EQx
7	CEQXLN	0.75(1.3*DL +1.6*LL)	-EQx
8	CEQXR	0.75(1.3*DL +1.6*LL)	+EQy
9	CEQXRN	0.75(1.3*DL +1.6*LL)	-EQy

Out of the nine combinations the critical case was taken for the analysis and design of beams, slab and columns.

Analysis

The vertical loads on the slabs were calculated and applied to the different slab panels. The slab reactions on the supporting beams were determined and input in the 3D ETABS model so that they would be analyzed together with the calculated lateral loads.

The seismic analysis of the building was more involved. One type of analysis was conducted BASED ON EBCS 8

- A pseudo-static analysis

According to EBCS 8 section 2.2.2 "Structural Regularity", Table 2.1 a building with plan regularity but elevation irregularity can be analyzed using the pseudo-static analysis method but with spatial (3D) model. The particular building falls under that category, i.e. it has plan irregularity but enjoys vertical regularity.

The pseudo-static analysis followed the procedure below

Calculate FloorLoads & FloorCenter of Mass

⇩

Calculate Seismic Base Shear

⇩

Distribute the Seismic Load on Each at the Center of Mass of the Floor

⇩

Apply the Seismic Load on Each at the Center of Mass of the Floor

⇩

From Slab Analysis, get reactions of Slab loads on Beams

⇩

Gravity loads combined in accordance with EBCS 8 1.4.3

All joints on a particular floor were connected together by the "diaphragm" feature of ETABS Non Linear version 9.7.0. This feature represents the presence of a solid slab (diaphragm), which will allow the distribution of lateral forces to the lateral force resisting systems in accordance with their stiffness. The seismic force on the floors was applied in two orthogonal directions with positive and negative signs.

Earthquake Analysis

Location of the Building = Addis Ababa – Zone 2

Zone = 0(EBCS 8-1995 TABLE 1.3)

α_o = 0.05Bedrock Acceleration. (EBCS 8-1995 TABLE 1.1)

I = 1.00 Importance Factor (EBCS 8-1995 TABLE 2.4)

$\alpha = \alpha_o I$ = 0.05(EBCS 8-1995 ARTICLE 1.4.2.2(5))

S = 1.5 Site Coefficient from Subsoil Condition(EBCS 8-1995 TABLE 1.2)

H = 30.23 Height of the Building.

C_1 =0.075(EBCS 8-1995 ARTICLE 2.3.3.2.2)$T_1=C_1H^{3/4}$ =0.967 (Fundamental Period of Building, in sec) (EBCS 8-1995 ARTICLE 2.3.3.2.2)

$\beta = 1.2S/T^{2/3} \leq 2.5$ 1.[illegible] Design Response Factor(EBCS 8-1995 ARTICLE 1.4.2.2(6))

γ_o = 0.20 Basic Value of Behavior Factor(EBCS 8-1995 TABLE 3.2)

K_D= 2.00 Factor Reflecting Ductility Class(EBCS 8-1995 ARTICLE 3.3.2.1(4))

K_R= 1.00 Factor Reflecting Regularity in Elevation(EBCS 8-1995 ARTICLE 3.3.2.1(6))

K_W= 1.00 Factor Reflecting Prevailing Failure Mode(EBCS 8-1995 ARTICLE 3.3.2.1(7))

$\gamma = \gamma_o K_D K_R K_W \leq 0.70$0.40 Behavior Factor(EBCS 8-1995 ARTICLE 3.3.2.1(1))

$S_d(T_1) = \alpha\beta\gamma$ =0.03

Structural *Design*

Modeling

The model of the building shall adequately represent the distribution of stiffness and mass. So that all significant deformation shapes and inertia force Forces are properly accounted for under the seismic action consider.

The structure must be considered to consist and the number of vertical and lateral load resting systems, connected by horizontal diaphragm.

Additional Torsional Effect

In additional to the actual eccentricity in order to include uncertainties the location of masses and in the spatial variation of the sample motion, for each floor accidental eccentricity is provided.

$e_{ii} = \pm\ 0.05Li$

e_{ii} = accidental eccentricity of storey mass is i from normal condition

Li floor dimension perpendicular to the direction of the seismic action

Methods of analysis

The basic method for determining the seismic effect is static analysis elastic

Base shear force

$F_b = S_d(T_1)\ w$

$S_d(T_1)$ design spectrum

T_1 =fundamental period of vibration.

W =Seismic Dead lode Computed

$T1 = C_1H^{3/4}$ for building up to 80mheight

H= Height of building

C1=0.085 for steel moment resisting frames.

0.075 for reinforced concert moment resisting and elastic braced system

0.050 for all other building.

METHOD OF DESIGN AND DESIGN CODES

Serviceability limit state design method was used for member sizing and designing. The codes used are:

Ethiopian Building Code Standard (EBCS-1)

Ethiopian Building Code Standard (EBCS-8)

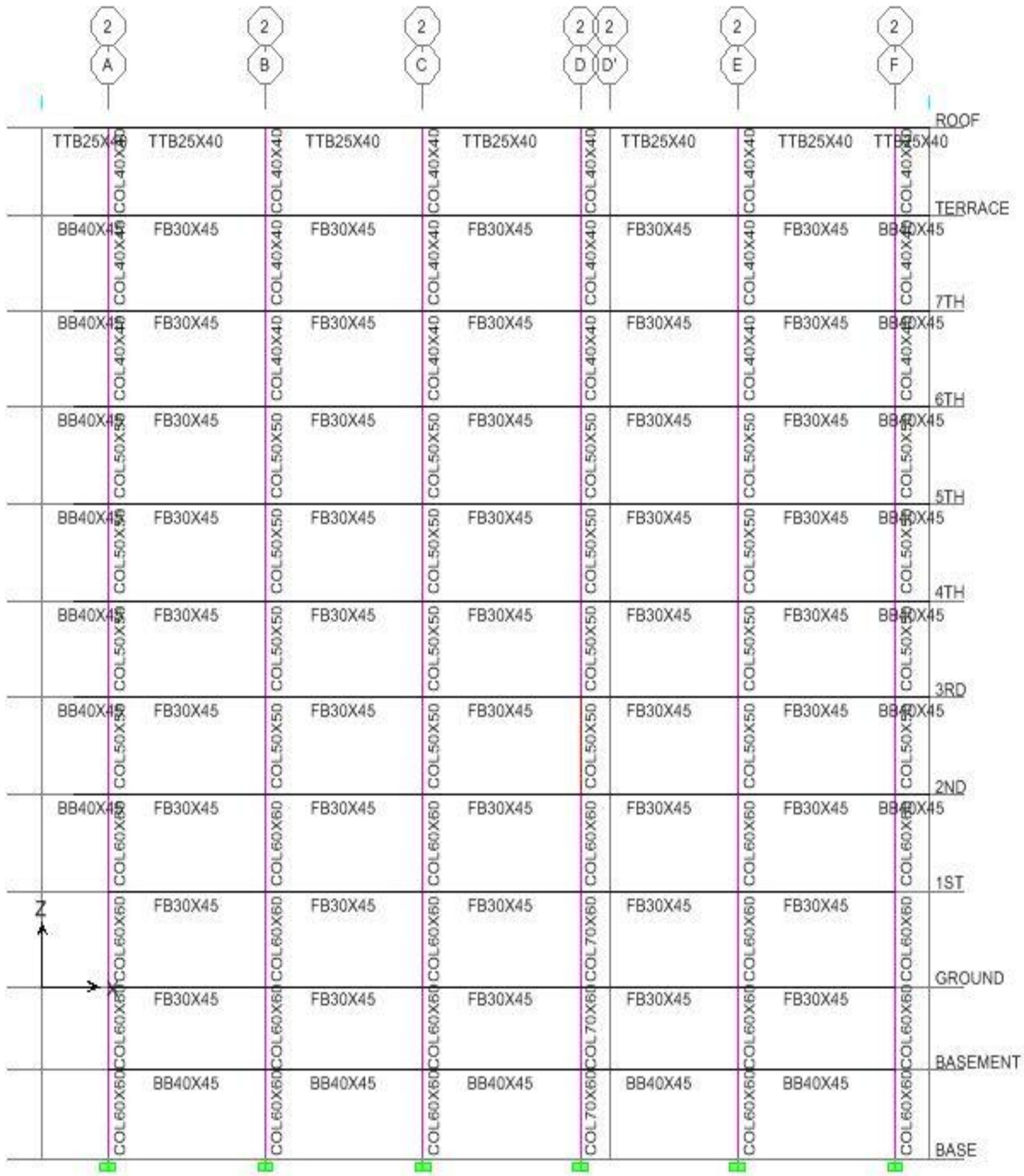
Ethiopian Standard code of practice (ESCP-1)

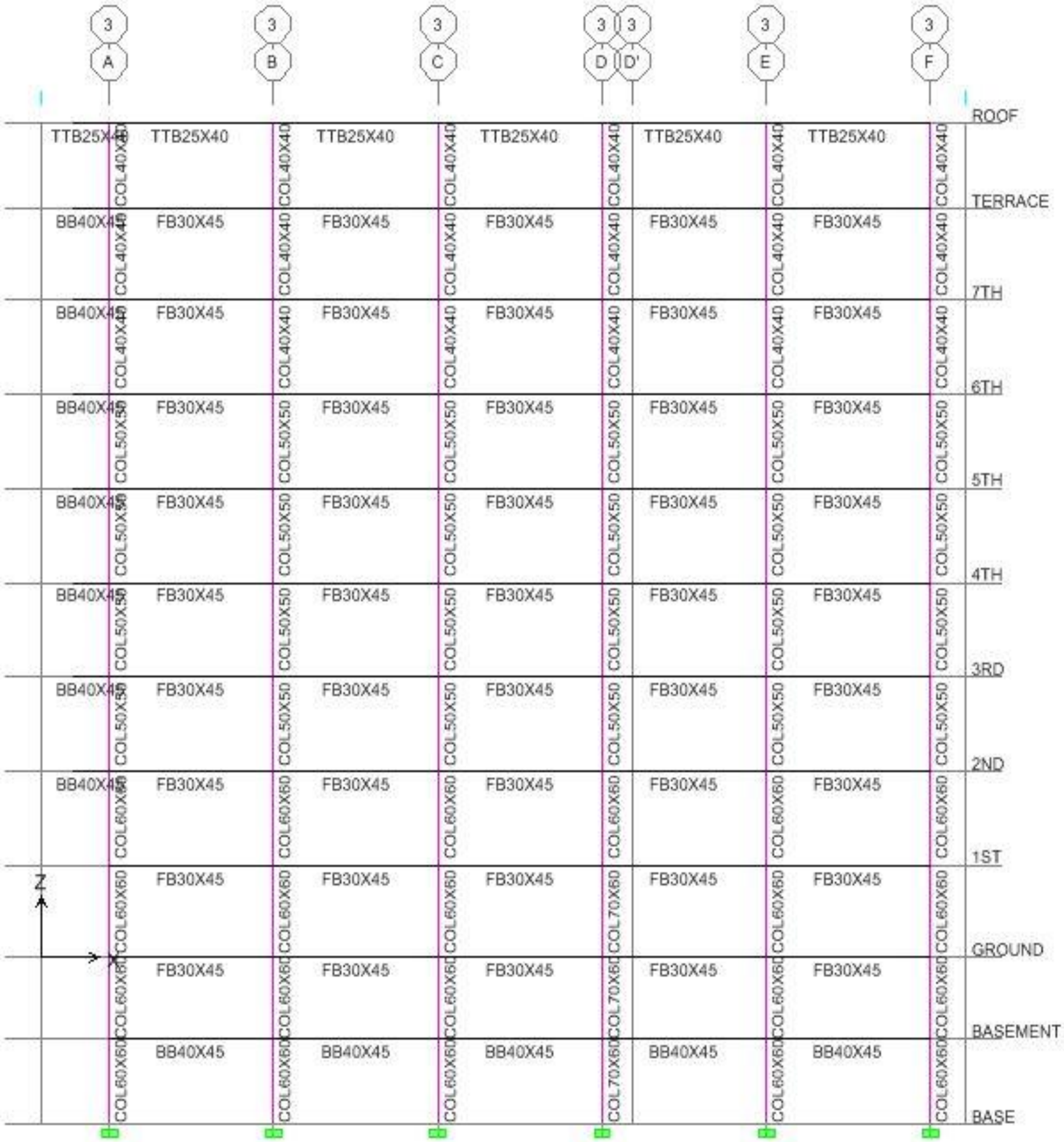
Euro code 2-1992(as used by software)-almost similar to EBCS-2, 1995.

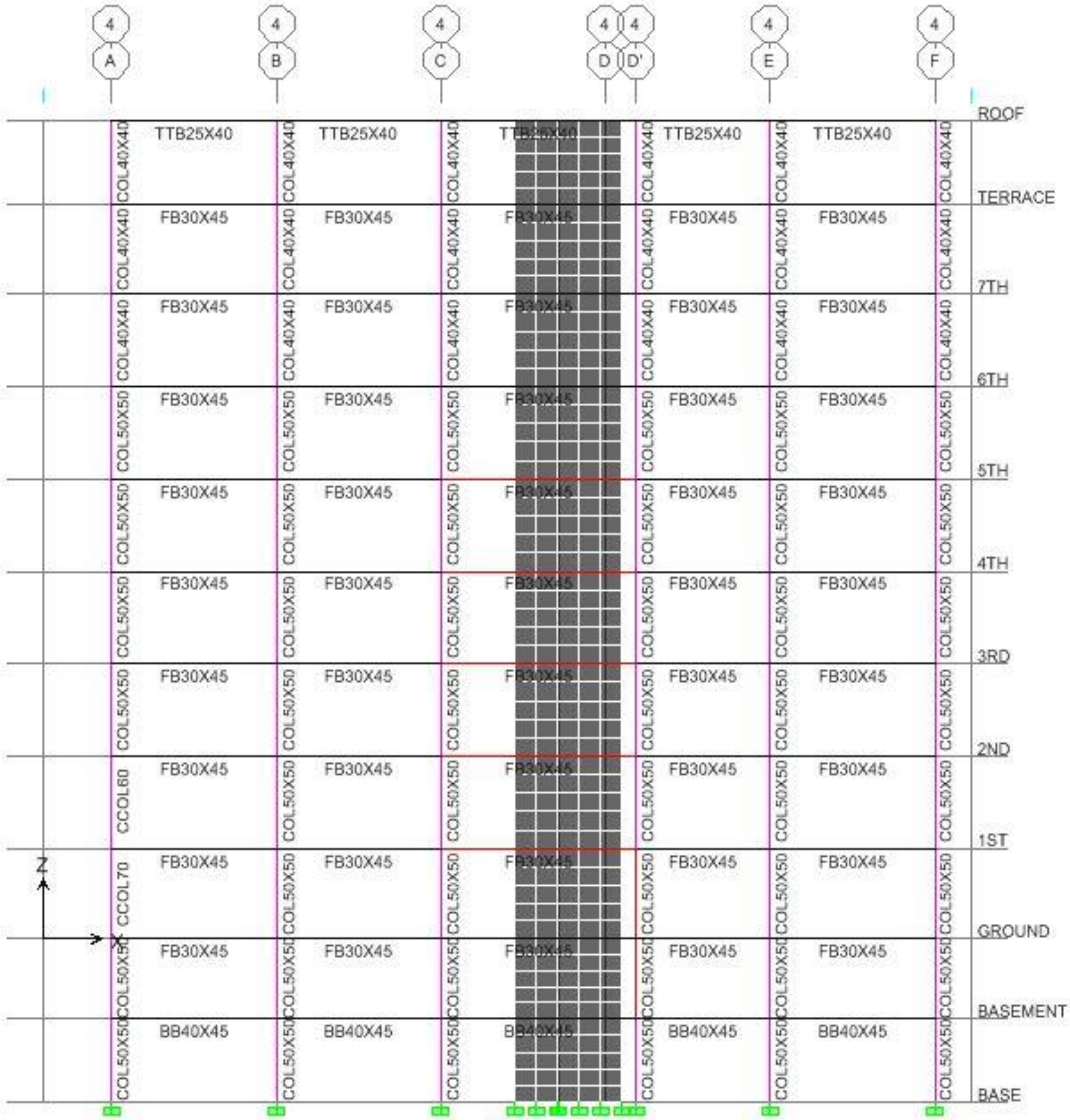
SOFTWARE

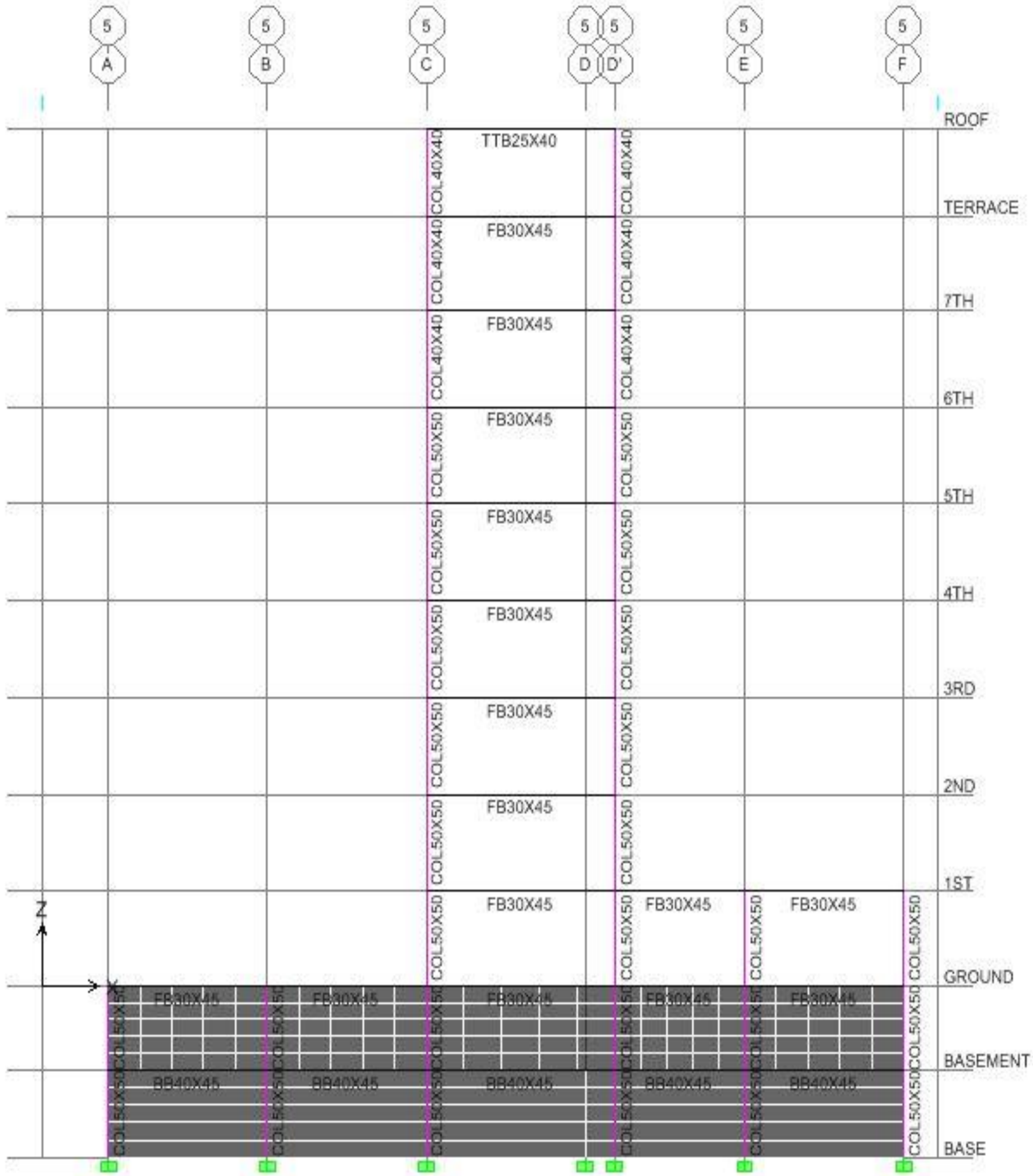
ETABS, version, 9.7.0 is used for the analysis and design of the building by modeling as a 3-D space frame.

Frame Section

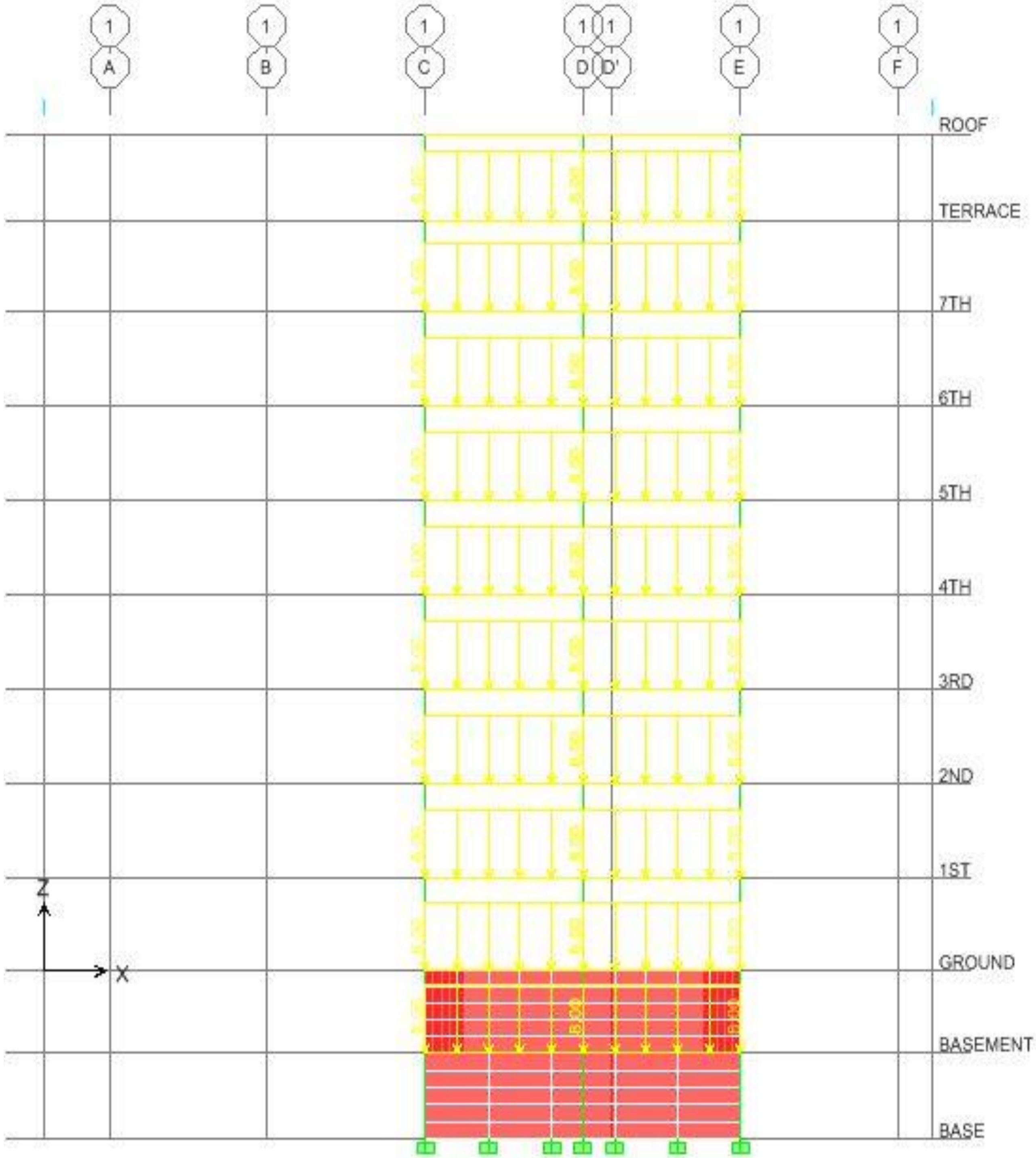


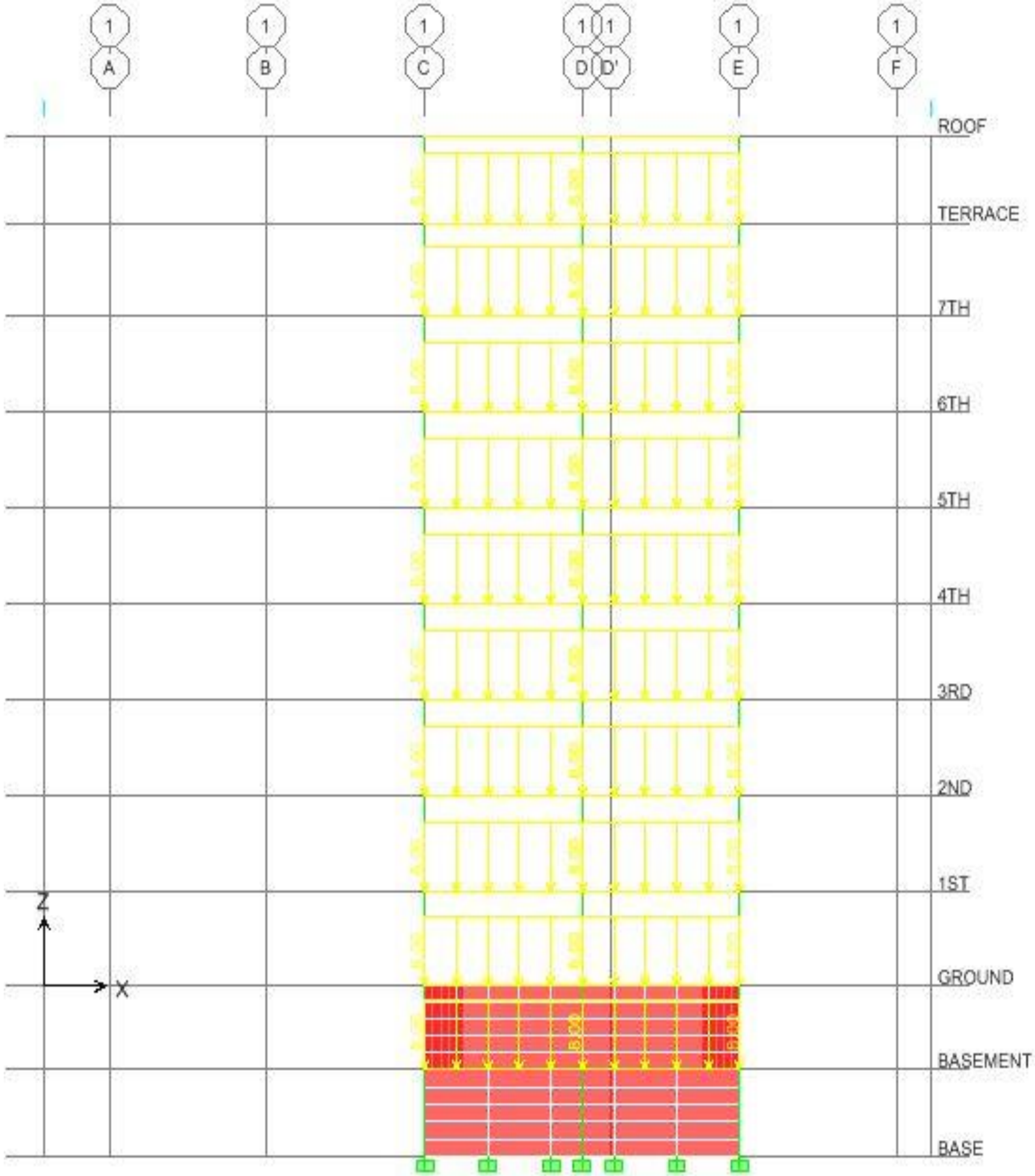

3 A
3 B
3 C
3 D
3 D'
3 E
3 F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
TTB25X40
BB40X45
FB30X45
COL40X40
COL50X50
COL60X60
COL70X60
Z
X


4
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
TTB25X40
FB30X45
BB40X45
COL40X40
COL50X50
CCOL60
CCOL70
Z
X

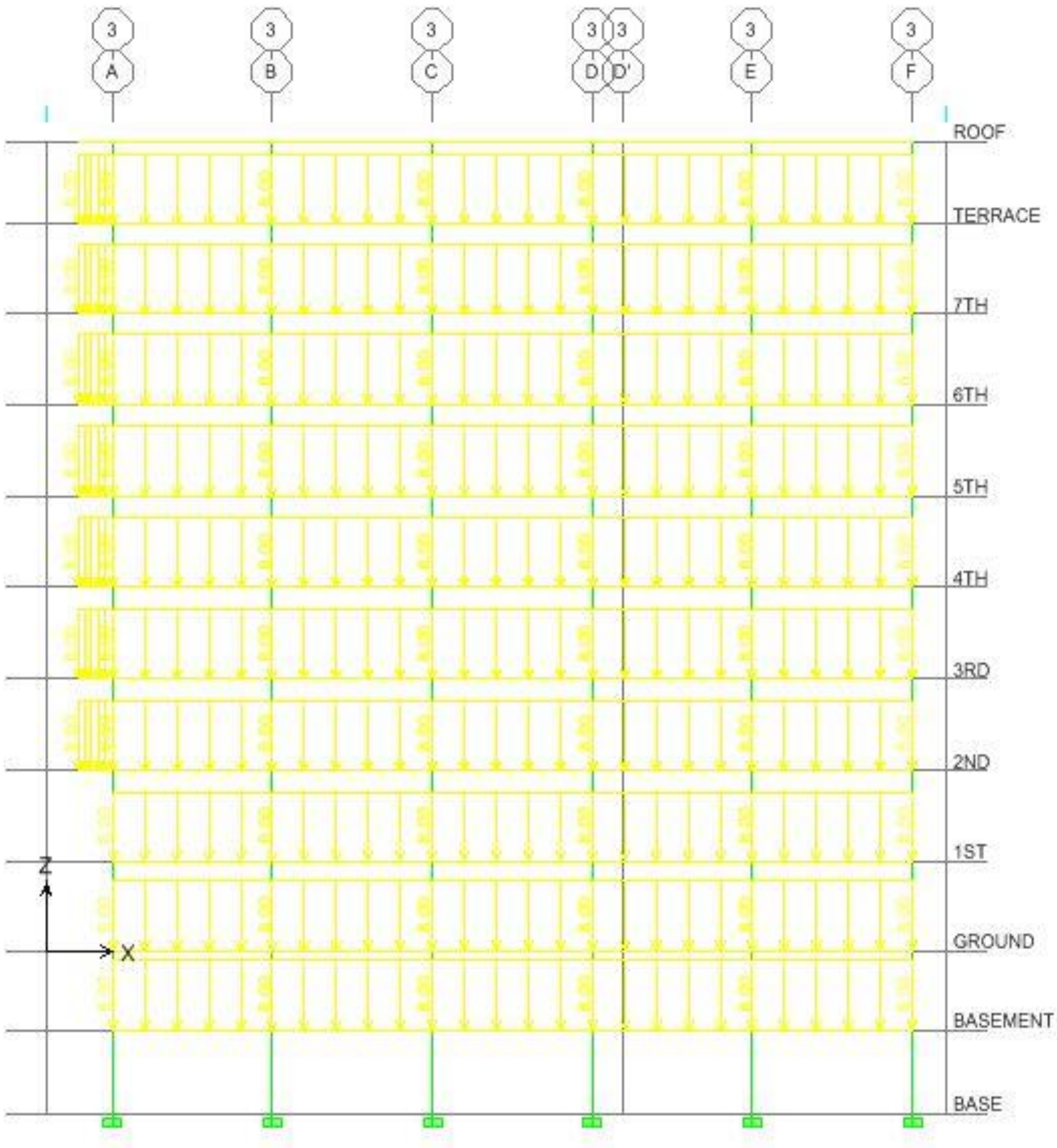

5
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
TTB25X40
FB30X45
BB40X45
COL40X40
COL50X50
Z
X

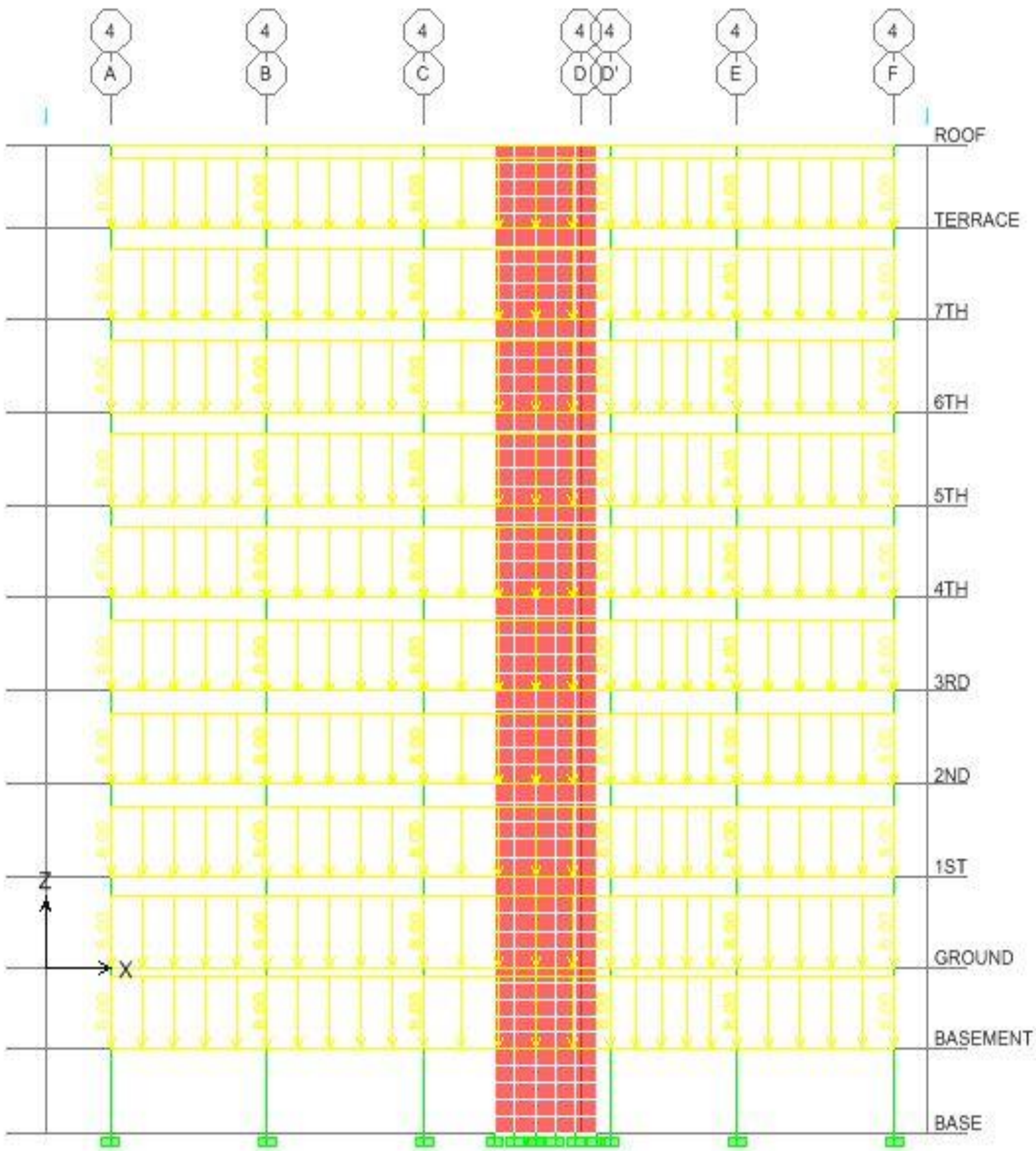
Frame Loading



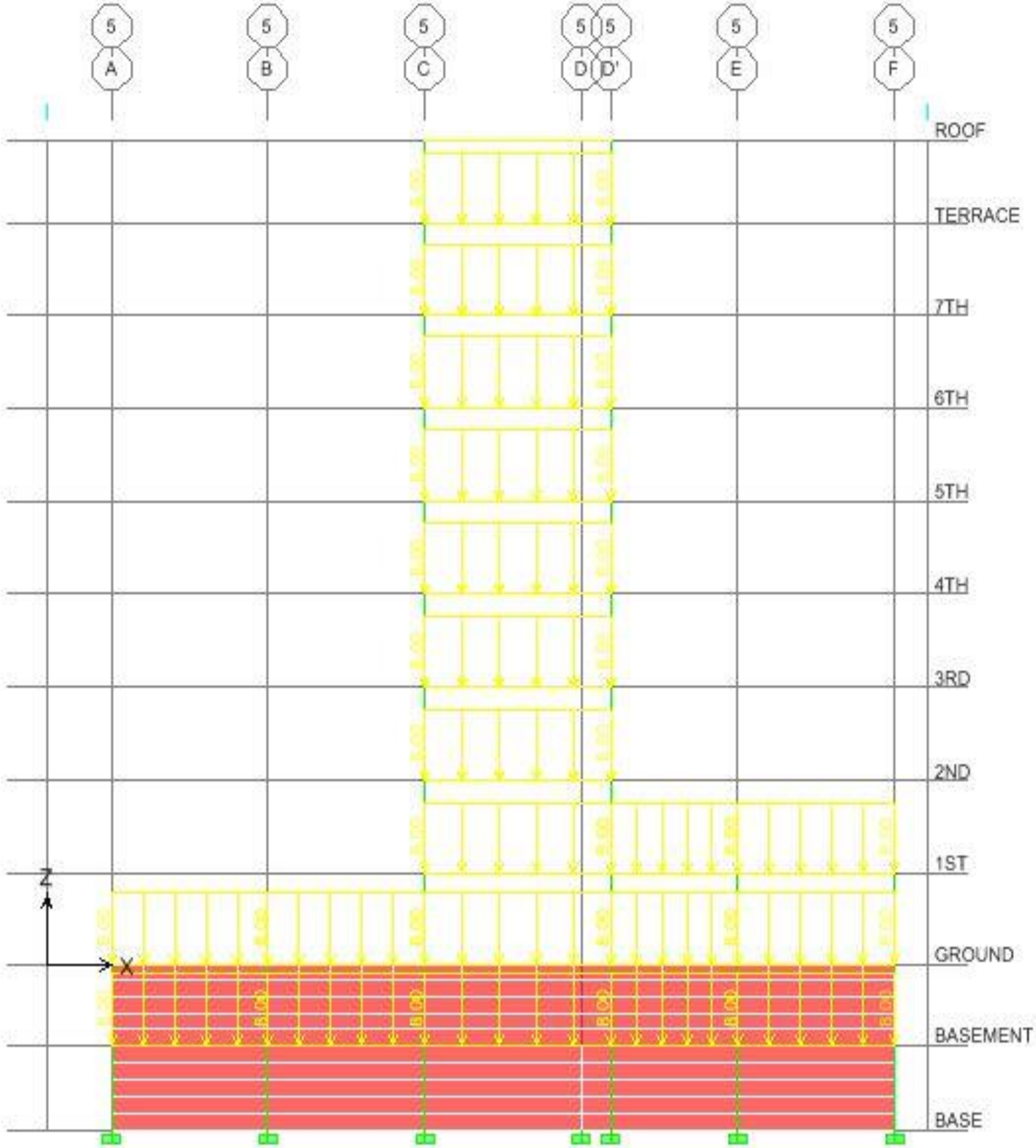


1
A
1
B
1
C
1
D
1
D'
1
E
1
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

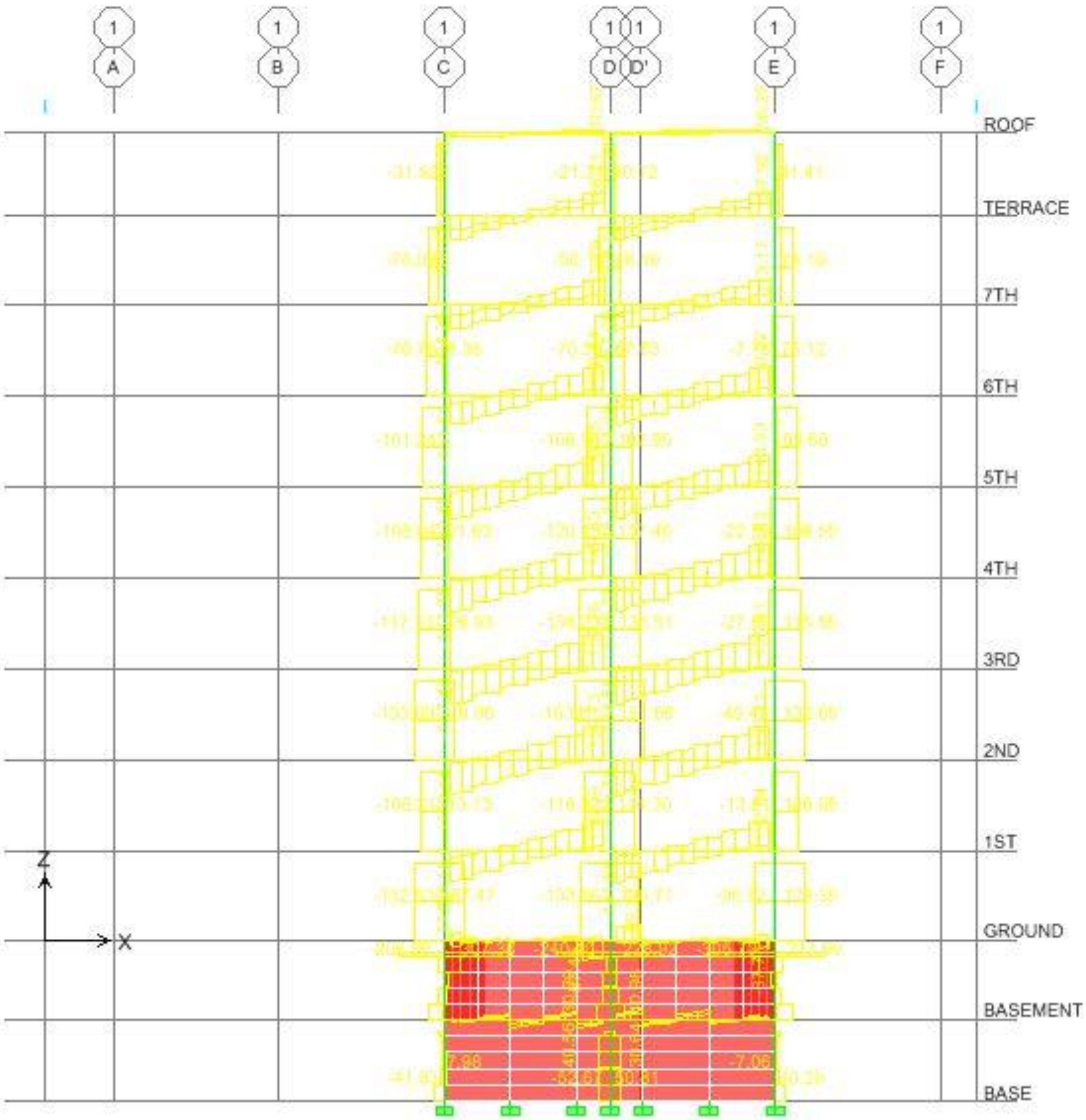

3
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

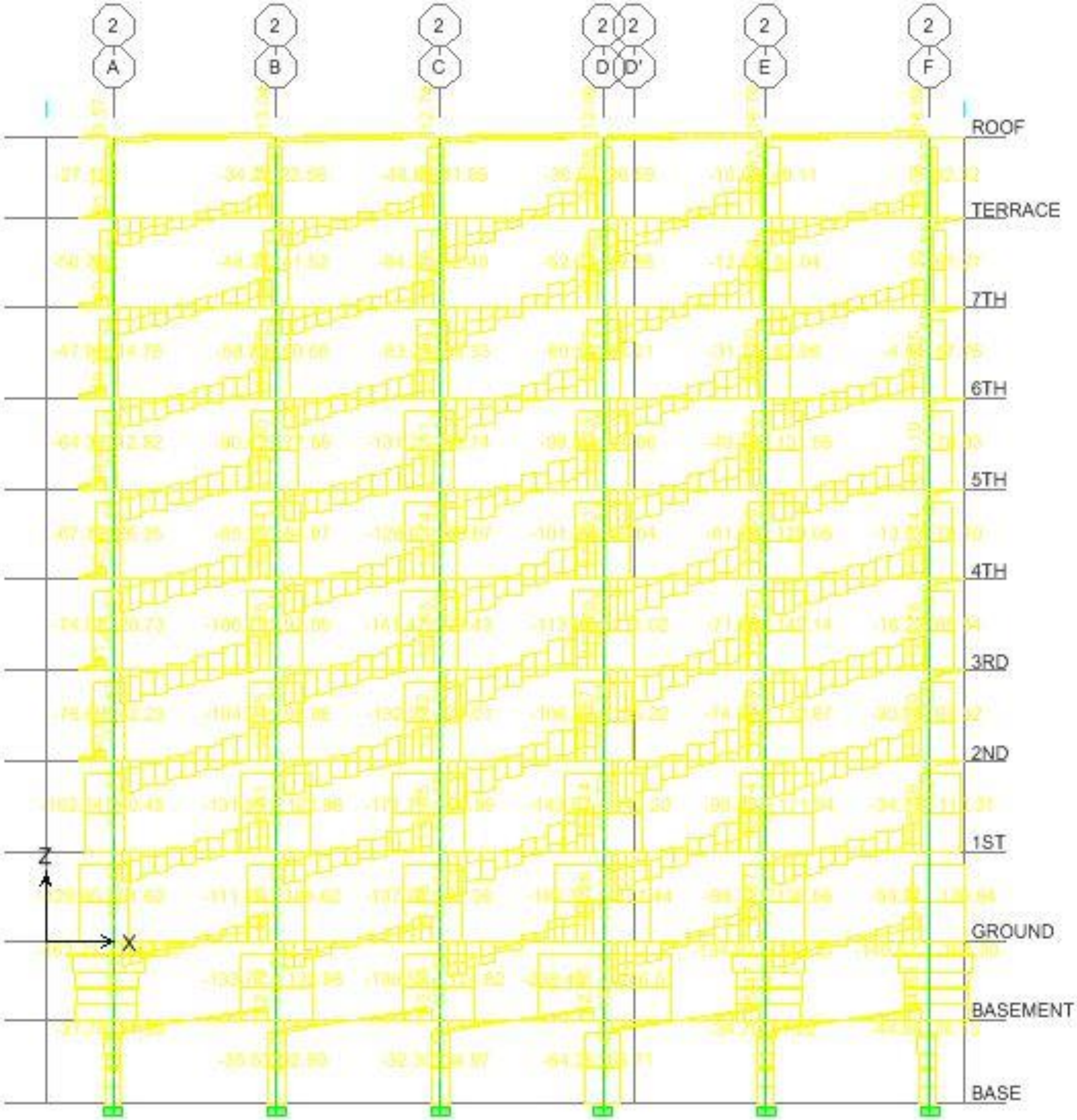


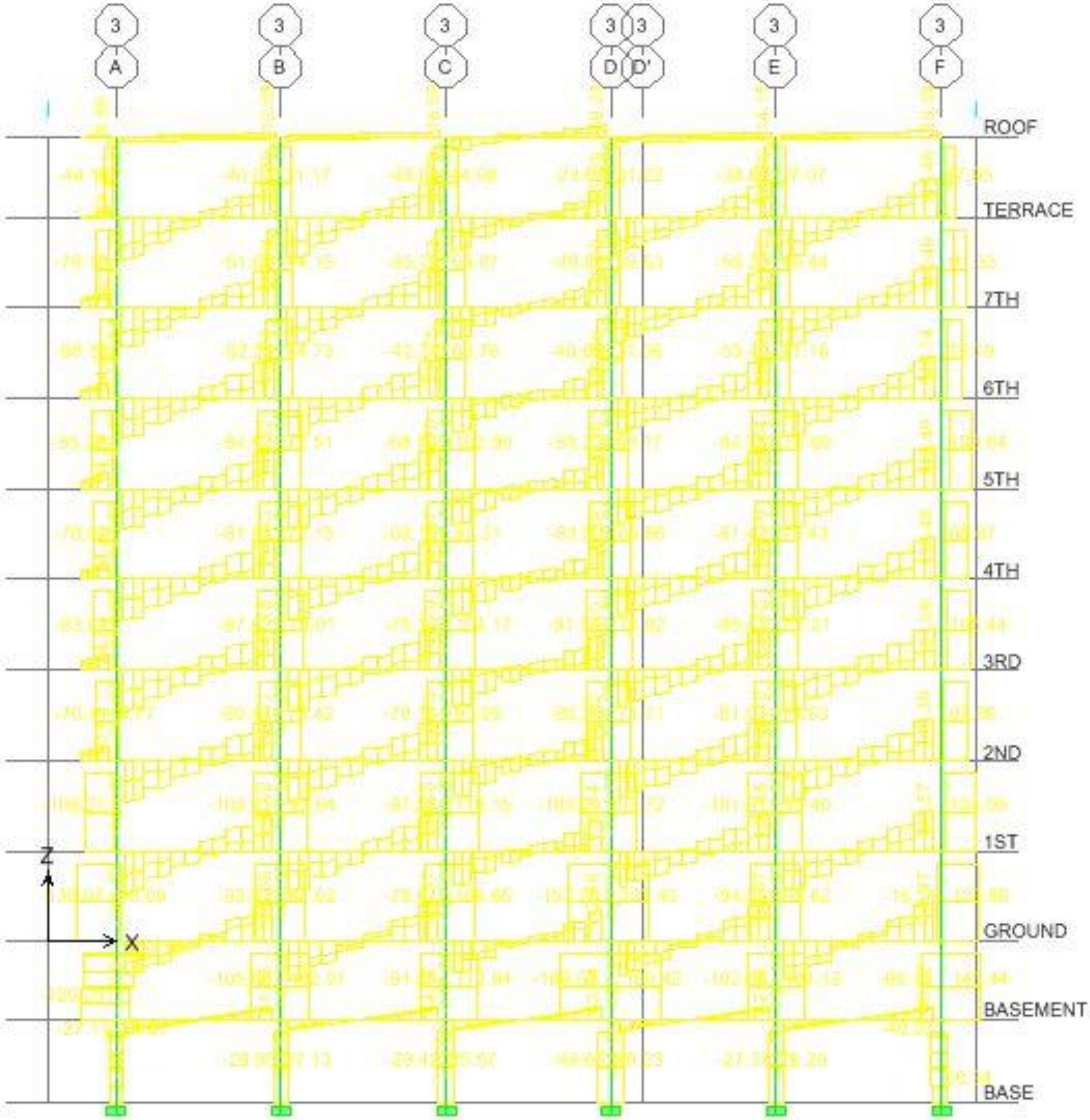
4
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

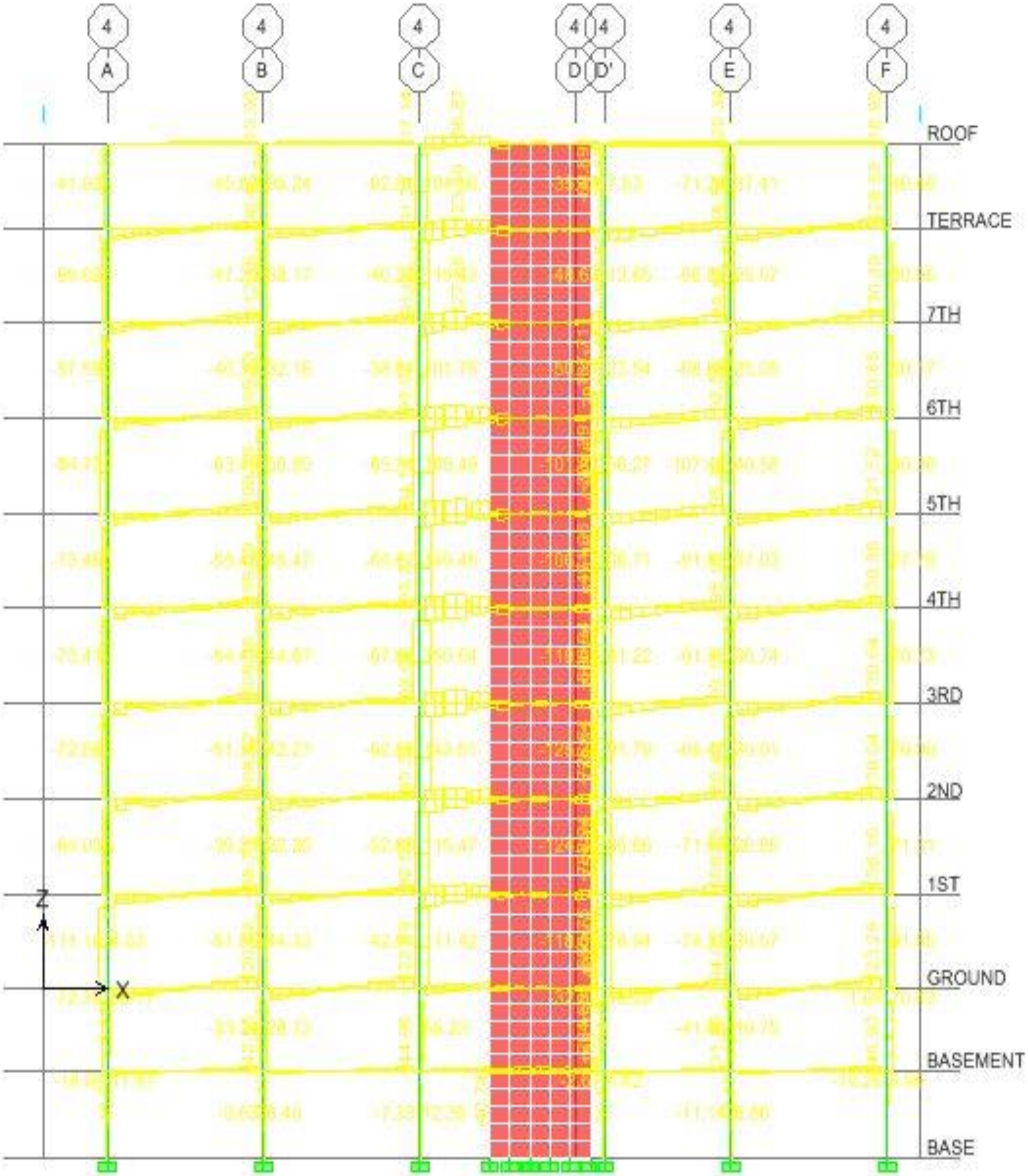

5
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

Shear Force Diagram

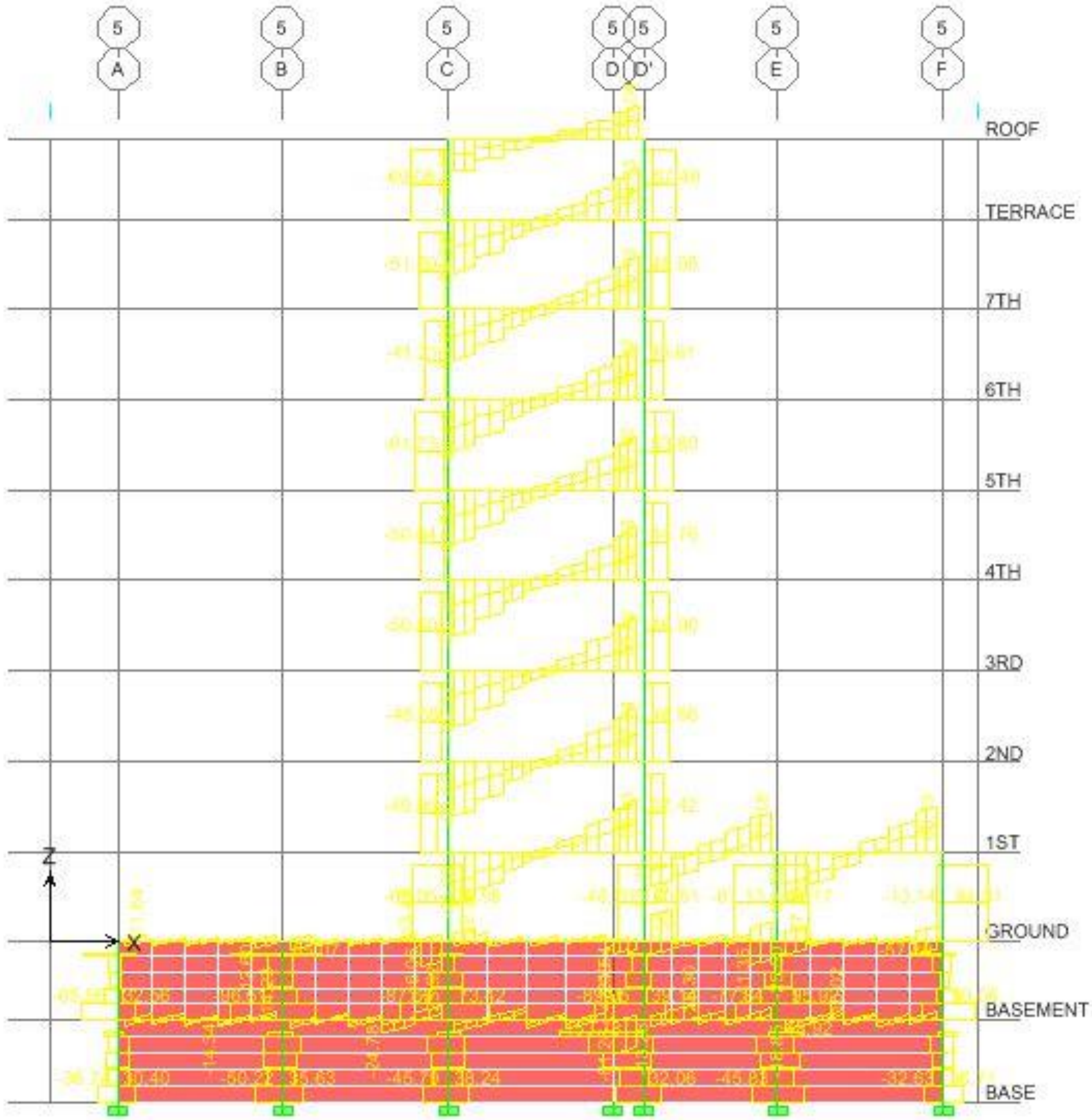



2
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X


3
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

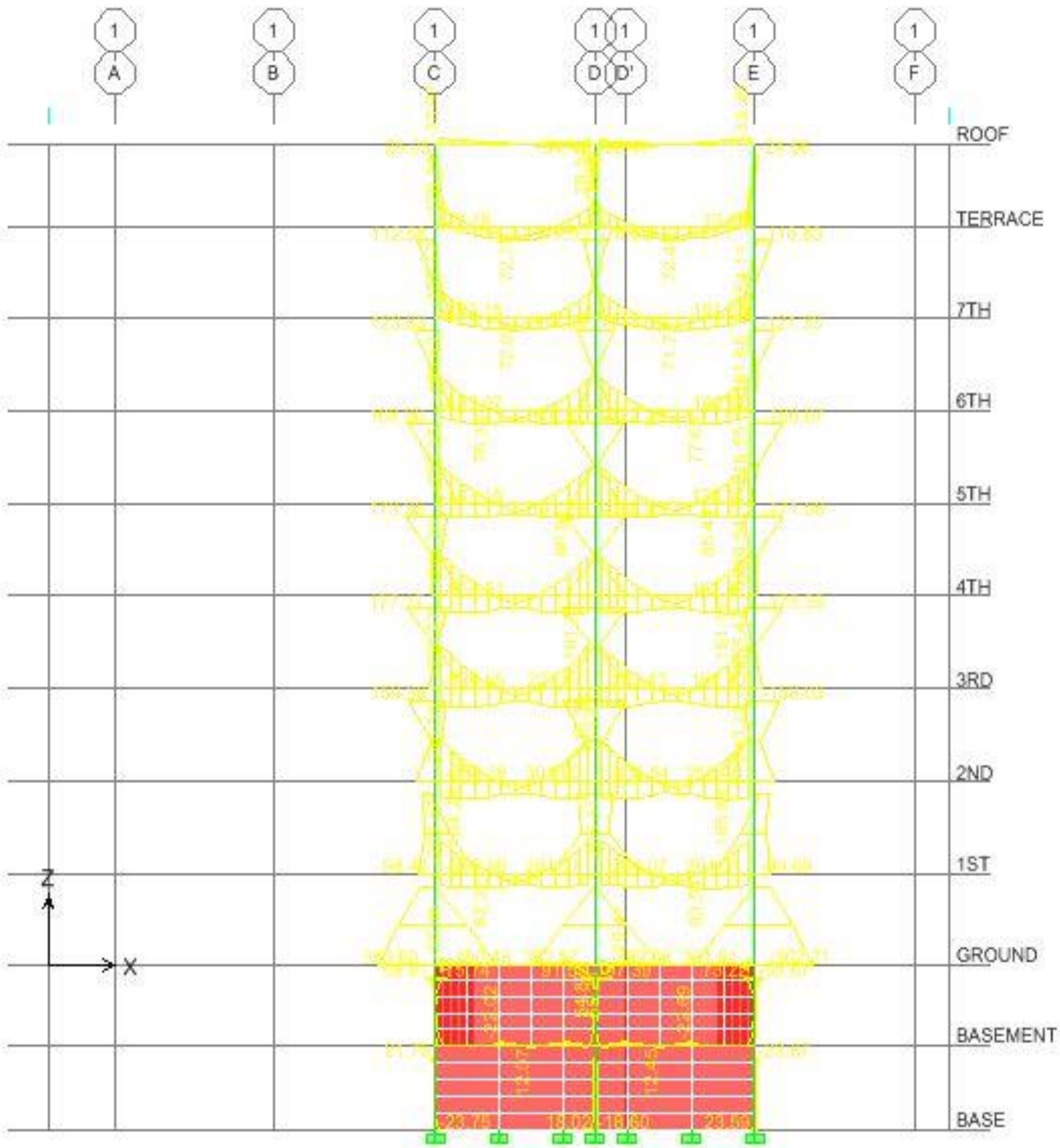


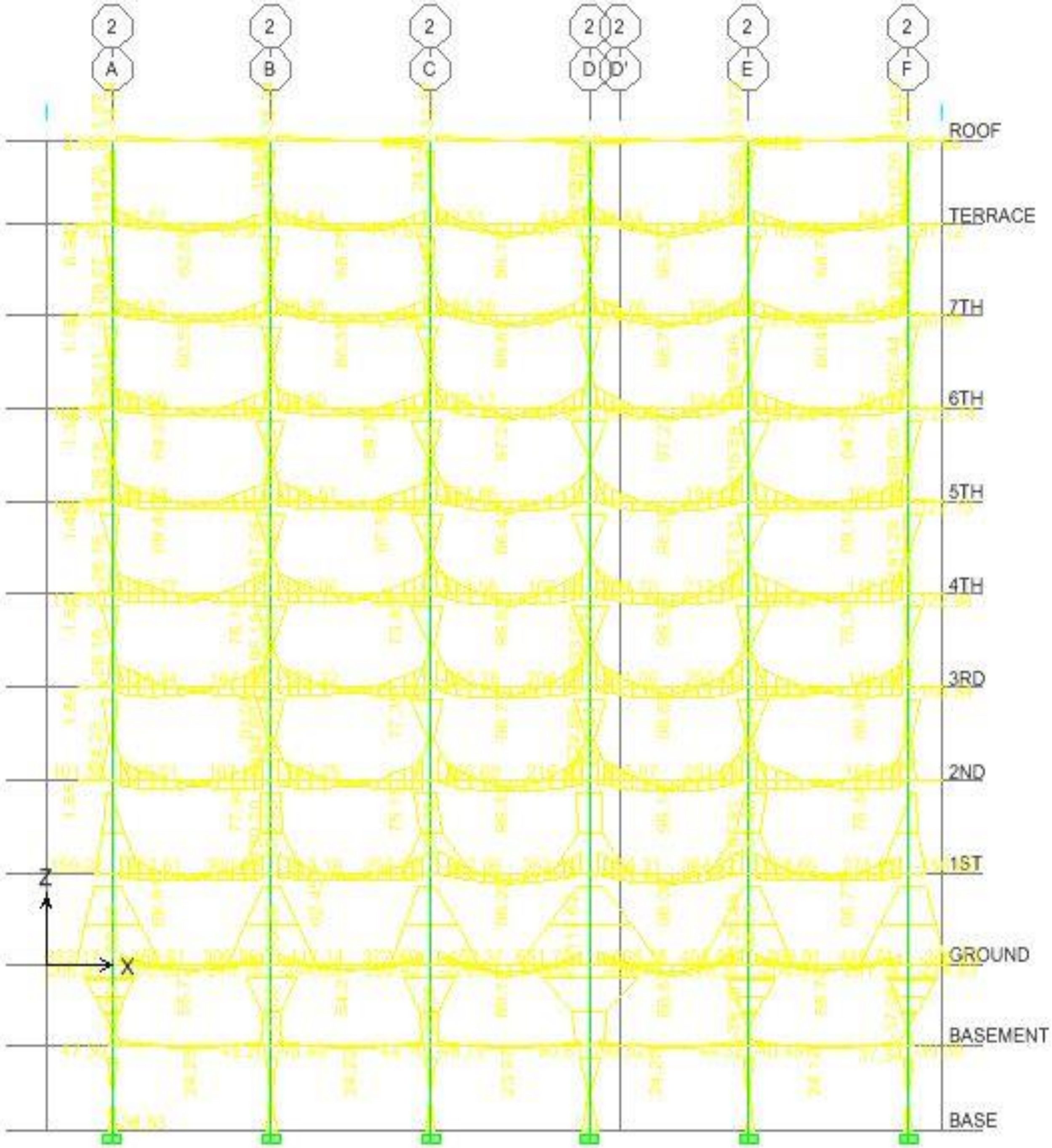
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X



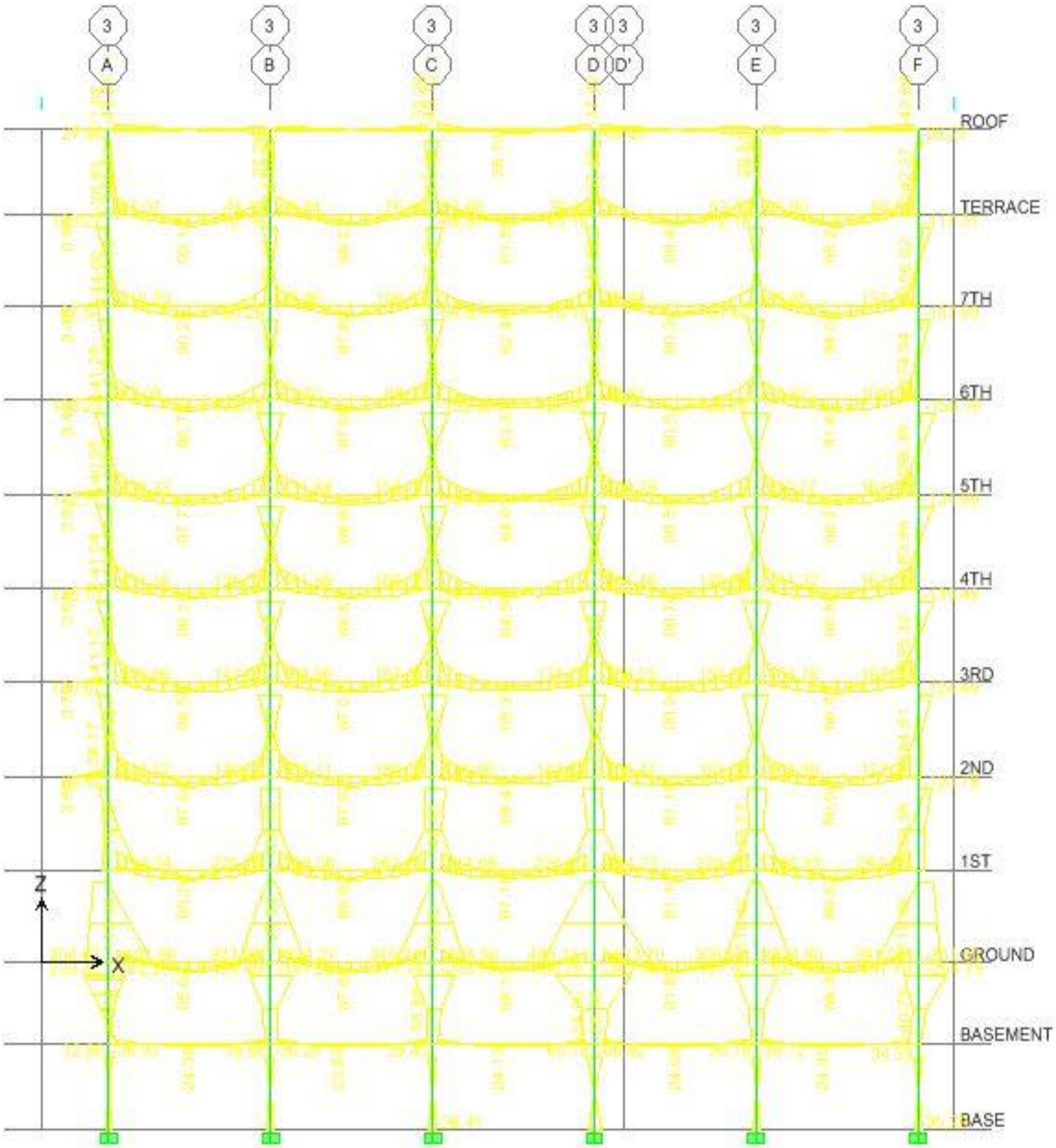
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

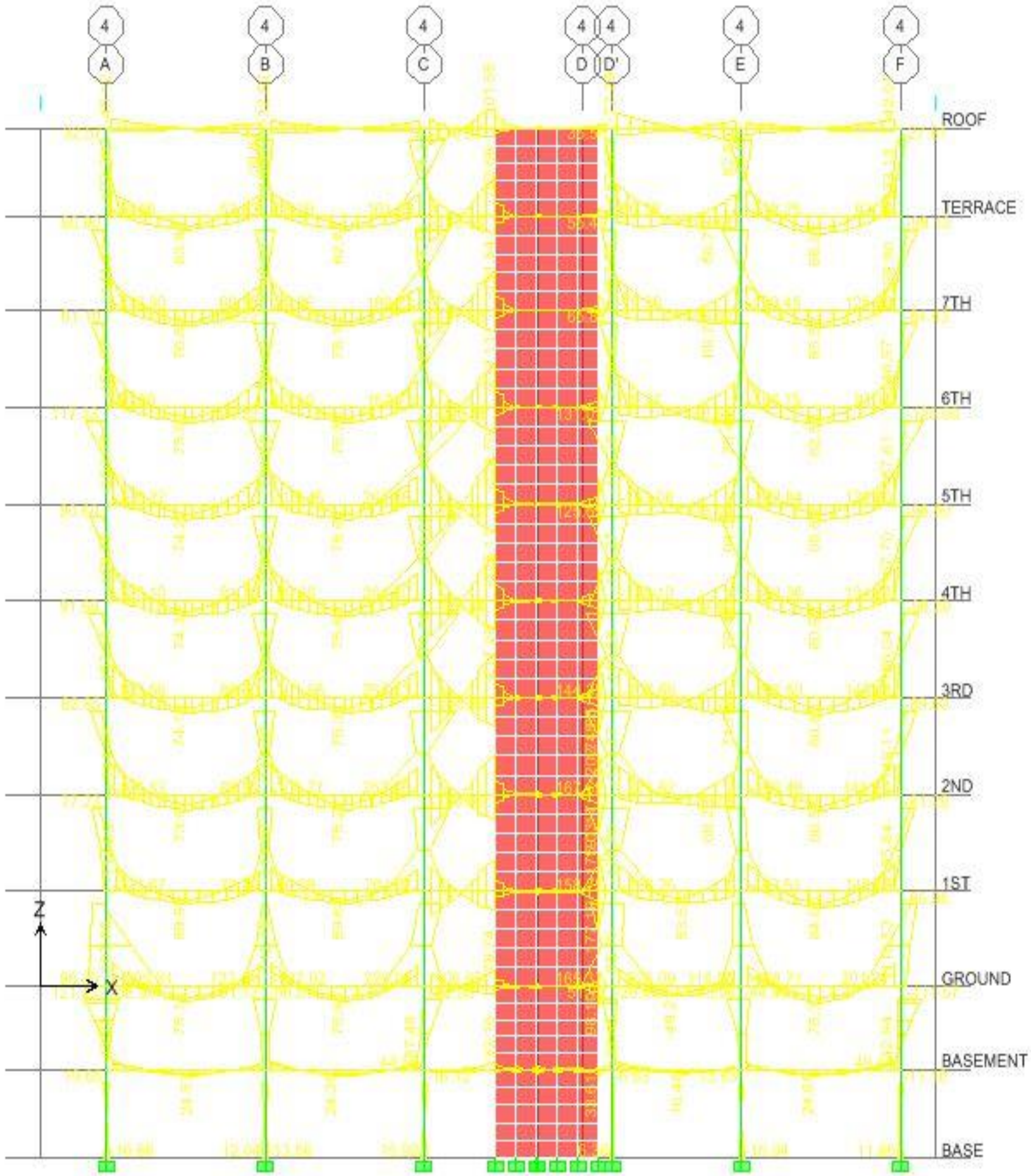
Bending Moment Diagram



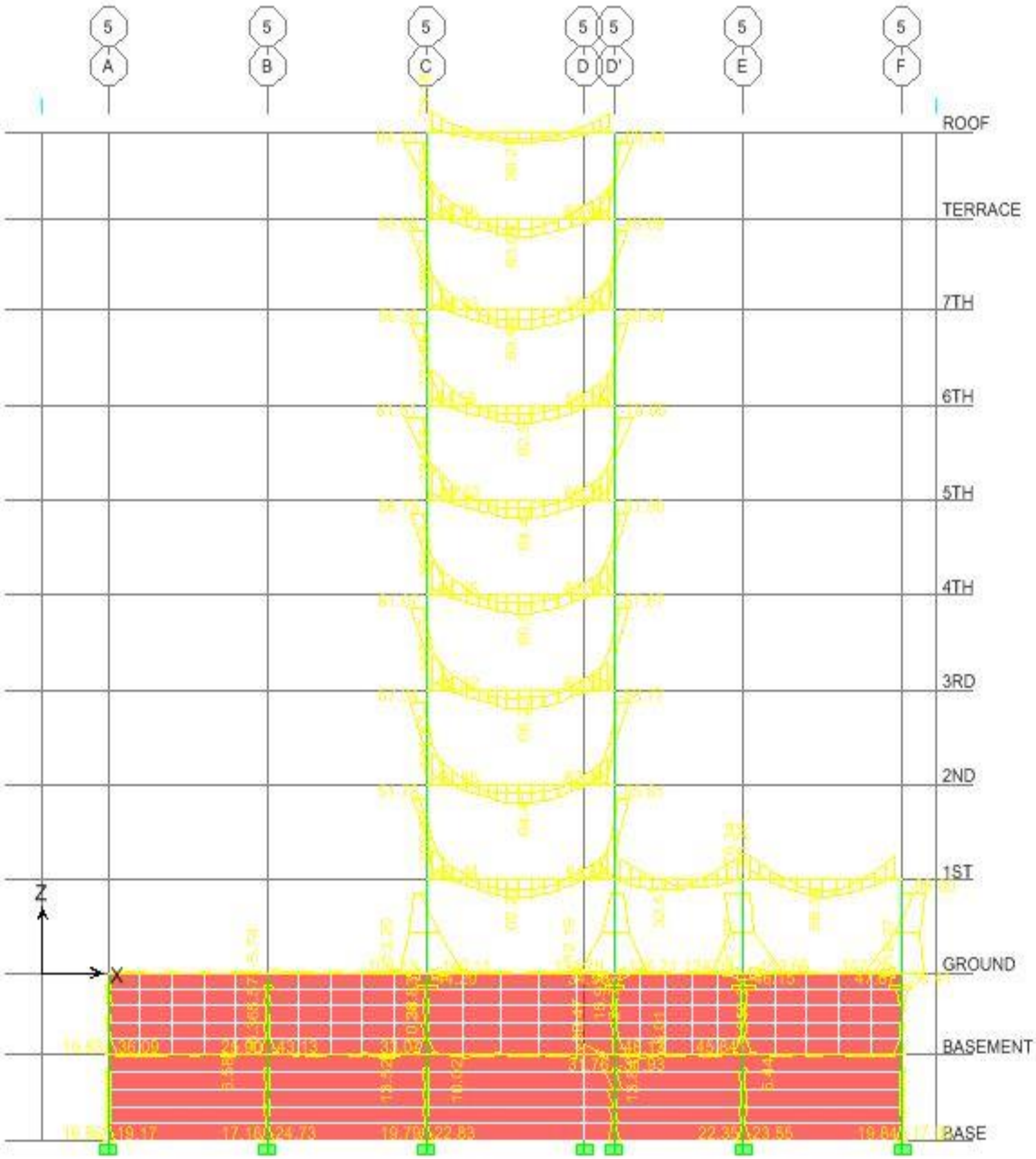


2
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

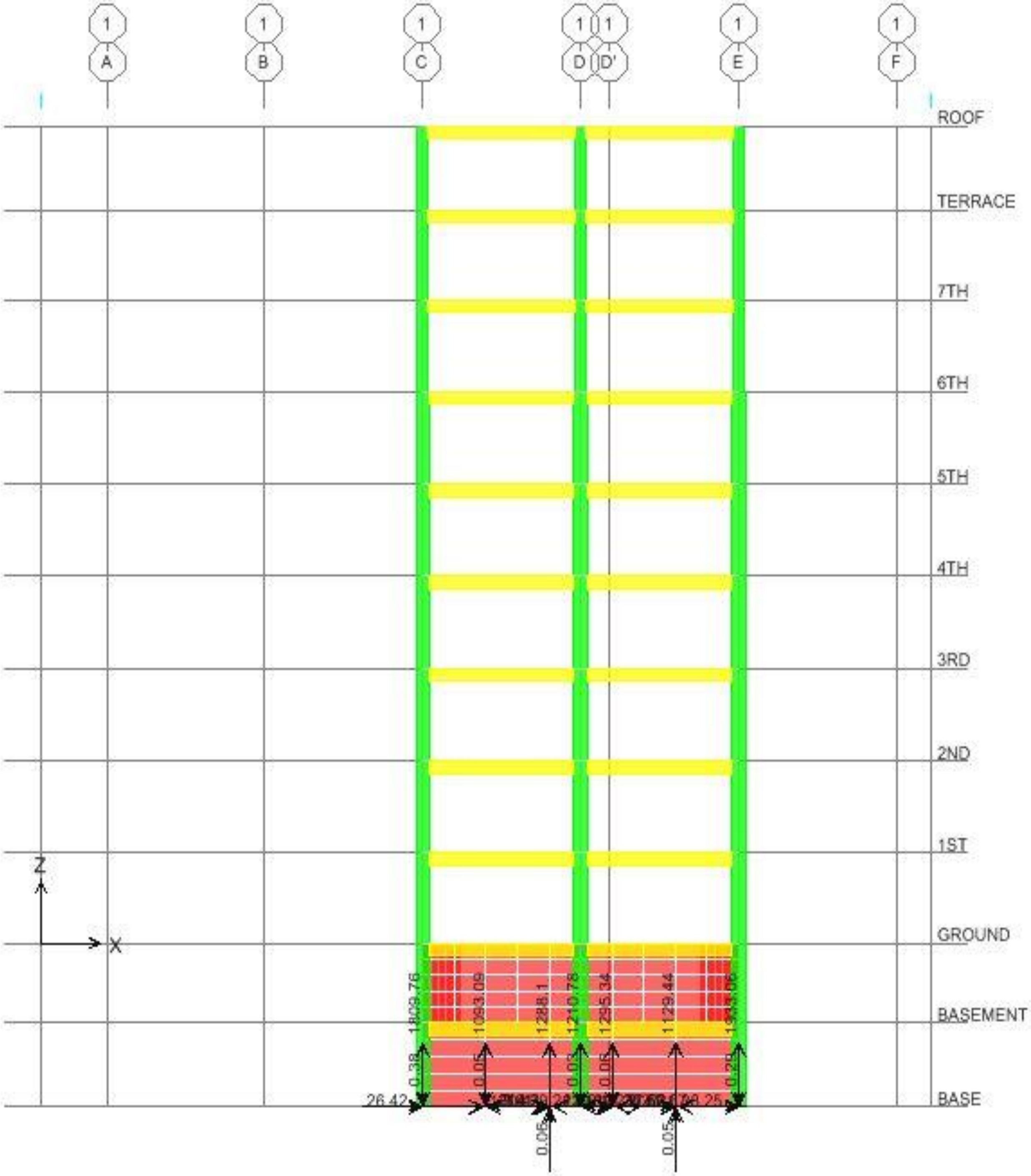

3
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

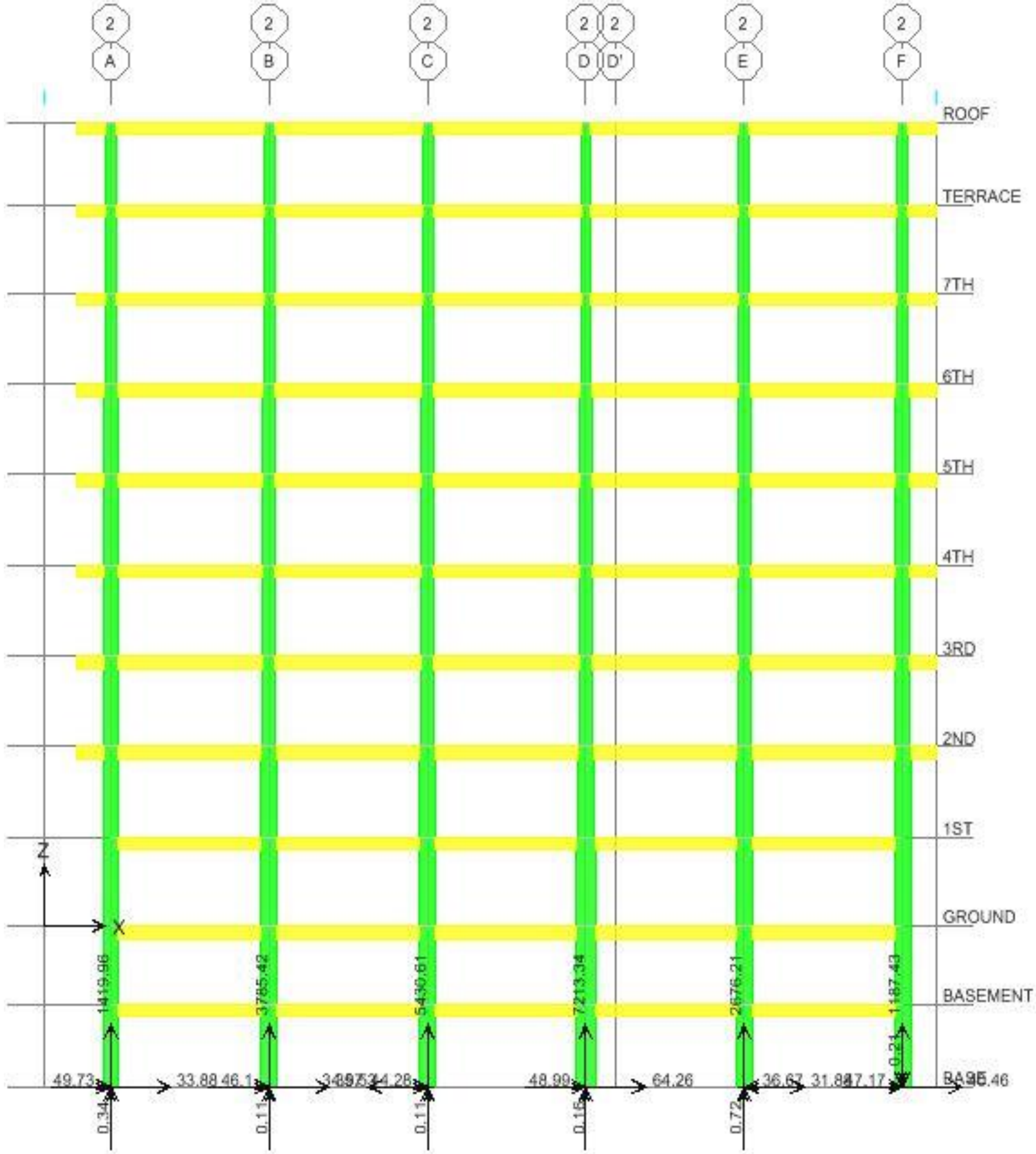


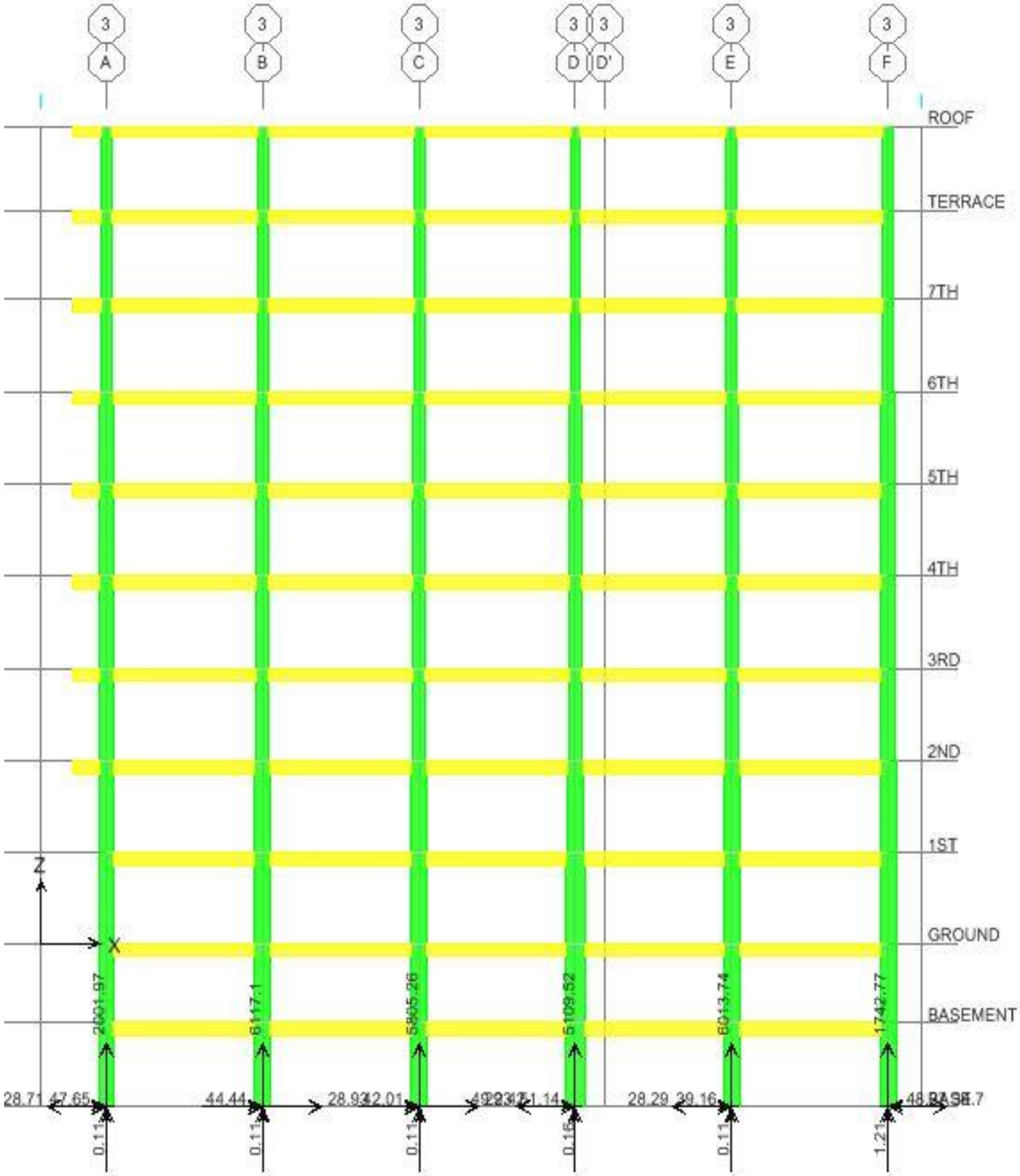
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X


5
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

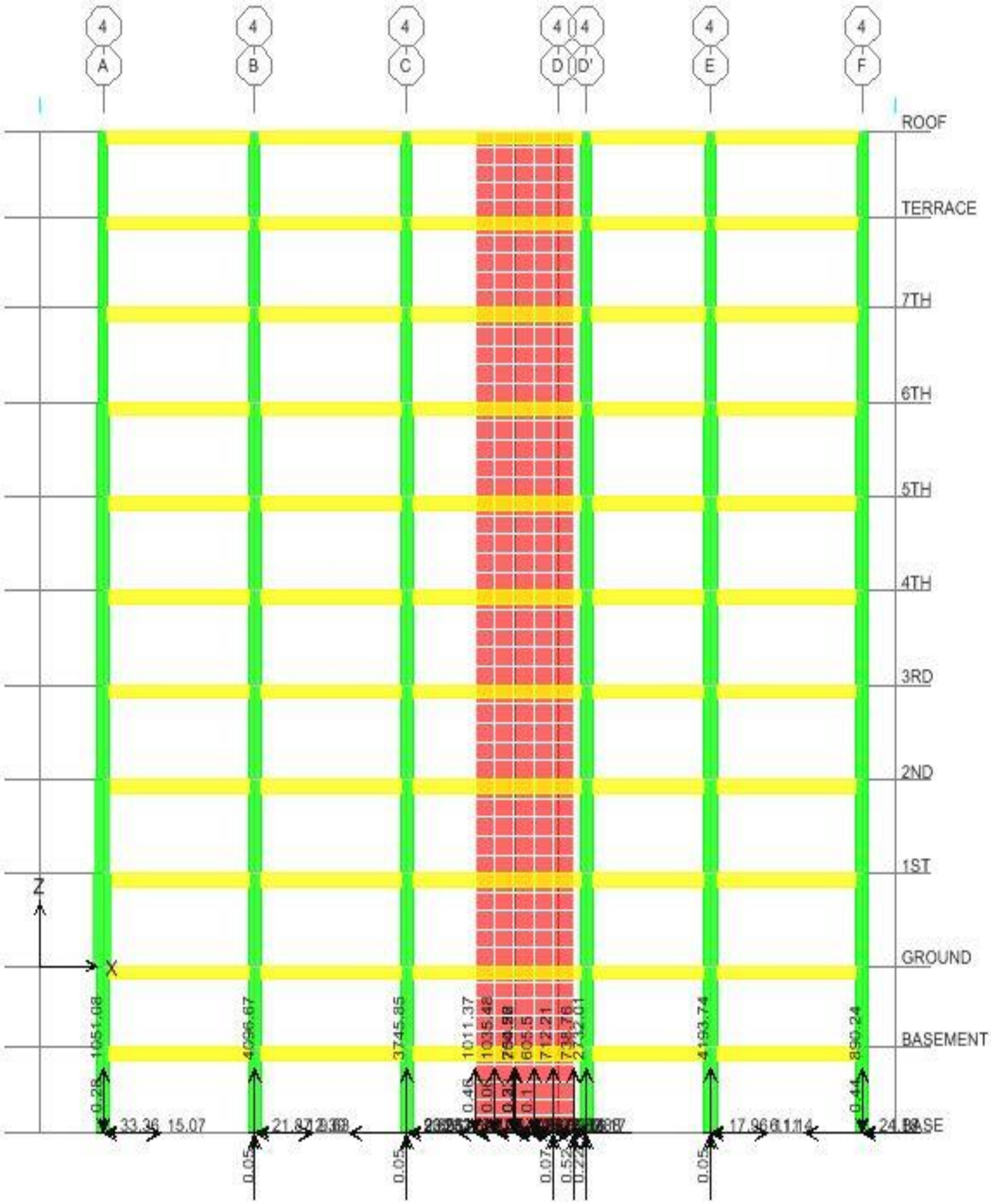
Footing Reaction

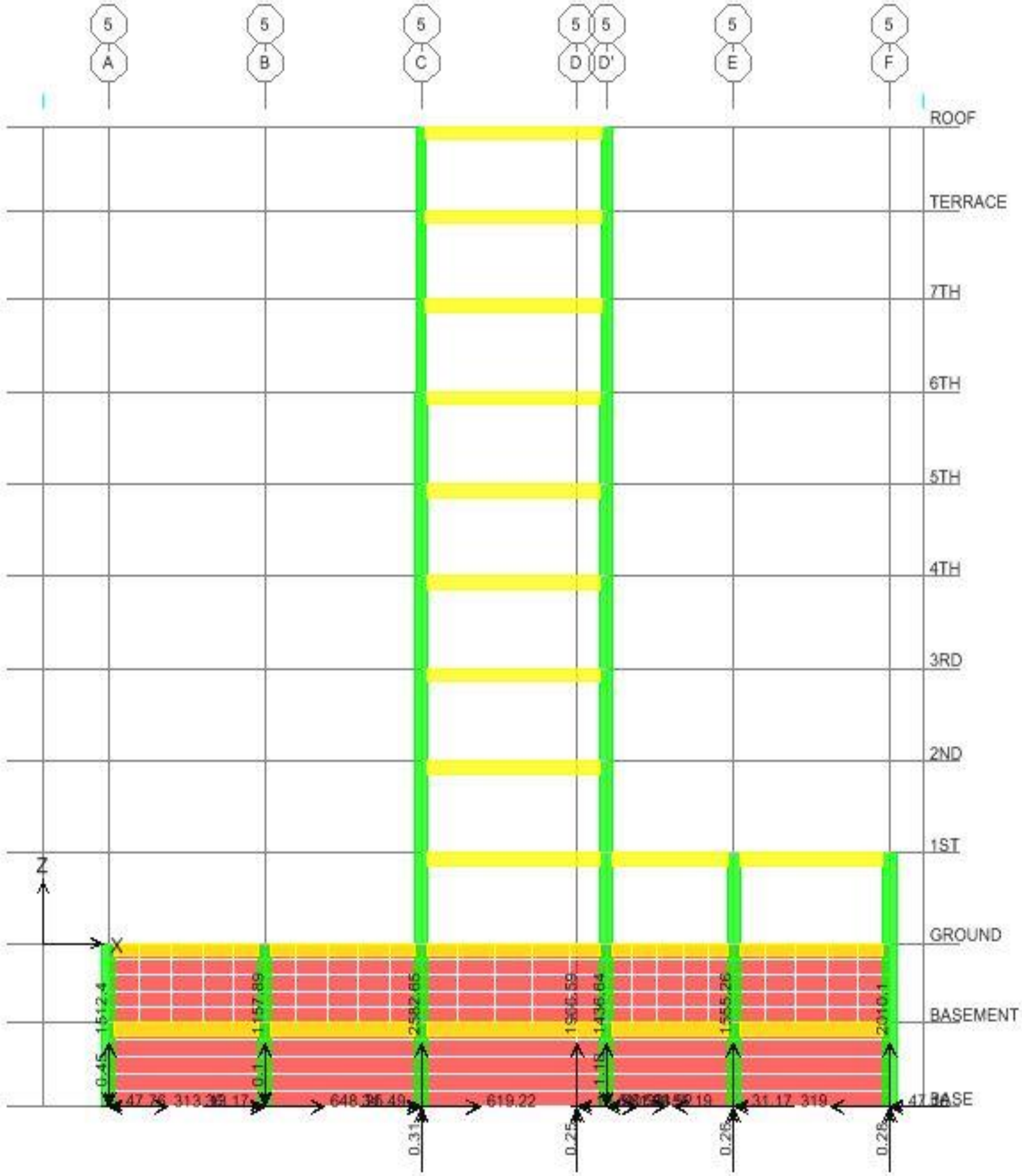



2
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X
1419.96
3785.42
5430.61
7213.34
2676.21
1187.43
49.73
33.88
46.1
48.99
64.26
36.67
31.8
47.17
0.34
0.11
0.11
0.15
0.72
0.21



3
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
Z
X
2001.97
6117.1
5805.26
5109.52
6013.74
1742.77
28.71
47.65
44.44
28.93
42.01
28.29
39.16
0.11
0.16
1.21


ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
A
B
C
D
D'
E
F
Z
X
1051.08
4096.67
3745.85
1011.37
1035.48
605.5
712.21
4193.74
890.24
0.28
0.46
0.44
33.36
15.07
0.05
0.07
0.52
0.22



ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE

Slab Design

Analysis and design of the slab system has been done in accordance with EBCS 2 Appendix A. Panels with similar spans and loading conditions were grouped as shown on the Slab design section of the structural calculations. The moments, shears and reactions of the panels were calculated using coefficients appropriate for the dimensions and support conditions of the slabs using Excel based spreadsheet.

Analysis and Design of Solid Slab For Ground Floor

F_{ck}	25.00	F_{cd}	11.33		0	
F_{yk}	300.00	F_{yd}	260.87			
Cover [mm]	15.00			1	S-1	1
β_a for 2:1	β_a for 1:1	Ratio calc.	β_a calc		1	
30.00	40.00	1.17	38.33			
Panel Name	S-1				D [mm]	
Depth	Lx [m]	Ly [m]	β_a calc	d [mm]	160.00	
	6.00	7.00	38.33	133.04	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			

Wall Load					
	Wall height				
	length		1.50	Wall Load	
Total DL			7.91	Total DL	
Total fact DL			10.28		
Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	?xs	?ys	?xf	?yf	?
	0.053	0.037	0.04	0.028	0.25
Moment	Mxs	Mys	Mxf	Myf	
	34.88	24.35	26.33	18.43	
k-value	1530	1070	1160	810	
?	0.63	0.43	0.47	0.32	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	955	650	706	486	
Asfinal [sq.mm]	955	650	706	486	
??					
8.00	22	16	16	12	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	50.00	70.00	70.00	100.00	
S.final [mm]	50.00	70.00	70.00	100.00	

⍰⍰					
10.00	14	10	11	8	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	80.00	120.00	110.00	160.00	
S.final [mm]	80.00	120.00	110.00	160.00	
	Load Transfer on Beams			Ly/Lx=	1.17
Shear coeffcient	⍰$_{vcx}$	⍰$_{vdx}$	⍰$_{vcy}$	⍰$_{vdy}$	
	0.39	0.26	0.36	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	42.63	28.42	39.49	N.A.	
Live Load	18.65	12.43	17.28	N.A.	
Dead Load	23.97	15.98	22.21	N.A.	

Note:- N.A.-Means Not Applicable

1 -Continuous

input data

0 -Discontinous

<u>Analysis and Design of Solid Slab For Ground Floor</u>

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			1	S-2	1
⍰a for 2:1	⍰a for 1:1	Ratio calc.	⍰a calc		1	
35.00	45.00	1.20	43.00			
Panel Name	S-2				D [mm]	
Depth	Lx [m]	Ly [m]	⍰a calc	d [mm]	140.00	
	5.00	6.00	43.00	116.28	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		

Total fact DL			10.28		
Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	□xs	□ys	□xf	□yf	□
	0.043	0.032	0.032	0.024	0.29
Moment	Mxs	Mys	Mxf	Myf	
	19.65	14.63	14.63	10.97	
k-value	870	650	650	490	
□	0.26	0.19	0.19	0.14	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	390	287	287	213	
Asfinal [sq.mm]	390	287	287	252	
□□					
8.00	10	7	7	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	120.00	170.00	170.00	190.00	
S.final [mm]	120.00	170.00	170.00	190.00	
□□					
10.00	7	5	5	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	200.00	270.00	270.00	310.00	

S.final [mm]	200.00	270.00	270.00	310.00	
	Load Transfer on Beams			Ly/Lx= 1.20	
Shear coeffcient	β_{vcx}	β_{vdx}	β_{vcy}	β_{vdy}	
	0.39	0.00	0.33	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	35.51	N.A.	30.17	N.A.	
Live Load	15.54	N.A.	13.20	N.A.	
Dead Load	19.97	N.A.	16.97	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

Analysis and Design of Solid Slab For Ground Floor

F_{ck}	25.00	F_{cd}	11.33		0	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			1	S-3	1
⍰$_a$ for 2:1	⍰$_a$ for 1:1	Ratio calc.	⍰$_a$ calc		1	
30.00	40.00	1.20	38.00			
Panel Name	S-3				D [mm]	
Depth	Lx [m]	Ly [m]	⍰$_a$ calc	d [mm]	140.00	
	5.00	6.00	38.00	111.84	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		

Total fact DL			10.28		
Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	□xs	□ys	□xf	□yf	□
	0.055	0.037	0.041	0.028	0.27
Moment	Mxs	Mys	Mxf	Myf	
	25.14	16.91	18.74	12.80	
k-value	1110	750	830	570	
□	0.33	0.22	0.25	0.17	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	505	333	371	250	
Asfinal [sq.mm]	505	333	371	252	
□□					
8.00	13	8	9	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	90.00	150.00	130.00	190.00	
S.final [mm]	90.00	150.00	130.00	190.00	
□□					
10.00	8	6	6	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	150.00	230.00	210.00	310.00	
S.final [mm]	150.00	230.00	210.00	310.00	

	Load Transfer on Beams			Ly/Lx= 1.20	
Shear coeffcient	⍰$_{vcx}$	⍰$_{vdx}$	⍰$_{vcy}$	⍰$_{vdy}$	
	0.40	0.27	0.36	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	36.57	24.38	32.91	N.A.	
Live Load	16.00	10.67	14.40	N.A.	
Dead Load	20.57	13.71	18.51	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

<u>Analysis and Design of Solid Slab For Ground Floor</u>

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			0	S-4	1
⍰$_a$ for 2:1	⍰$_a$ for 1:1	Ratio calc.	⍰$_a$ calc		0	
30.00	40.00	1.20	38.00			
Panel Name	S-4				D [mm]	
Depth	Lx [m]	Ly [m]	⍰$_a$ calc	d [mm]	160.00	
	5.00	6.00	38.00	131.58	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		

Total fact DL			10.28		
Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	□xs	□ys	□xf	□yf	□
	0.063	0.045	0.047	0.034	0.30
Moment	Mxs	Mys	Mxf	Myf	
	28.80	20.57	21.48	15.54	
k-value	1270	910	950	690	
□	0.39	0.27	0.28	0.20	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	583	409	428	305	
Asfinal [sq.mm]	583	409	428	305	
□□					
8.00	14	10	11	8	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	80.00	120.00	110.00	160.00	
S.final [mm]	80.00	120.00	110.00	160.00	
□□					
10.00	9	7	7	6	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	130.00	190.00	180.00	250.00	

S.final [mm]	130.00	190.00	180.00	250.00	
	Load Transfer on Beams			Ly/Lx=	1.20
Shear coeffcient	β_{vcx}	β_{vdx}	β_{vcy}	β_{vdy}	
	0.47	0.31	0.40	0.26	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	42.66	28.44	36.57	23.77	
Live Load	18.67	12.44	16.00	10.40	
Dead Load	23.99	16.00	20.57	13.37	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

<u>Analysis and Design of Solid Slab For Ground Floor</u>

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			1	S-5	0
⍰$_a$ for 2:1	⍰$_a$ for 1:1	Ratio calc.	⍰$_a$ calc		0	
30.00	40.00	1.62	33.78			
Panel Name	S-5				D [mm]	
Depth	Lx [m]	Ly [m]	⍰$_a$ calc	d [mm]	120.00	
	3.70	6.00	33.78	93.09	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			
Live Load	5		8.00		15.00	

Design load			18.28		
r values	r_1	r_2	r_3	r_4	n_d
	1.33	0.00	1.33	0.00	2.00
Alpha values	□xs	□ys	□xf	□yf	□
	0.083	0.045	0.062	0.034	0.40
Moment	Mxs	Mys	Mxf	Myf	
	20.77	11.26	15.52	8.51	
k-value	920	500	690	380	
□	0.27	0.15	0.20	0.11	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	413	219	305	165	
Asfinal [sq.mm]	413	252	305	252	
□□					
8.00	10	7	8	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	120.00	190.00	160.00	190.00	
S.final [mm]	120.00	190.00	160.00	190.00	
□□					
10.00	7	5	6	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	190.00	310.00	250.00	310.00	
S.final [mm]	190.00	310.00	250.00	310.00	

	Load Transfer on Beams			Ly/Lx= 1.62	
Shear coeffcient	⍰$_{vcx}$	⍰$_{vdx}$	⍰$_{vcy}$	⍰$_{vdy}$	
	0.55	0.37	0.40	0.26	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	37.43	24.95	27.06	17.59	
Live Load	16.38	10.92	11.84	7.70	
Dead Load	21.05	14.04	15.22	9.89	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

Analysis and Design of Solid Slab For Ground Floor

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			0	S-6	1
⍰a for 2:1	⍰a for 1:1	Ratio calc.	⍰a calc		0	
30.00	40.00	1.32	36.76			
Panel Name	S-6				D [mm]	
Depth	Lx [m]	Ly [m]	⍰a calc	d [mm]	120.00	
	3.70	4.90	36.76	100.66	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		

Total fact DL			10.28		
Live Load	5		8.00		15.00
Design load			18.28		
r values	r_1	r_2	r_3	r_4	n_d
	0.00	1.33	1.33	0.00	2.00
Alpha values	$\square_{xs}$	$\square_{ys}$	$\square_{xf}$	$\square_{yf}$	$\square$
	0.069	0.045	0.052	0.034	0.33
Moment	Mxs	Mys	Mxf	Myf	
	17.27	11.26	13.02	8.51	
k-value	760	500	580	380	
$\square$	0.23	0.15	0.17	0.11	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	341	219	254	165	
Asfinal [sq.mm]	341	252	254	252	
$\square\square$					
8.00	9	7	7	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	140.00	190.00	190.00	190.00	
S.final [mm]	140.00	190.00	190.00	190.00	
$\square\square$					
10.00	6	5	5	5	# of bars

S.max [mm]	340	340	340	340	
S.calc[mm]	230.00	310.00	300.00	310.00	
S.final [mm]	230.00	310.00	300.00	310.00	
	Load Transfer on Beams			Ly/Lx= 1.32	
Shear coeffcient	β_{vcx}	β_{vdx}	β_{vcy}	β_{vdy}	
	0.50	0.33	0.40	0.26	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	33.69	22.46	27.06	17.59	
Live Load	14.74	9.83	11.84	7.70	
Dead Load	18.95	12.63	15.22	9.89	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

Analysis and Design of Solid Slab For 1st Floor

F_{ck}	25.00	F_{cd}	11.33		0	
F_{yk}	300.00	F_{yd}	260.87			
Cover [mm]	15.00			1	S-1	1
$\square_a$ for 2:1	$\square_a$ for 1:1	Ratio calc.	$\square_a$ calc		1	
30.00	40.00	1.17	38.33			
Panel Name	S-1				D [mm]	
Depth	Lx [m]	Ly [m]	$\square_a$ calc	d [mm]	160.00	
	6.00	7.00	38.33	133.04	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			

Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	□xs	□ys	□xf	□yf	□
	0.053	0.037	0.04	0.028	0.25
Moment	Mxs	Mys	Mxf	Myf	
	34.88	24.35	26.33	18.43	
k-value	1530	1070	1160	810	
□	0.63	0.43	0.47	0.32	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	955	650	706	486	
Asfinal [sq.mm]	955	650	706	486	
□□					
8.00	22	16	16	12	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	50.00	70.00	70.00	100.00	
S.final [mm]	50.00	70.00	70.00	100.00	
□□					
10.00	14	10	11	8	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	80.00	120.00	110.00	160.00	
S.final [mm]	80.00	120.00	110.00	160.00	
	Load Transfer on Beams			Ly/Lx=	1.17

Shear coeffcient	⍰$_{vcx}$	⍰$_{vdx}$	⍰$_{vcy}$	⍰$_{vdy}$	
	0.39	0.26	0.36	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	42.63	28.42	39.49	N.A.	
Live Load	18.65	12.43	17.28	N.A.	
Dead Load	23.97	15.98	22.21	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83		S-2	
Cover [mm]	15.00			1		1
□$_a$ for 2:1	□$_a$ for 1:1	Ratio calc.	□$_a$ calc		1	
35.00	45.00	1.20	43.00			
Panel Name	S-2				D [mm]	
Depth	Lx [m]	Ly [m]	□$_a$ calc	d [mm]	140.00	
	5.00	6.00	43.00	116.28	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			

Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	□$_{xs}$	□$_{ys}$	□$_{xf}$	□$_{yf}$	□
	0.043	0.032	0.032	0.024	0.29
Moment	Mxs	Mys	Mxf	Myf	
	19.65	14.63	14.63	10.97	
k-value	870	650	650	490	
□	0.26	0.19	0.19	0.14	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	390	287	287	213	
Asfinal [sq.mm]	390	287	287	252	
□□					
8.00	10	7	7	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	120.00	170.00	170.00	190.00	
S.final [mm]	120.00	170.00	170.00	190.00	
□□					
10.00	7	5	5	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	200.00	270.00	270.00	310.00	
S.final [mm]	200.00	270.00	270.00	310.00	

	Load Transfer on Beams			Ly/Lx=	1.20
Shear coeffcient	$?_{vcx}$	$?_{vdx}$	$?_{vcy}$	$?_{vdy}$	
	0.39	0.00	0.33	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	35.51	N.A.	30.17	N.A.	
Live Load	15.54	N.A.	13.20	N.A.	
Dead Load	19.97	N.A.	16.97	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

F_{ck}	25.00	F_{cd}	11.33		0	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			1	S-3	1
⍰$_a$ for 2:1	⍰$_a$ for 1:1	Ratio calc.	⍰$_a$ calc		1	
30.00	40.00	1.20	38.00			
Panel Name	S-3				D [mm]	
Depth	Lx [m]	Ly [m]	⍰$_a$ calc	d [mm]	140.00	
	5.00	6.00	38.00	111.84	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			

Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	⍰$_{xs}$	⍰$_{ys}$	⍰$_{xf}$	⍰$_{yf}$	⍰
	0.055	0.037	0.041	0.028	0.27
Moment	Mxs	Mys	Mxf	Myf	
	25.14	16.91	18.74	12.80	
k-value	1110	750	830	570	
⍰	0.33	0.22	0.25	0.17	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	505	333	371	250	
Asfinal [sq.mm]	505	333	371	252	
⍰⍰					
8.00	13	8	9	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	90.00	150.00	130.00	190.00	
S.final [mm]	90.00	150.00	130.00	190.00	
⍰⍰					
10.00	8	6	6	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	150.00	230.00	210.00	310.00	
S.final [mm]	150.00	230.00	210.00	310.00	
	Load Transfer on Beams			Ly/Lx=	1.20

Shear coeffcient	β_{vcx}	β_{vdx}	β_{vcy}	β_{vdy}	
	0.40	0.27	0.36	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	36.57	24.38	32.91	N.A.	
Live Load	16.00	10.67	14.40	N.A.	
Dead Load	20.57	13.71	18.51	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			0	S-4	1
$\square_a$ for 2:1	$\square_a$ for 1:1	Ratio calc.	$\square_a$ calc		0	
30.00	40.00	1.20	38.00			
Panel Name	S-4				D [mm]	
Depth	Lx [m]	Ly [m]	$\square_a$ calc	d [mm]	160.00	
	5.00	6.00	38.00	131.58	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			

Live Load	5		8.00		15.00
Design load			18.28		
Alpha values	□xs	□ys	□xf	□yf	□
	0.063	0.045	0.047	0.034	0.30
Moment	Mxs	Mys	Mxf	Myf	
	28.80	20.57	21.48	15.54	
k-value	1270	910	950	690	
□	0.39	0.27	0.28	0.20	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	583	409	428	305	
Asfinal [sq.mm]	583	409	428	305	
□□					
8.00	14	10	11	8	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	80.00	120.00	110.00	160.00	
S.final [mm]	80.00	120.00	110.00	160.00	
□□					
10.00	9	7	7	6	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	130.00	190.00	180.00	250.00	
S.final [mm]	130.00	190.00	180.00	250.00	

	Load Transfer on Beams			Ly/Lx= 1.20	
Shear coeffcient	β_{vcx}	β_{vdx}	β_{vcy}	β_{vdy}	
	0.47	0.31	0.40	0.26	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	42.66	28.44	36.57	23.77	
Live Load	18.67	12.44	16.00	10.40	
Dead Load	23.99	16.00	20.57	13.37	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

F_{ck}	25.00	F_{cd}	11.33		0	
F_{yk}	300.00	F_{yd}	260.87			
Cover [mm]	15.00			1	S-1	1
$\square_a$ for 2:1	$\square_a$ for 1:1	Ratio calc.	$\square_a$ calc		1	
30.00	40.00	1.17	38.33			
Panel Name	S-1				D [mm]	
Depth	Lx [m]	Ly [m]	$\square_a$ calc	d [mm]	160.00	
	6.00	7.00	38.33	133.04	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			

Live Load	3		4.80		15.00
Design load			15.08		
Alpha values	□xs	□ys	□xf	□yf	□
	0.053	0.037	0.04	0.028	0.25
Moment	Mxs	Mys	Mxf	Myf	
	28.78	20.09	21.72	15.20	
k-value	1270	890	960	670	
□	0.51	0.35	0.38	0.26	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	777	532	577	398	
Asfinal [sq.mm]	777	532	577	398	
□□					
8.00	18	13	14	10	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	60.00	90.00	80.00	120.00	
S.final [mm]	60.00	90.00	80.00	120.00	
□□					
10.00	12	9	9	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	100.00	140.00	130.00	190.00	
S.final [mm]	100.00	140.00	130.00	190.00	
	Load Transfer on Beams			Ly/Lx=	1.17

Shear coeffcient	$\square_{vcx}$	$\square_{vdx}$	$\square_{vcy}$	$\square_{vdy}$	
	0.39	0.26	0.36	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	35.16	23.44	32.58	N.A.	
Live Load	11.19	7.46	10.37	N.A.	
Dead Load	23.97	15.98	22.21	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			1	S-2	1
$?_a$ for 2:1	$?_a$ for 1:1	Ratio calc.	$?_a$ calc		1	
35.00	45.00	1.20	43.00			
Panel Name	S-2				D [mm]	
Depth	Lx [m]	Ly [m]	$?_a$ calc	d [mm]	140.00	
	5.00	6.00	43.00	116.28	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			

Live Load	3		4.80		15.00
Design load			15.08		
Alpha values	□xs	□ys	□xf	□yf	□
	0.043	0.032	0.032	0.024	0.29
Moment	Mxs	Mys	Mxf	Myf	
	16.21	12.07	12.07	9.05	
k-value	720	530	530	400	
□	0.21	0.16	0.16	0.12	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	319	235	235	175	
Asfinal [sq.mm]	319	252	252	252	
□□					
8.00	8	7	7	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	150.00	190.00	190.00	190.00	
S.final [mm]	150.00	190.00	190.00	190.00	
□□					
10.00	6	5	5	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	240.00	310.00	310.00	310.00	
S.final [mm]	240.00	310.00	310.00	310.00	

	Load Transfer on Beams			Ly/Lx= 1.20	
Shear coeffcient	⍰$_{vcx}$	⍰$_{vdx}$	⍰$_{vcy}$	⍰$_{vdy}$	
	0.39	0.00	0.33	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	29.30	N.A.	24.89	N.A.	
Live Load	9.32	N.A.	7.92	N.A.	
Dead Load	19.97	N.A.	16.97	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

Analysis and Design of Solid Slab (2nd - 7th Floor)

F_{ck}	25.00	F_{cd}	11.33		0	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			1	S-3	1
⍰$_a$ for 2:1	⍰$_a$ for 1:1	Ratio calc.	⍰$_a$ calc		1	
30.00	40.00	1.20	38.00			
Panel Name	S-3				D [mm]	
Depth	Lx [m]	Ly [m]	⍰$_a$ calc	d [mm]	140.00	
	5.00	6.00	38.00	111.84	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			
Live Load	3		4.80		15.00	

Design load			15.08		
Alpha values	$\square_{xs}$	$\square_{ys}$	$\square_{xf}$	$\square_{yf}$	□
	0.055	0.037	0.041	0.028	0.27
Moment	Mxs	Mys	Mxf	Myf	
	20.74	13.95	15.46	10.56	
k-value	910	620	680	470	
□	0.27	0.18	0.20	0.14	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	412	273	304	205	
Asfinal [sq.mm]	412	273	304	252	
□□					
8.00	10	7	8	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	120.00	180.00	160.00	190.00	
S.final [mm]	120.00	180.00	160.00	190.00	
□□					
10.00	7	5	6	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	190.00	280.00	250.00	310.00	
S.final [mm]	190.00	280.00	250.00	310.00	
	Load Transfer on Beams			Ly/Lx=	1.20
Shear	$\square_{vcx}$	$\square_{vdx}$	$\square_{vcy}$	$\square_{vdy}$	

coeffcient					
	0.40	0.27	0.36	0.00	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	30.17	20.11	27.15	N.A.	
Live Load	9.60	6.40	8.64	N.A.	
Dead Load	20.57	13.71	18.51	N.A.	

Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

Analysis and Design of Solid Slab (2nd - 7th Floor)

F_{ck}	25.00	F_{cd}	11.33		1	
F_{yk}	400.00	F_{yd}	347.83			
Cover [mm]	15.00			0	S-4	1
⍰$_a$ for 2:1	⍰$_a$ for 1:1	Ratio calc.	⍰$_a$ calc		0	
30.00	40.00	1.20	38.00			
Panel Name	S-4				D [mm]	
Depth	Lx [m]	Ly [m]	⍰$_a$ calc	d [mm]	160.00	
	5.00	6.00	38.00	131.58	170.00	
	thickness t	unit weight	load			
DL	0.17	25.00	4.25	SF DL	1.30	
Floor finish	0.02	16.00	0.32	SF LL	1.60	
Ceiling Plaster	0.03	23.00	0.69			
5cm Screed	0.05	23.00	1.15			
Wall Load						
	Wall height					
	length		1.50	Wall Load		
Total DL			7.91	Total DL		
Total fact DL			10.28			
Live Load	3		4.80		15.00	

Design load			15.08		
Alpha values	□xs	□ys	□xf	□yf	□
	0.063	0.045	0.047	0.034	0.30
Moment	Mxs	Mys	Mxf	Myf	
	23.76	16.97	17.72	12.82	
k-value	1050	750	780	570	
□	0.31	0.22	0.23	0.17	
Amin[sq.mm]	252	252	252	252	
Ascal [sq.mm]	475	334	350	250	
Asfinal [sq.mm]	475	334	350	252	
□□					
8.00	12	8	9	7	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	100.00	150.00	140.00	190.00	
S.final [mm]	100.00	150.00	140.00	190.00	
□□					
10.00	8	6	6	5	# of bars
S.max [mm]	340	340	340	340	
S.calc[mm]	160.00	230.00	220.00	310.00	
S.final [mm]	160.00	230.00	220.00	310.00	
	Load Transfer on Beams		Ly/Lx= 1.20		

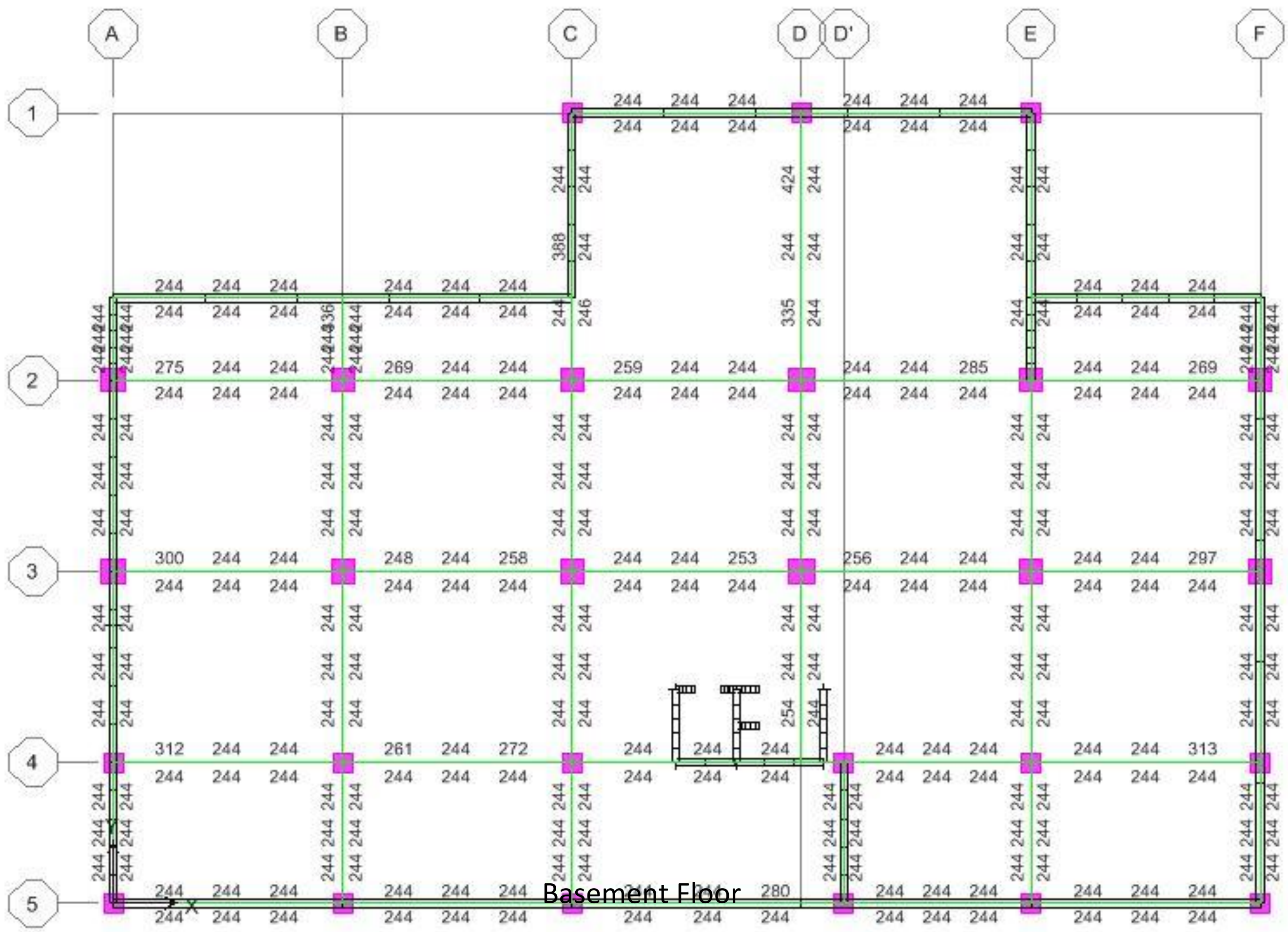
Shear coeffcient	$\boxed{?}_{vcx}$	$\boxed{?}_{vdx}$	$\boxed{?}_{vcy}$	$\boxed{?}_{vdy}$	
	0.47	0.31	0.40	0.26	
Load on Beams	R_{cx}	R_{dx}	R_{cy}	R_{dy}	
Total design load	35.19	23.46	30.17	19.61	
Live Load	11.20	7.47	9.60	6.24	
Dead Load	23.99	16.00	20.57	13.37	

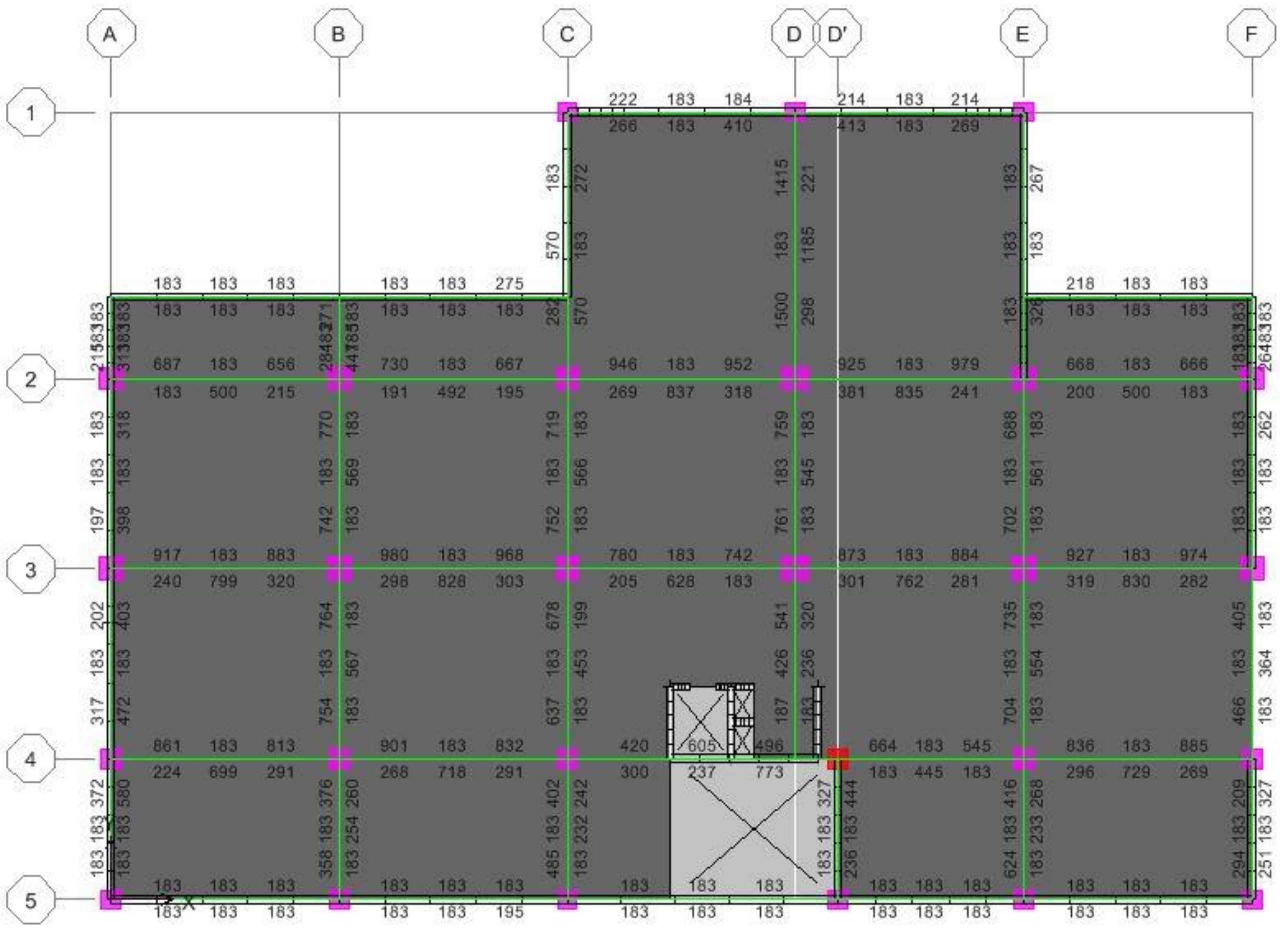
Note:- N.A.-Means Not Applicable 1 -Continuous

input data 0 -Discontinous

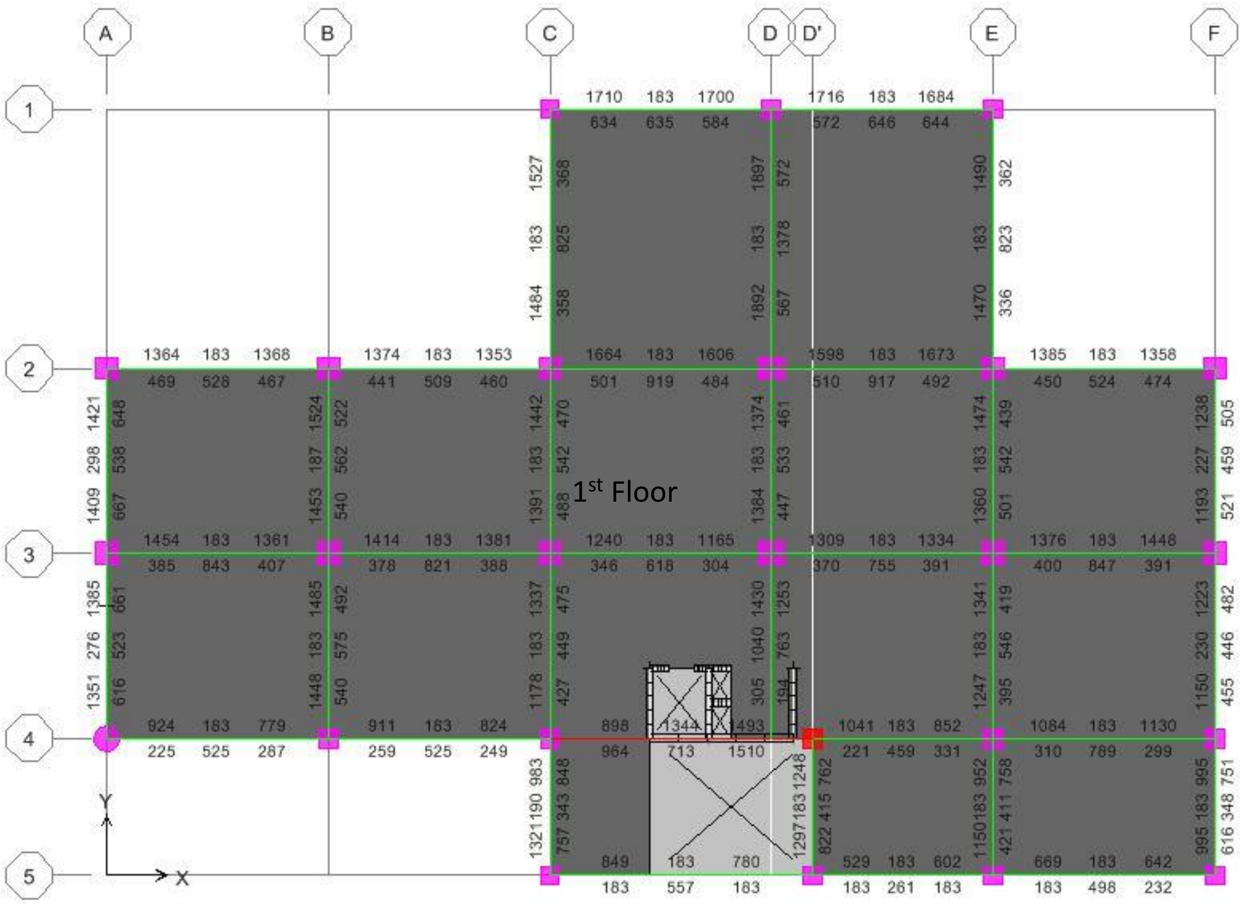
Beam Design

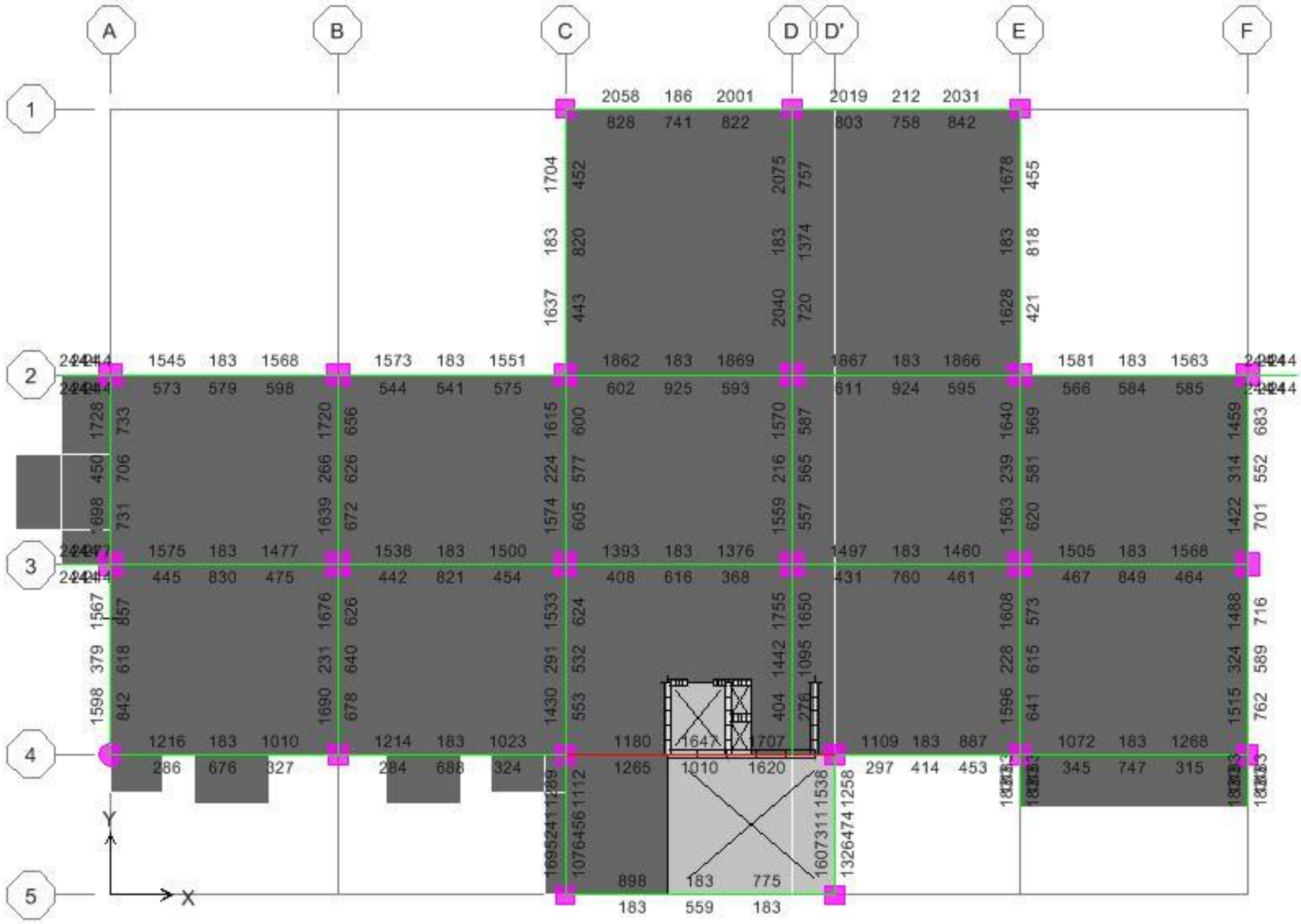
The force envelope i.e. maximum positive and negative moments, and maximum shear (envelope) is automatically selected by the ETABS software. The beam reinforcements are designed to resist these loads.



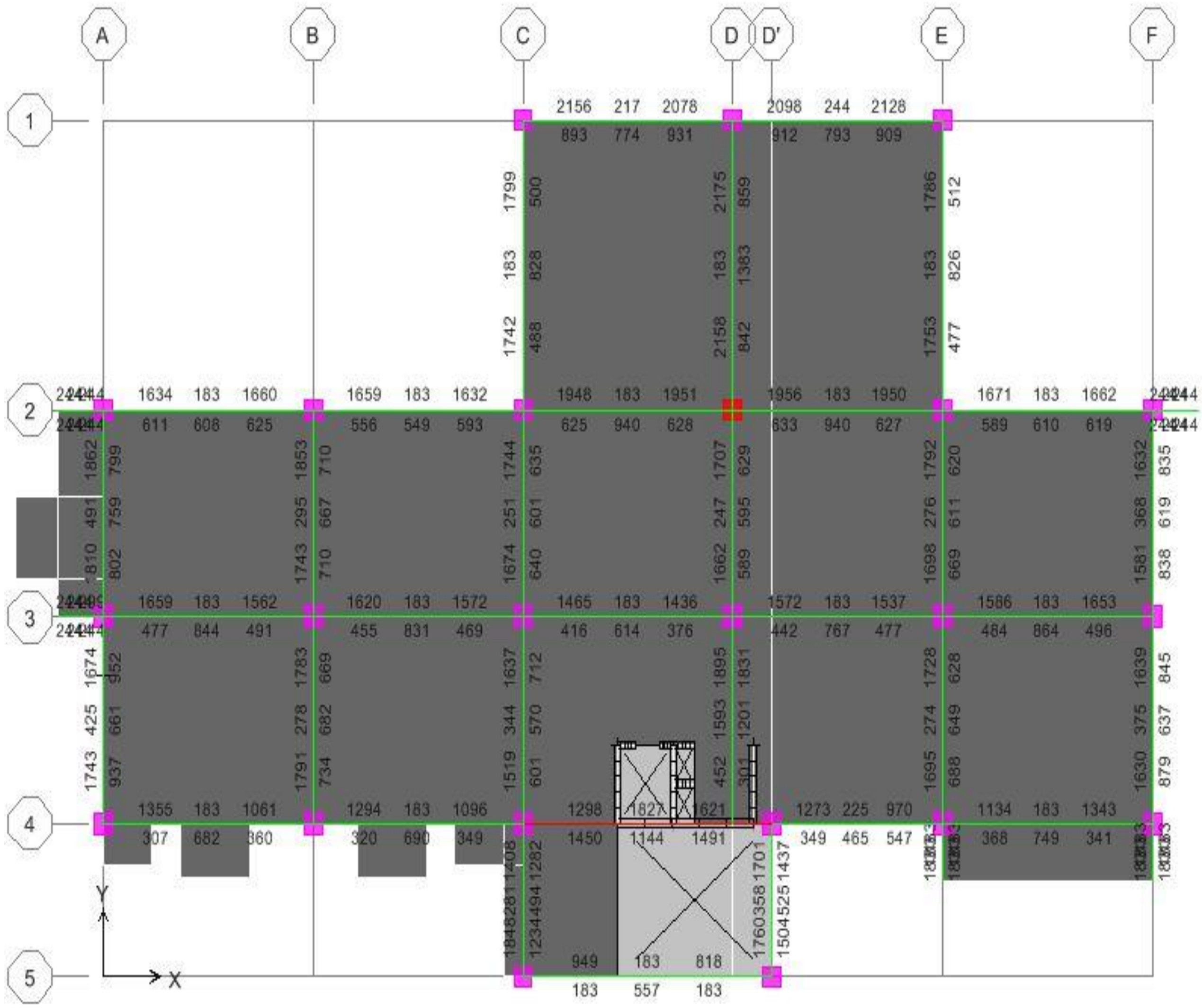


Ground Floor


A
B
C
D
D'
E
F
1
2
3
4
5
1st Floor
Y
X

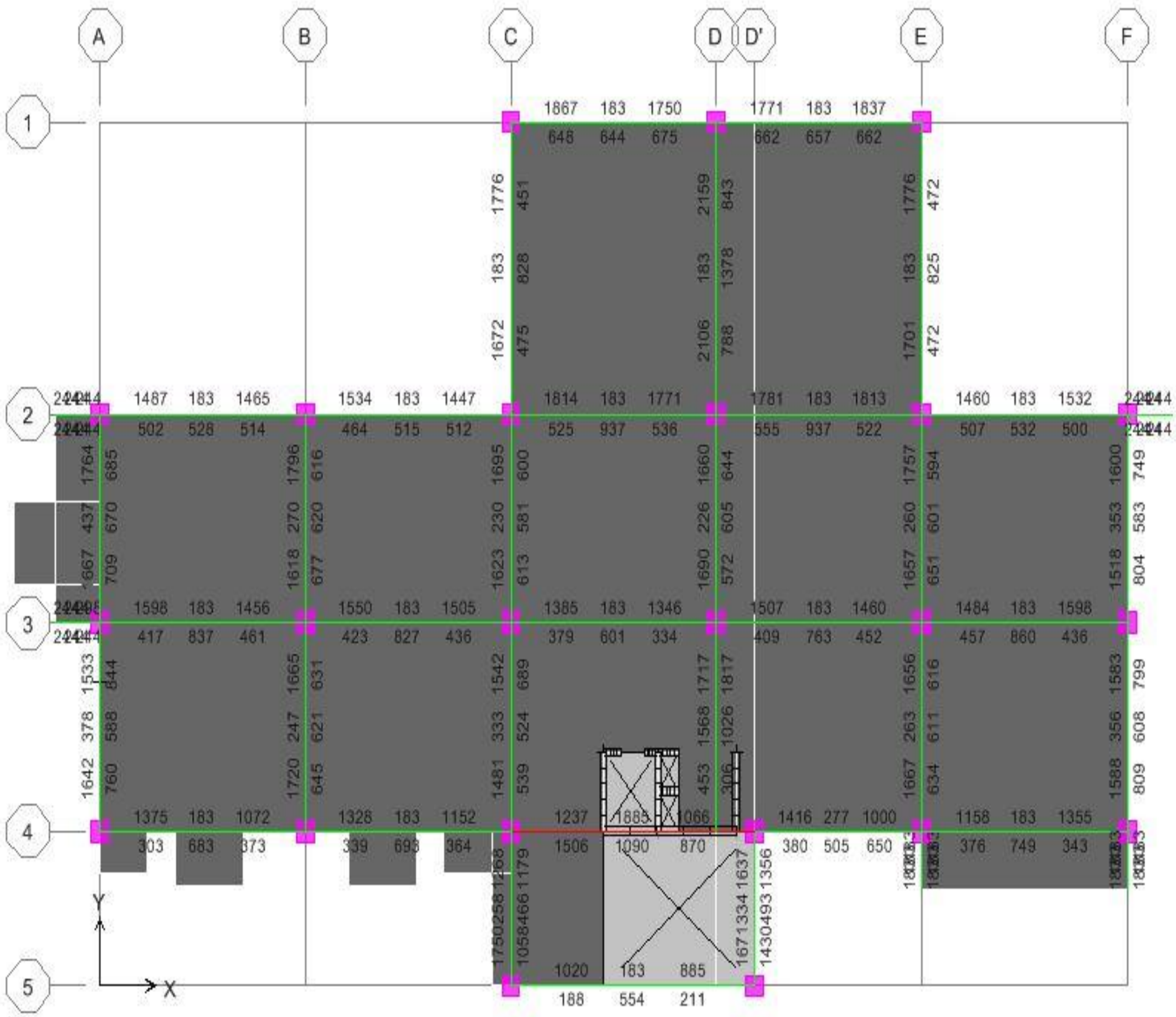


2nd Floor


A
B
C
D
D'
E
F
1
2
3
4
5
X
Y

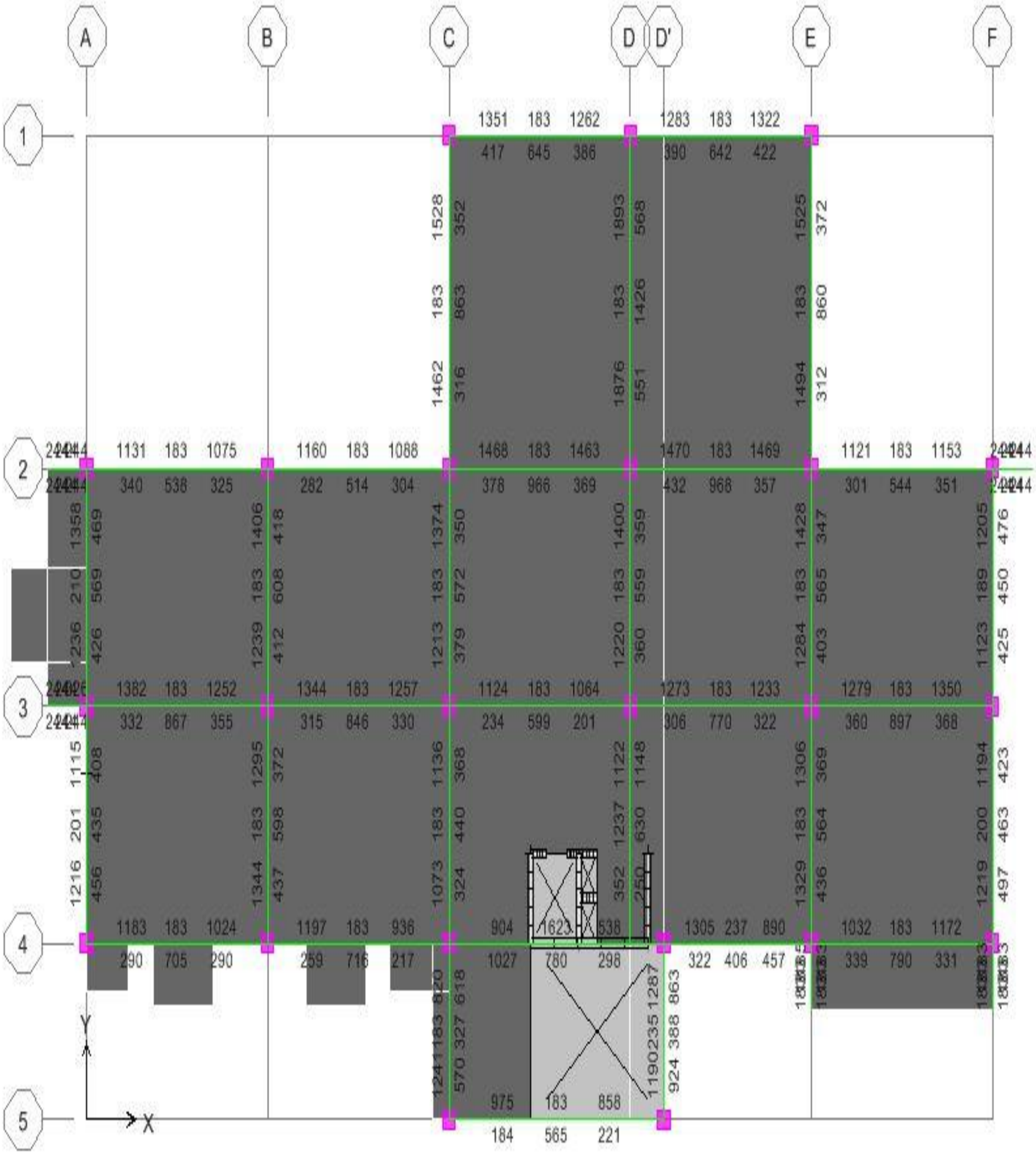
3rd Floor

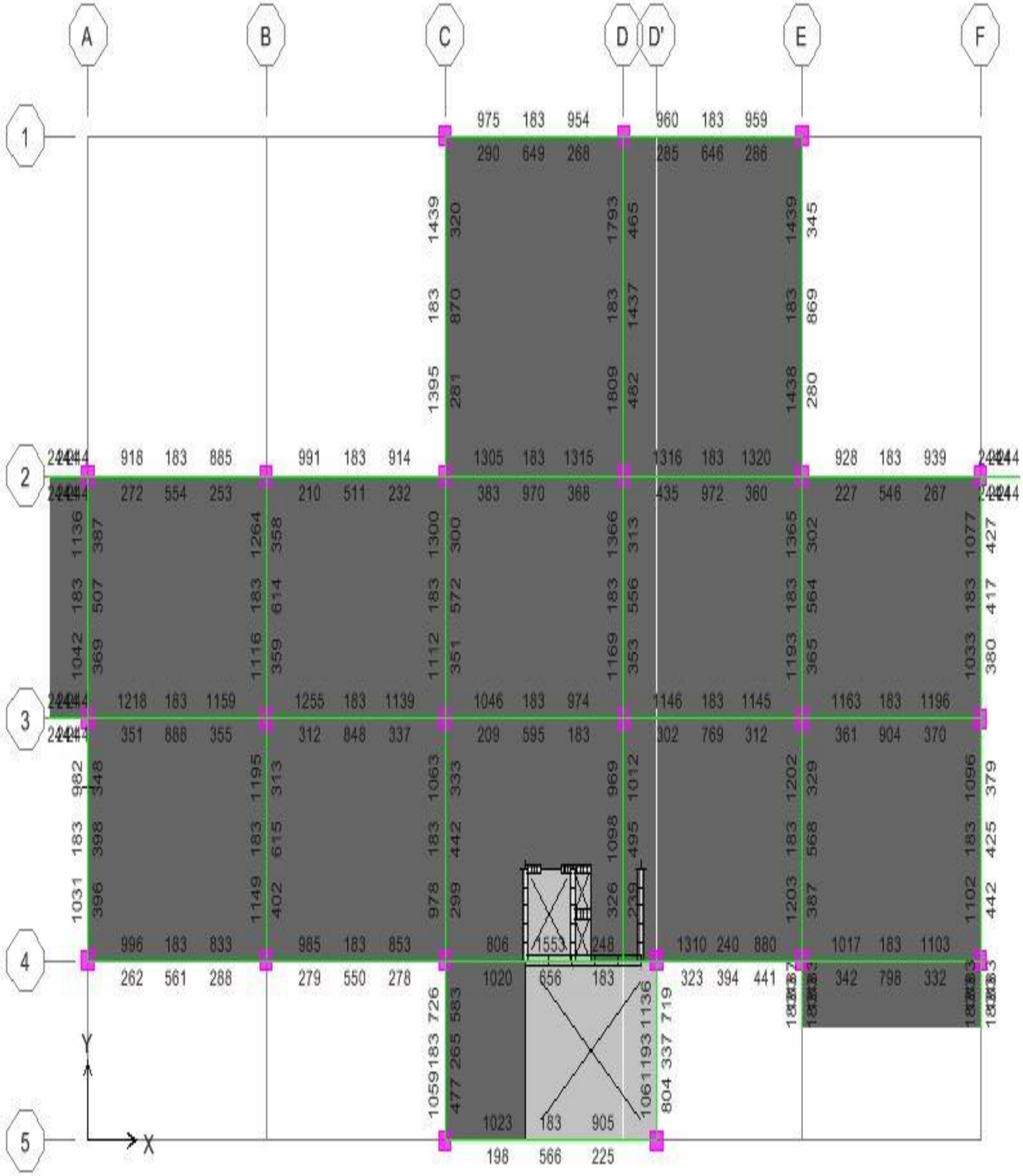
4th Floor



5th Floor

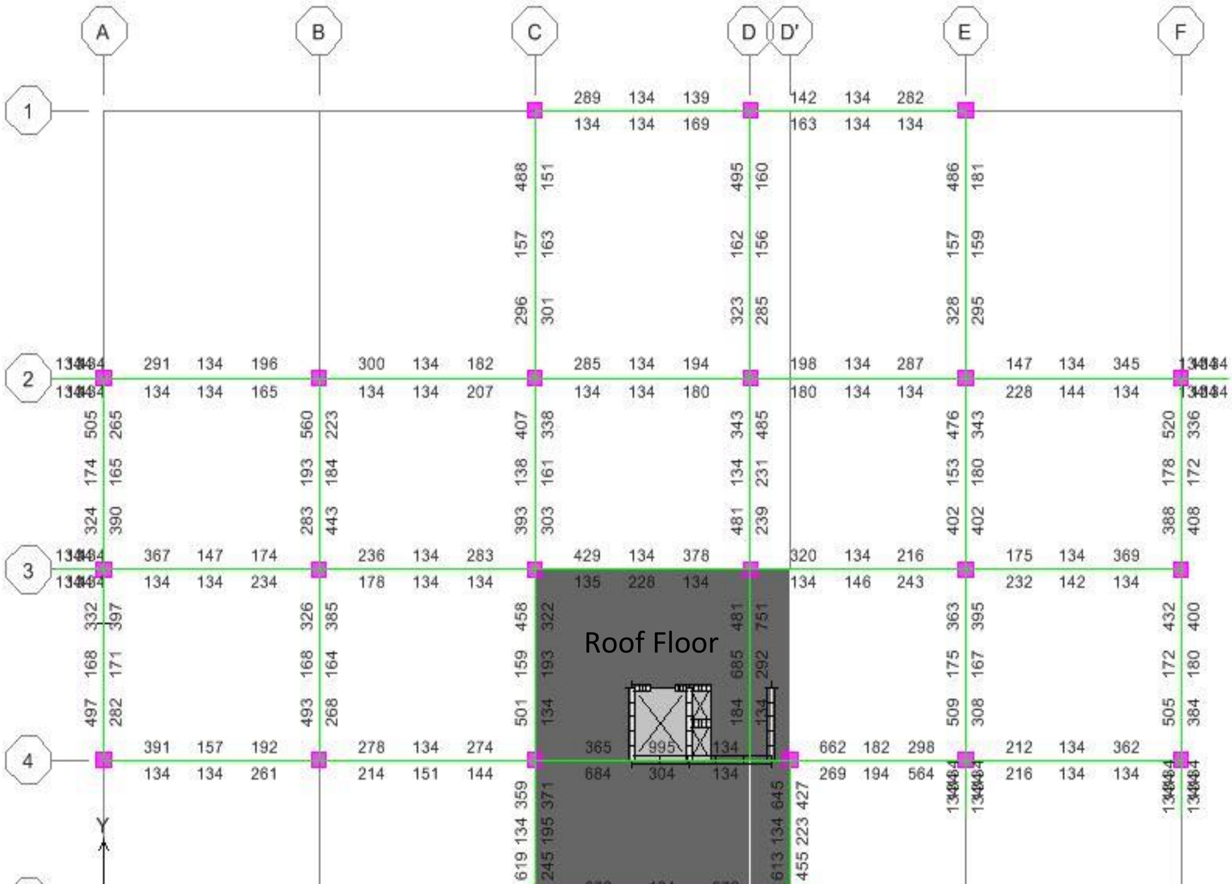
6^{th} Floor





7th Floor

Terrace Floor

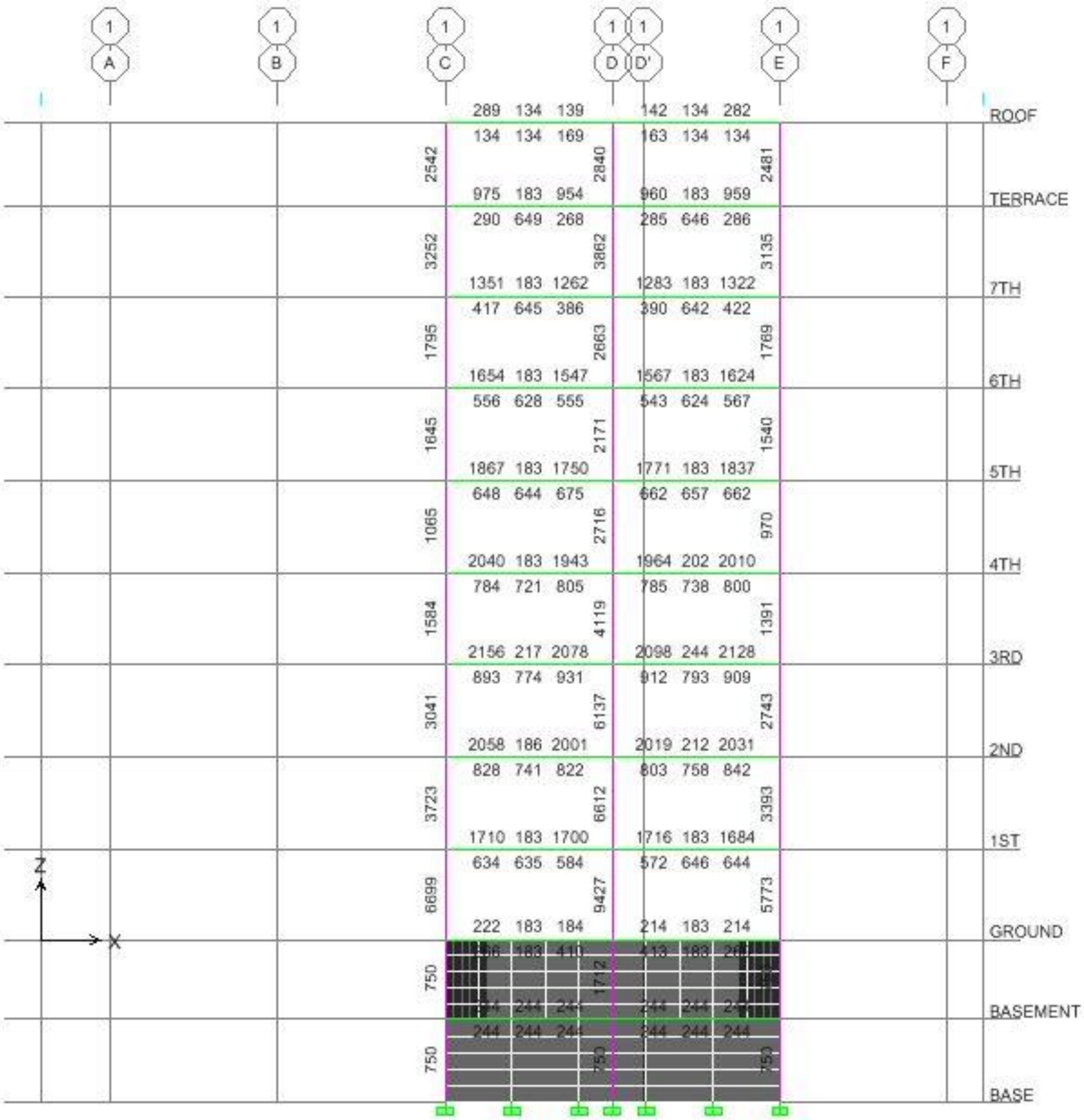


Column Design

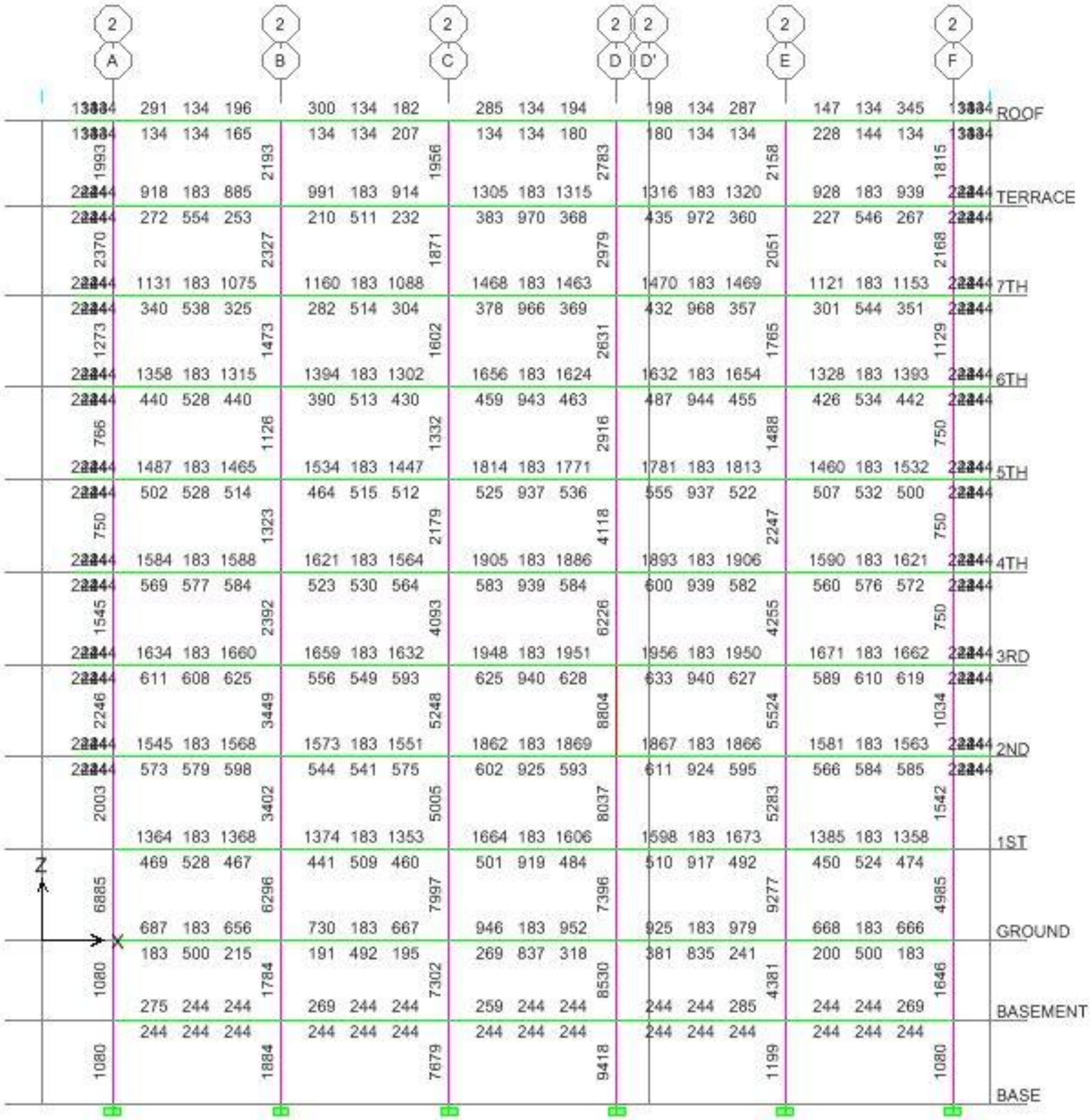
Columns are designed for the first order effects as well as for the second order effects, First order effects are those caused by direct application of the loads. Second order effects are those that occur from either of two sources:

- P-Delta Effects
- Slenderness Effects

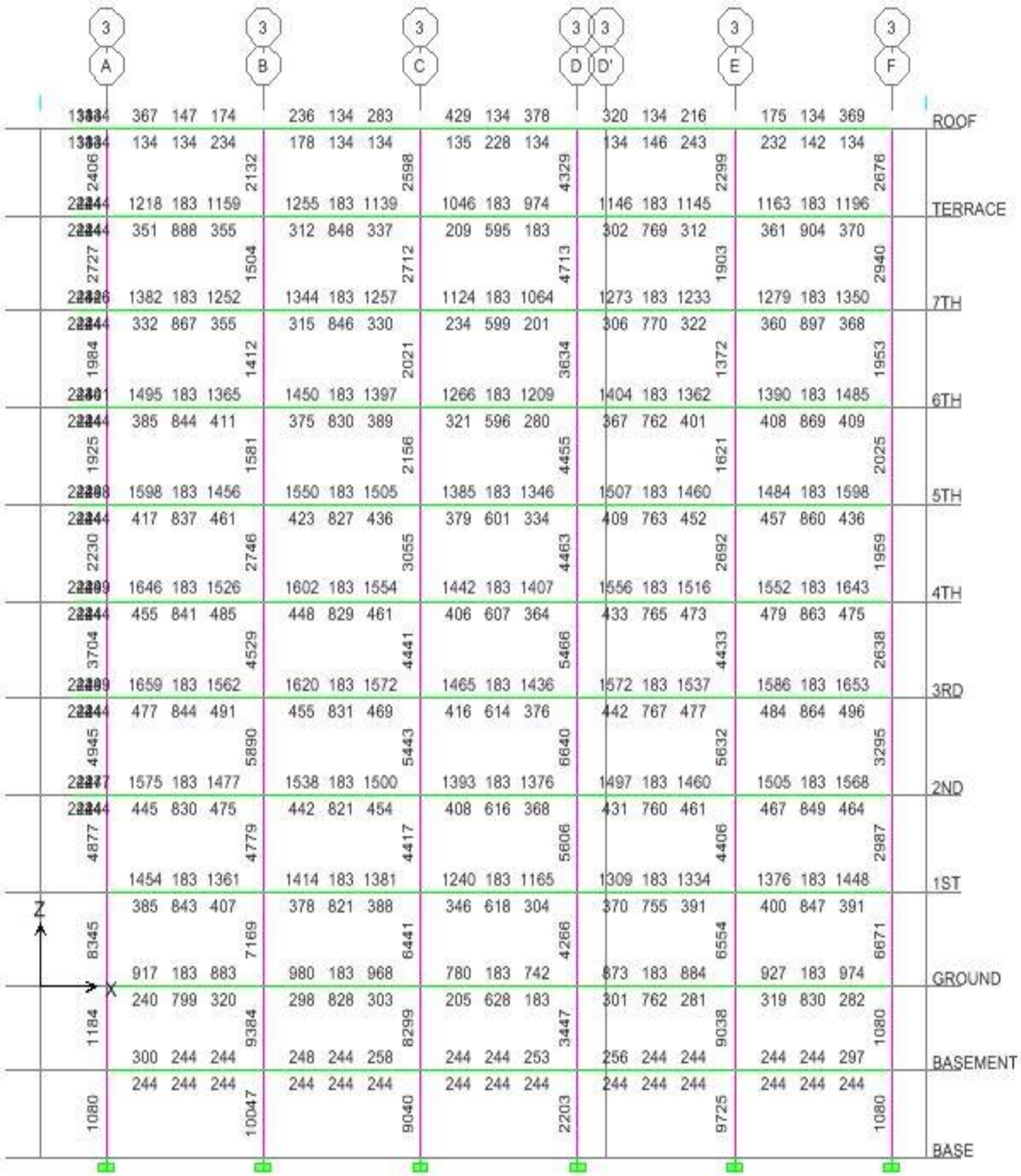
The 3D ETABS Analysis was set up so that it will take into account any p-delta effects resulting from lateral loads. Therefore the analysis results from ETABS give both the first and second order effects of the loads.


ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
A
B
C
D
D'
E
F

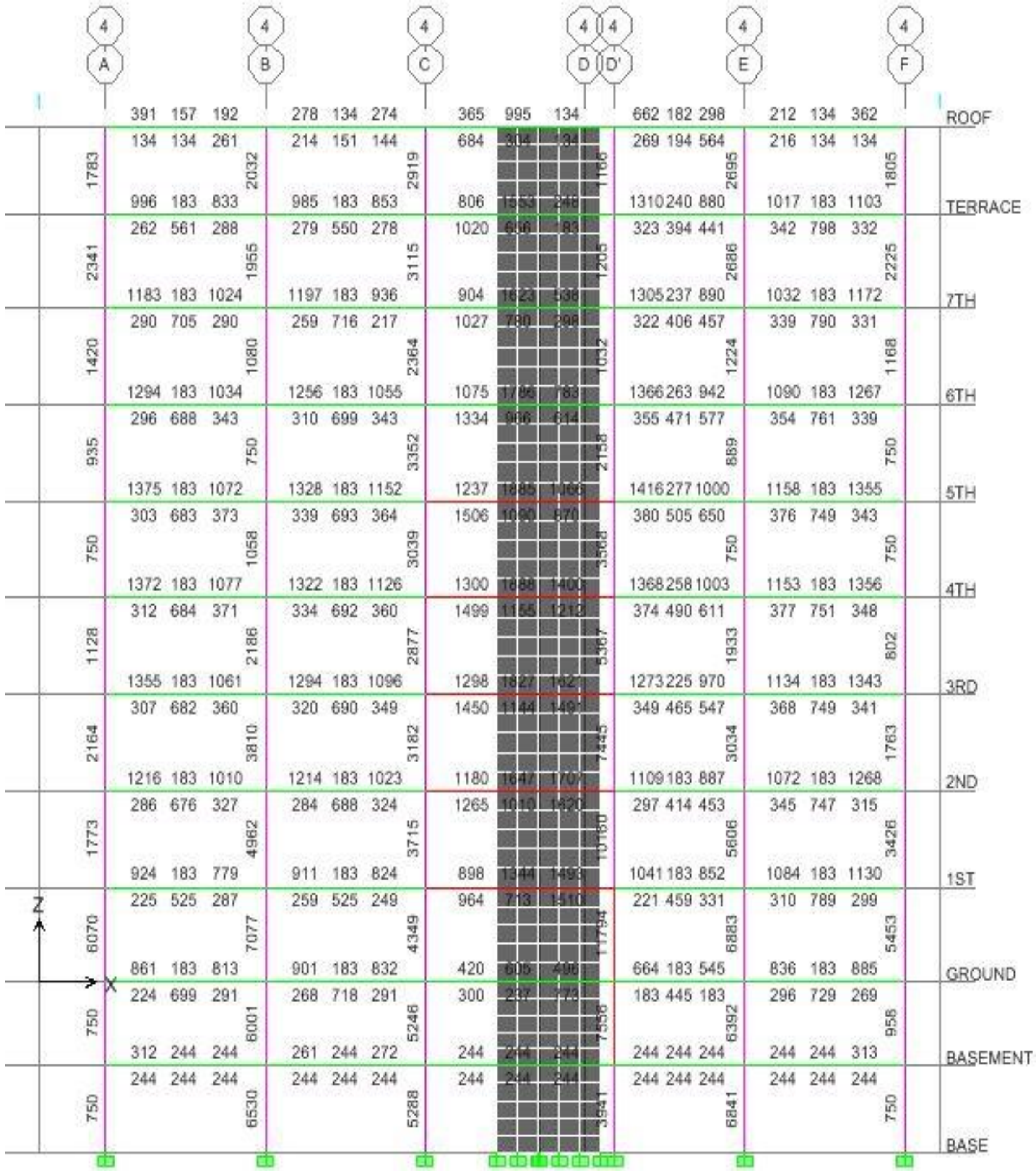
Axis 1



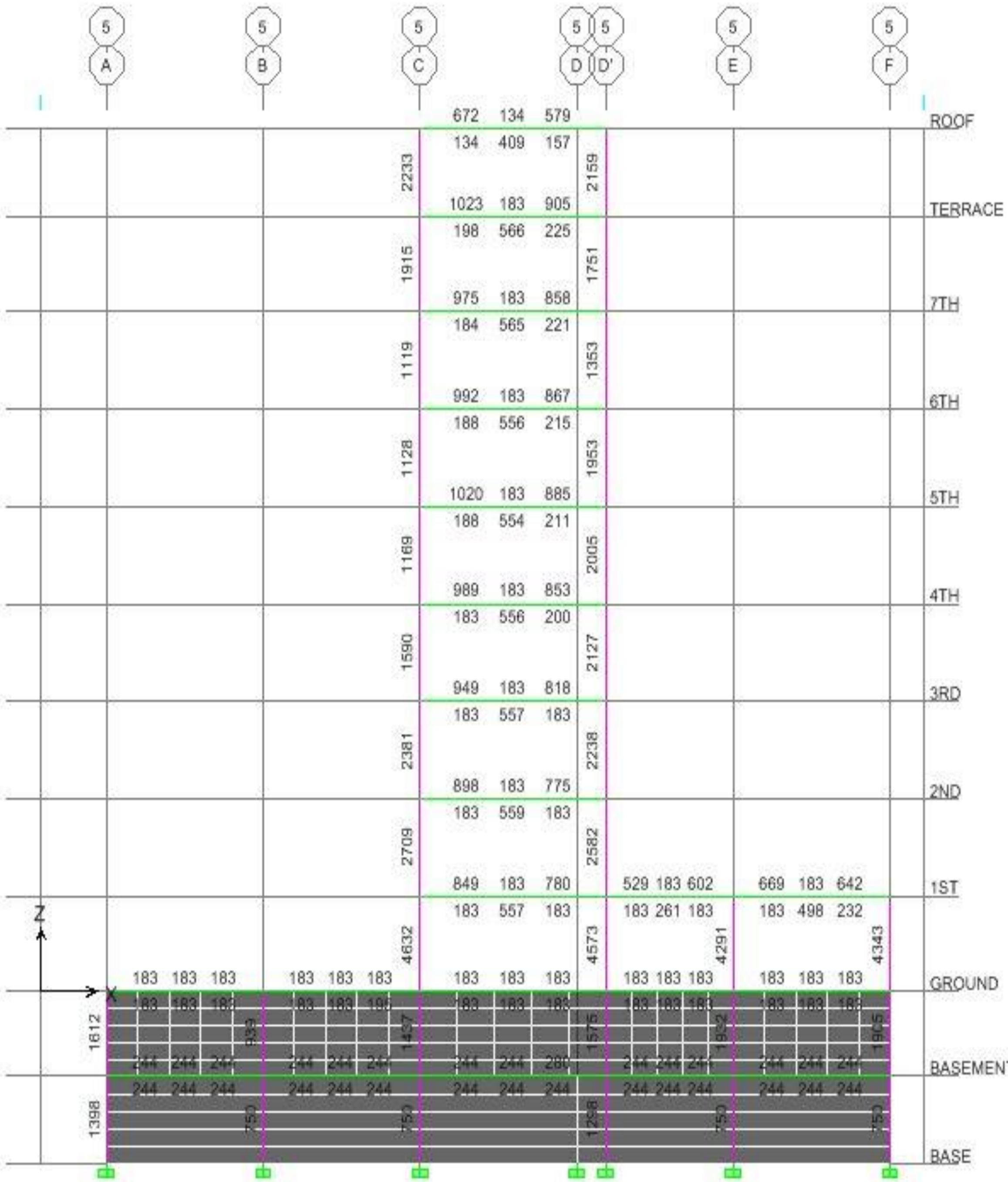
Axis 2



Axis 3



Axis 4


5
A
B
C
D
D'
E
F
ROOF
TERRACE
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
BASEMENT
BASE
Z
X

Axis 5

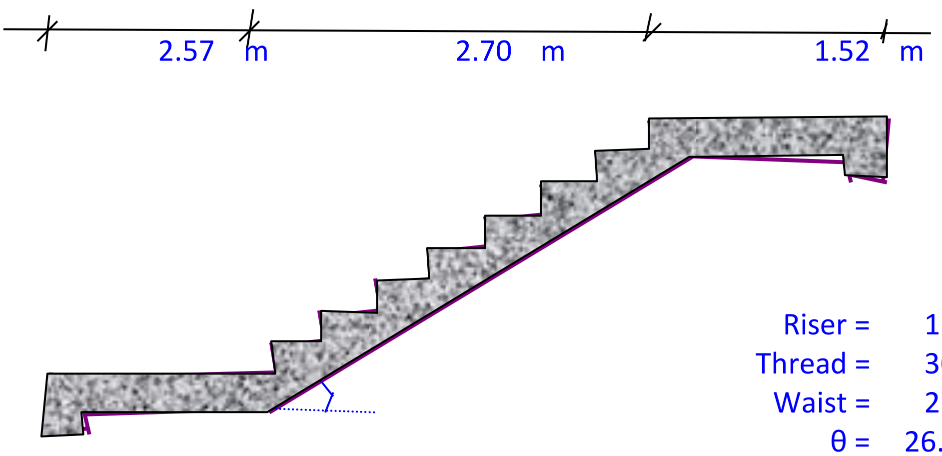
Stair Design

The stair was modeled as one way slabs running in longitudinal direction. And the design was conducted for each design actions, bending moment and shear force.

Analysis and design of the stair case system has been using excel sheet.

MAIN STAIR TYPE A

-



Riser =	15	cm
Thread =	30	cm
Waist =	25	cm
θ =	26.57	º
Stair width =	1.50	m

θ

Loading

Dead load at Steps:

Average slab thicknes s = 0.355 m

				x 25				
-	Own wt. of slab	=	1.50	x	0.355		13.29	KN/m
				x 23	0.03			
-	3cm cement screed	=	1.50	x	=		1.035	KN/m
				x 27	0.03			
-	3cm marble floor finish	=	1.50	x	=		1.215	KN/m

15.54 KN/m

- Live load = **3.00 KN/m²** [EBCS-1 Table2.10]

Design Load = 1.3DL + 1.6LL = **25.01 KN/m**

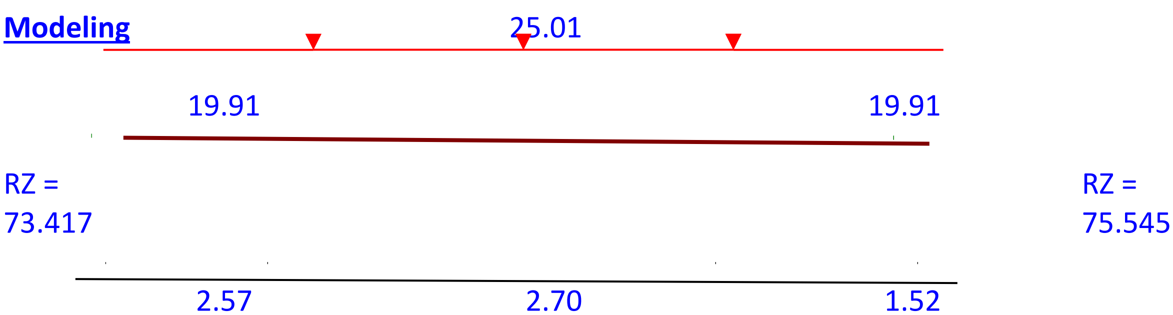
Dead load at Landing:

-	Own wt. of slab	=	1.50	x 25 x	0.25	9.38 KN/m
-	2cm cement screed	=	1.50	x 23 x	0.02 =	1.035 KN/m
-	3cm marble floor finish	=	1.50	x 27 x	0.03 =	1.215 KN/m
						11.63 KN/m

- Live load = **3.00 KN/m²** [EBCS-1 Table2.10]

Design Load = 1.3DL + 1.6LL = **19.91 KN/m**

Modeling



Design actions

- design moment = 186. KNm

81
73.4

- design shear = 17 KN

Check for Deflection

$$\Delta_{max} = \frac{5wL^4}{384EI}$$

w = 18.5 KN/m

L = 6.79 m

E = 29.0 Gpa

Icr = 0.0007 m^4

= 25.89 mm

- According to EBCS-2 Sec. 5.2.2 The final deflection shall not exceed the value:

$$\delta = \frac{L_e}{200}$$

= 33.95 mm **OK!**

Check for Shear Capacity

$$V_c = 0.25\, f_{ctd}\, k_1 k_2 b_w d$$

fctd = 1000 Kpa

$$k_1 = 1 + 50\,\rho$$

k1 = = 1.1

k2 = 1.6-d = 1.381

Calculated Shear Capacity = 120.50 OK!

Reinforcement calculation

$$\rho = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2mR_n}{f_{yd}}}\right]$$

b = 1.5

d = 0.219

f_{yd} = 347,826

f_{cd} = 11333.33333

ρ_{min} = 0.00125 [EBCS 2 7.2.2.2]

$m = \frac{f_{yd}}{f_{cd}}$ = 30.69

$R_n = \frac{Mu}{bd^2} = 2596.76$

? = 0.0086

$$A_s = \rho bd$$

A_s = 0.0028

Using ? **16** (As = 2E-04)

Use 14.05 Bars

Use ? 16 @ 100 mm

(As/m = 0.00302)

Check Capacity

$$M_n = A_s f_{yd}\left[d - \frac{a}{2}\right]$$

$a = \frac{A_s f_{yd}}{f_{cd} b} = 0.0617$

M_n = 197.37 = 5.6% Reserve

- According to EBCS-2 Sec.7.2.2.2 the ratio of the secondary reinforcement to the main reinforcement shall be at least equal to 0.2. Thus transverse reinforcement:

A_s = 0.0006

Using ? **8** (As = 5E-05)

Use 11.24 Bars

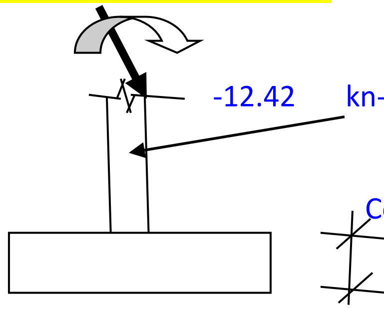
Use ? 8 @ 130 mm

Foundation Design

The foundation was sized so that the assumed bearing capacity of the soil is not exceeded. Bearing capacity of the soil was assumed to be ***290KPa***from the test result and this has to be verified by the engineer before construction. And also the soil investigation result recommends the foundation type to be an Isolated Footing, so we have designed the Isolated Footing accordingly as shown on the structural detail.

FOOTING F1

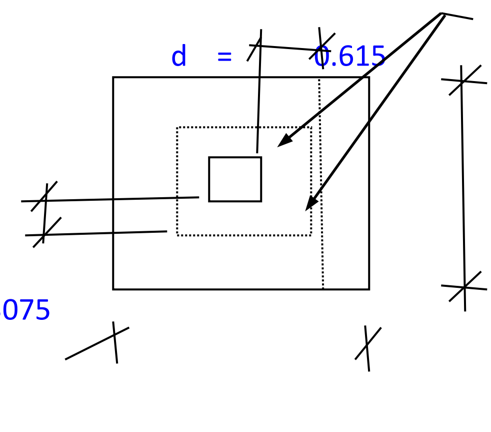
Service Load = 2250 kn



-12.42 kn-m

Col Size 0.4 x 0.35

t = 0.65



d = 0.615

Critical Sections

2.9 m

d/2 = 0.3075

2.9 m

Step 1 - Check Soil Pressure

Pressure from Self Weight of Ftg = 16.25 Kpa

Net Allowable Soil Pressure = 280 Kpa

e = -0.006 < $\frac{L}{6}$

$$q = \frac{P}{A} \pm \frac{My}{I}$$

$\frac{P}{A}$ = $\frac{2250}{8.41}$ = 267.5 Kpa

$\frac{My}{I}$ = y = 1.45 m

I = 5.89 m^4

= -3.06 Kpa

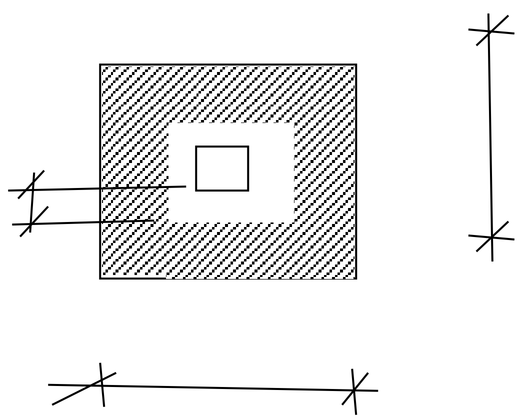
***q* = 264.5 Kpa** OK

***q* = 270.6 Kpa** OK

Step 2 - Check Thickness

Check Punching Shear

Ultimate Load= 2250



Tributary Area for Punching Shear

2.9 m

d/2 = 0.3075

2.9 m

Pressure = 267.54 Kpa

Trib. Area = 7.38 m^2

Shear = **1974.38** KN

Punching Shear Capacity of Footing

$$V_{RD} = 0.5 f_{ctd}(1 + 50\rho)ud$$

Assumptions:

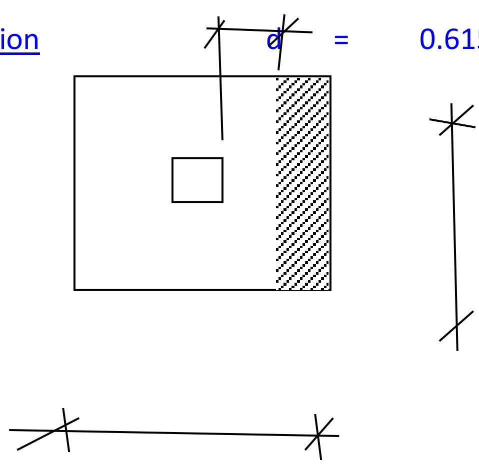
As = □□ @ 50

Critical Section Occurs at d/2 (Note: EBCS say critical section is at 1.5d, which contradicts all text books Therefore, use d/2)

f_{ctd}	=	1133	kpa
d	=	0.615	m
□	=	0.01022	
u	=	4.06	

= **2137.057** KN **OK**

Check one-way Action



d = 0.615

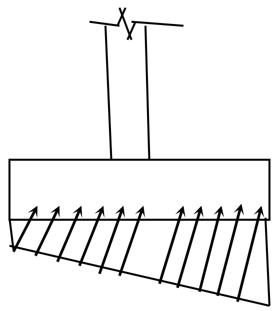
Tributary Area for one-way action

2.9 m

2.9 m

Calculate Shear @ Critical Section

270.6



264.5 Kpa

Critical Section From Edge of Footing is = 0.635 m

Pressure At Critical Section is = 265.821 3 Kpa

Shear At Critical Section is = 483.883 2 Kn

Shear Capacity of Section

$$V_c = 0.25 f_{ctd} k_1 k_2 b_w d$$

f_{ctd} = 1133

K_1 = (1+50ρ)

ρ = 0.01022

= 1.51

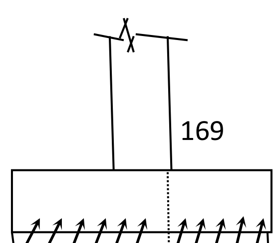
d = 0.615

K_2 = 1.6 -d = 1

b_w = 2.90

= **763.23 Kn** **OK**

Step 3 - Design for Flexure



270.6

264.5

Critical Section For Moment

Critical Section From Edge of Footing is = 1.25

Pressure At Critical Section is = 267.12 Kpa

Moment At Critical Section is = 207.3134 Kn - m/m

<u>Calculate Reinforcement</u>

$$\rho_{req} = \frac{1}{m}\left[1-\sqrt{1-\frac{2mR_n}{f_{yd}}}\right] \quad ; \quad m = \frac{f_{yd}}{f_{cd}} \quad ; \quad R_n = \frac{M_u}{bd^2}$$

f_{cd} = 11333 Kpa

f_{yd} = 260870.00 Kpa

d = 0.615 m

b = 1.00 m

m = 23.02

R_n = 548.12

= 0.0021546

As = 0.00133 = 6.590 Dia 16 bars

Use ⍰ 16 @ 150

151.738

Use f 16 @ 7

COST ANALYSIS OF REINFORCED CONCRETE

Ground Floor 460.03 m^2

	Item	Amount (Birr)
1	Column	237,122.15
2	Beam	210,842.00
3	Slab	397,476.70
	Sum	845,440.85

Unit

1837.80 ***birr/m²***

1st Floor Summary 429.24 m^2

	Item	Amount (Birr)
1	Column	241,667.25
2	Beam	210,842.00
3	Slab	370,861.60
	Sum	823,370.85

1918.2 ***birr/m²***

2nd Floor Summary 393.88 m^2

	Item	Amount (Birr)
1	Column	172,319.95
2	Beam	199,618.25
3	Slab	340,291.20
	Sum	712,229.40

1808.2 ***birr/m²***

3rd Floor Summary 393.88 m^2

	Item	Amount (Birr)
1	Column	172,319.95
2	Beam	199,618.25
3	Slab	340,291.20
	Sum	712,229.40

1808.2 ***birr/m²***

4th Floor Summary 393.88 m^2

	Item	Amount (Birr)
1	Column	172,319.95
2	Beam	199,618.25
3	Slab	340,291.20
	Sum	712,229.40

1808.2 ***birr/m²***

6th Floor Summary 393.88 m^2

	Item	Amount (Birr)
1	Column	172,319.95
2	Beam	199,618.25
3	Slab	340,291.20
	Sum	712,229.40

1808.2 ***birr/m²***

6th Floor Summary 393.88 m^2

	Item	Amount (Birr)
1	Column	122,185.20
2	Beam	199,618.25
3	Slab	340,291.20
	Sum	662,094.65

1681 ***birr/m²***

7th Floor Summary 393.88 m^2

	Item	Amount (Birr)
1	Column	122,185.20
2	Beam	199,618.25
3	Slab	340,291.20
	Sum	662,094.65

1681 ***birr/m²***

Terrace Floor Summary **432.1m^2**

	Item	Amount (Birr)
1	Column	93,090.40
2	Beam	188,972.25
3	Slab	347,683.35
	Sum	629,746.00

1457.4 ***birr/m^2***

Roof Floor Summary 48.56 m^2

	Item	Amount (Birr)
1	Column	93,090.40
2	Beam	188,972.25
3	Slab	39,005.40
	Sum	321,068.05

birr/m^2

Note:

Item	Unit Price (Birr)
C-25 Concrete	2200.00
Formwork	100.00
Reinforcement Steel	40.00

8.RESULTS

To make calculation simplify

8.1 Design of slab

8.1.1 Limit state design

Load and moment on the slab:

Dead load and live loads are calculated depending on the service of the slabs and self weight. Partition loads are distributed over the slab if they are not large enough to cause localized effects. The design loads are factored according to the following formula

$$P_d = 1.3G_k + 1.6Q_k$$

P_d —Design load
G_k − Total dead load on slab
Q_k − Total live load on slab
Panel:
Depth for deflection:

$$d \geq (0.4 + \frac{0.6f_{yk}}{400})\frac{L_e}{\beta_a}$$

Select the panel in the Ground floor panel size interior 4.52x5.02(4.752x5.27 , overall)

$$\frac{L_y}{L_X} = 1.10$$

$$L_e = 4.75m$$

$$\beta_a = 41.7$$

$$Therefore\ d = 97.84172(Two\ way\ slab)$$

Dead load:
Self weight= 0.21*25= 5.25 kn/m^2
Terrazzo floor finish(2cm)= 0.02*23= 0.46 kn/m^2
Cement screed(3cm)=0.03*23=0.69 kn/m^2
Total dead load: 6.4 kn/m^2
Live load:
Live load on floor slab5.0 kn/m^2 is considered for category general case (EBCS-1 Table-2.1)
From EBCS-1 the different live loads of rooms given below:

Table 7.0 Live loads for different usage

Room	Class	Live load
Shops	D1	5kn/m^2
Office stores	D1	5kn/m^2
Toilet	C1	2kn/m^2

For the chosen panel:

$$P_d = 1.3(6.4) + 1.6(5) = 16.32\ kn/m^2$$

Short span length: 4.725m

$$\frac{L_y}{L_X} = 1.10$$

1.10< 2 thus it is two way slab

From EBCS-2 1995 table A1 provision

$$\alpha_{xs} = 0.0345$$
$$\alpha_{xf} = 0.026$$
$$\alpha_{ys} = 0.032$$
$$\alpha_{yf} = 0.024$$
$$M_{xs} = \alpha_{xs} p_d (L_x)^2 = 0.0345(16.32)(4.75)^2 = 12.97\ kn - m$$
$$M_{xf} = \alpha_{xf} p_d (L_x)^2 = 0.026(16.32)(4.75)^2 = 9.77 kn - m$$
$$M_{ys} = \alpha_{ys} p_d (L_x)^2 = 0.032(16.32)(4.75)^2 = 12.03\ kn - m$$
$$M_{yf} = \alpha_{yf} p_d (L_x)^2 = 0.024(16.32)(4.75)^2 = 9.02\ kn - m$$

Reinforcement:

Table 7.1 Reinforcement distribution for the R.C.C slab

M_{xs}	M_{xf}	M_{ys}	M_{yf}
$10\emptyset\ @240\ mmc/c$	$10\emptyset\ at\ 260 mmc/c$	$10\emptyset @240 mmc/c$	$10\emptyset @260 mmc/c$

8.1.2 Composite slab design:

Hi bond 55/100 steel profiled sheeting. A reinforcing steel mesh is placed in the concrete slab with 25 mm to cover providing 188mm^2/m of reinforcement ($\emptyset = 6\ mm, p = 150 mm$). For the check of the longitudinal shear the "m" and "k" parameters for the Hi bond 55/100 for the profiled sheeting are equal to 86 and 0.69 respectively. The analysis is related to 1000mm.

Characteristic materials:

The design strength of the materials is assumed

Concrete $f_{ck=} 25 n/mm^2 \gamma_c = 1.5$

Steel: Reinforcement $f_{ysrk} = 235\ n/mm^2 \gamma_s = 1.15$
Steel sheeting $f_{ypk} = 320\ n/mm^2 \gamma_p = 1.10$
Verification of the profiled sheeting:
Loads:
Self-weight of the sheet $g_p = 0.13\ knm/m^2$
Weight of the wet concrete construction load $g_c = 2.47\ knm/m^2$
Therefore self-weight of the slab and floor finishes g= 6.4 kn/m^2
Live load = q= 5 kn/m^2
Design load = P=$(\gamma_g * g + \gamma_q * q)b$
=(1.3*6.4+1.6*5)1=16.32 kn/m
Ultimate limit state:
Design bending moment = $\frac{Pl^2}{8} = \frac{16.32*4.75^2}{8} = 46.02\ kn - m$
Determination of the design bending resistance :
Calculation of the plastic neutral axis:

$$x_{pl} = \frac{A_p {f_{ypk}}/{\gamma_p}}{b(0.85 f_{ck})/{\gamma_c}} = \frac{1482 * 320/1.1}{1000(0.85*25)/1.5} = 30.43mm < 160mm$$

Design bending resistance $M_{plcs} = A_p \frac{f_{ypk}}{\gamma_p}\left(d_p - \frac{x_{pl}}{2}\right) = \frac{1482*320(160-27.5-\frac{30.43}{2})}{1.1} = 55.62\ kn - m$

$$M^+ = 46.02 < 55.62\ kn - m$$

Longitudinal shear:

$$V_1 = \frac{PL}{2} = \frac{16.32(4.75)}{2} = 38.76\ KN$$

Design shear reisistance:

$$V_{1u} = \frac{bd_p(\frac{mA_p}{bl_s} + k)}{\gamma} = 1000(160 - 27.5)\frac{\{\frac{86(1482)}{1000(4750)} + 0.69)\}}{1.25} = 75\ kn$$

38.76< 75 kn.
Vertical shear:
Calculation of design vertical shear resistance

$$V_u = b_o d_p \tau_{wd} k_v (1.2 + 40\rho)$$

$$\tau_{wd} = 0.3\ kn/m^2$$

$$K_v = 1.6 - d_p = 1.6 - \frac{132.5}{1000} = 1.4675$$

1.2+40$\rho_o = 1.2 + 40\frac{A_p}{b_o d_p} = 1.2 + 40\frac{188}{500*132.5} = 1.314$

$$V_u = 500(132.5)(0.3)(1.4675)(1.314) = 38.32\ kn$$

$$V_1 \cong V_u$$

Steel profile:

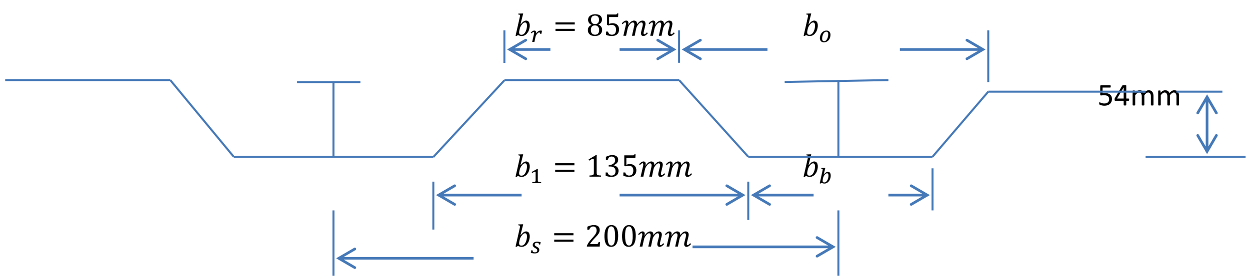


Fig 7.0 Steel Profile

Area of concrete $A_c = 1000h - [\frac{1000}{b_s}\left\{\frac{b_1+b_r}{2}\right\} h_p]$

8.2 Column:

8.2.1 Limit state design

The loads on the column:

The height of the column is 4m

Design axial load- 6395.989 Kn

Design bending moment about XX-axis – 625 kn-m

Design bending moment about yy-axis – 428.74 kn-m

Considering the section of 800X500 mm

For this loading the design steel requirement is 5.5%

Therefore the weight of the steel = (5.5/100) *800*500*4000*7800X10^{-9}= 154.74kg

Volume of the concrete =0.8*0.5*4= 1.6 m^3

8.2.2. Composite design

For the above loads the section dimensions chosen are 500X500mm and the steel profile is HE400AA and the reinforcement provided is 4 numbers of 12 mm diameter with each one at the corner.

The geometric characteristics of the section is

$$b_c = 500mm, \quad d_c = 500mm$$

$$b_{rs} = 430mm \ and \ \ d_{rs} = 430mm$$

$$b_s = 300mm \ and \ d_s = 378 \ mm$$

HE400AA properties:

Weight 92.4kg/m

H=378mm

B=300mm

$t_w = 9.5 \ mm$

$t_f = 13.0mm$

A=118 cm^2

Second moment of the area-

Strong axis $I_y = 31250 \ cm^4$

Weak axis $I_z = 5861 \ cm^4$

Elastic section modulus:

$W_y = 1654 \ cm^3$

$W_z = 391 \ cm^3$

Plastic section modulus:

$W_{ply} = 1824 \ cm^3$

$W_{plz} = 600 \ cm^3$

For rebar's☹If the longitudinal reinforcement is considered in the design , the minimum share of 0.3% and maximum share of 4% of the concrete area shall be provided.)

$A_{sr} = 4 * 113 = 452 \ mm^2$

$I_{sr} = 4 * 113 * (\frac{430}{2})^2 = 2089.37 mm^4$

For concrete:

$$A_c = 500 * 500 - 11800 - 452 = 237748 mm^2$$

Strong Axis $I_c = \frac{500*500^3}{12} - 31250X10^4 - 2089.37 = 4895.83X10^6 mm^4$

Weak Axis $I_c = \frac{500*500^3}{12} - 5861X10^4 - 2089.37 = 3149.72X10^6 mm^4$

Therefore $N_{pl} = \frac{355(118X10^2)}{1.10} + \frac{420*452}{1.15} + \frac{0.85*25*237748}{1.50} = 7302.83Kn.$

The rules suggested by the Eurocode /EBCS-4 can be used only if the following restrictions are satisfied.

1) The cross sections are symmetric about two axes and the cross section is constant along the member
2) The factor δ_s, that represents the contribution of structural steel in the plastic axial load capacity varies between 0.2 and 0.9, otherwise the member has to be designed as a reinforced concrete element corresponding to a lower restriction or as a steel element corresponding to a higher restriction.

$$\delta_s = \frac{11800 * \frac{355}{1.1}}{7302.83X10^3} = 0.52$$

$$M_{max} = W_s f_{ysd} + W_{sr} f_{ysrd} + \frac{W_c}{2}(0.85) f_{cd}$$

$$M_{pl} = (W_s - W_{sn}) f_{ysd} + (W_{sr} - W_{srn}) f_{ysrd} + \frac{(W_c - W_{cn})}{2} 0.85 f_{cd}$$

Plastic moduli:

$$W_s = 1824X10^3 mm^3$$

$$W_{sr} = 4 * 113 * 215 = 97180 \ mm^3$$

$$W_c = \frac{b_c {h_c}^2}{4} - W_s - W_{sr} = \frac{500 * 500^2}{4} - 1824X10^3 - 97180 = 29328.82X10^3 mm^3$$

$$N_{plc} = 0.85 * 16.67 * 237748 = 3368.770\ KN$$

$$h_n = \frac{3368.770/2}{500 * 0.85 * 16.67 + 9.5(2 * 322.7 - 0.85 * 16.67)} = 128mm$$

$$W_{sn} = t_w {h_n}^2 = 9.5(128)^2 = 155648\ mm^3$$

$$W_{srn} = 0$$

$$W_{cn} = b_c {h_n}^2 - W_{sn} = 500 * (128)^2 - 155648 = 8036.35X10^3 mm^3$$

Therefore $M_{max} = 1824X10^3(322.7) + 97180(365.5) + \frac{29328.82X10^3}{2}$ *0.85*16.67=831.82kn-m

$$M_{pl} = (1824X10^3 - 155.468X10^3)322.7 + 97.180X10^3 * 365.2 + \frac{1}{2}(29328.82X10^3 - 8036.35X10^3) * 0.85 * 16.67 = 723\ kn - m$$

8.3 Beam design:

In designing the beams almost all of the beams are coming under the continuous beams. So analysis and design of the beams under continuity effect should be taken for finding the various parameters. For doing cost comparison first group all the continuous beams into different ranges then take one of that do for that both composite analysis and design along with the analysis and design of the Reinforced concrete method. Based on the derived parameters for both composite and R.C., cost of the elements can be compared. This will be applicable for both slabs and columns also.

Ultimate limit state

Different methods can be adopted to analyze the structure in ultimate conditions, the main features of which are summarized in the following.

Plastic analysis

The requirement that all relevant cross sections are plastic or compact may not be sufficient for a composite beam to achieve the plastic collapse condition; It has been proven that the rotational capacity is sufficient to develop the collapse mechanism only if further particular limitations are meet as to the structural regularity, the loading pattern, and the lateral restraint. The limitations, more in detail, are the following.

i) Adjacent spans do not differ in length by more than 50% of the shorter span and end spans do not exceed 15% of the length of the adjacent spans.
ii) In any span in which more than half of the total design load is concentrated within a length of 1/5th of the span , at any hinge location under sagging moment , no more than 15% of the overall depth of the member is in compression.
iii) The steel flange under compression at a plastic hinge location is laterally restrained.

If these requirements are fulfilled, the limit design approach can be applied in design analysis. In this case at the ultimate limit condition in the external spans there is the following relation between the total applied load "p" and the negative and positive plastic moment of resistance of the beam

$$M_{pl}{}^{(+)} + \frac{1}{2} M_{pl}{}^{(-)} \cong \frac{pl^2}{8}$$

Whereas in the intermediate spans it is:

$$M_{pl}{}^{(+)} + M_{pl}{}^{(-)} = \frac{pl^2}{8}$$

Design of the composite beam:

For beam analysis take a beam at the ground floor level of two equal spans each is having 5.01m and 5.02m , it is designed with profiled steel sheeting. It is subjected to 16 kn/m as dead load and 24kn/m of live load

For this the structural steel IPE500 is characterized by $A_s = 11600\ mm^2$ and $I_s = 4.82X10^8 mm^4$. The slab thickness is 55+65 . The spacing of beam is 5000mm.

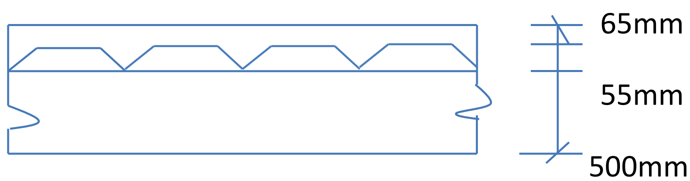


Fig.7.1 Composite steel concrete slab profile

Cross section of a simply supported beam with a span length "l" equals to 5m.

Evaluation of the effective width :$b_{eff} = 2\frac{5000}{8} = 1250mm$

Design of the R.C.C beam:

For the dimensions specified above if designed as R.C beam the dimensions required are 650mmX300mm with 6 no's of 20 mm bars

Shear connector Design

It is assumed that the connection is full interaction , with headed stud connectors having $H_{sc} = 100mm$ and $d_{sc} = 19mm.$ The ultimate strength of the connectors is $f_{u.sc.B} = 450\ {}^{N}/_{mm^2}.$ And partial safety factor $\gamma_v = 1.25$ is considered.

Step-1:

Longitudinal shear force- In accordance with the hypothesis of the full shear connection , the total design longitudinal shear V_1 to transfer by shear connectors , spaced between the point of maximum sagging moment and the end support is

$$V_1 = \min(F_{s.max,}F_{c.max,,})$$

Where $F_{s.max,} -$ the steel beam limit resistance

$F_{c.max,,} -$ Concrete limit resistance.

Therefore neglecting the contribution offered by the longitudinal slab reinforcement and with reference to the steel beam

$$F_{s.max,} = \frac{A_{a*} f_{y.s.k}}{\gamma_s} = \frac{11600 * 235}{1.1} = 2478.2\ Kn$$

With reference to the concrete

$$F_{c.max,,} = \frac{0.85 * A_c * f_{ck}}{\gamma_c} = \frac{0.85 * b_{eff} * h_c * f_{c.k}}{\gamma_c} = \frac{0.85 * 1250 * 120 * 25}{1.5} = 2125\ Kn$$

Then $V_1 = 2125\ Kn$

Step-2:

Design resistance of shear connection – it is necessary to make reference to the lower value between

The resistance of concrete

$$q_{u.c} = \frac{k_c * A_{sc}\sqrt{f_{c.d} * E_c}}{\gamma_v} = \frac{0.36 * \frac{\pi * 19^2}{4} * \sqrt{25 * 30500}}{1.25} = 71.27\ Kn$$

The resistance of the stud connectors

$$q_{u.s} = k_s * f_{u.sc.d}\frac{A_{sc}}{\gamma_v} = 0.8 * 450 * \frac{\pi * 19^2}{4}\frac{1}{1.25} = 81.7\ Kn$$

Then $q_{u.d} = q_{u.c} = 71.27\ Kn$

Step-3:

Design the connection

Minimum number of connectors $N_f = 2 * \frac{V_1}{q_{u.d}} = 2 * \frac{2125}{71.27} = 59.63$

It is assumed N= 2 *30 = 60 stud connectors

Spacing of connectors

The stud connectors are spaced uniformly, being :

All critical sections are in class I

- $\frac{N}{N_f} = \frac{60}{59.63} = 1.006 \geq 0.25 + 0.03 * l = 0.25 + 0.03 * 5.02 = 0.406$
- $\frac{M_{pl}}{M_{pl.s}} = \frac{M_{pl.c}}{W *_s f_{y.s.k}/\gamma_s} = \frac{830}{2194 * 235/1.1} = 1.77 < 2.5$

It is assumed uniform spacing between studs$i = \frac{l}{N} = \frac{5020}{60} = 83.66\ mm$

It is assumed 64 connectors at a spacing of 80 mm

Step-4:

Detailing of the shear connection

Spacing:

$$i \leq 22 * t_f\sqrt{\frac{235}{f_{y.s.k}}} = 22 * 16 * \sqrt{\frac{235}{235}} = 352\ mm$$

Overall height:

$$6H_{sc} < i < 5d_{sc} = 6 * 100 < 125 < 5 * 19 = 95\ mm$$

Step-5:

Reinforcement in the slab: The area of the reinforcement in a solid slab must not be less than 0.02 times the concrete area being reinforced and should be uniformly spaced. Referring to a width of 1000mm , minimum amount of reinforcement is :

$$0.002 * A_c = 0.002 * 120 * 1000 = 240\ mm^2/m$$

It is assumed double layers of reinforcement $\emptyset\ \ 10\ mm/200\ mm$

$$A_{ef.v} = 2 * 392.7 = 785.4\ mm^2/m$$

Price calculation: The price based on United States of America.

The size of the shear connector is 19mm X100mm i.e, $\frac{3}{4}'' X\ 3\frac{7}{8}''$

i) For this size the cost of the each shear stud connector is \$ 2.58

ii) Labor cost per hour per standard work day = \$75(in Europe it is 20 $\epsilon/hour$)

iii) Rental of STUD welded w/Generator daily rental rate = \$550(generally the power required for doing welding is more than as used in the house holds supply, therefore the generator is required.)

iv) Typically, two workers can place 2000 studs on a building deck in one day. One worker will lay out studs and ferrules (welding shields/molds) where the studs are to be welded, and the other worker will follow with a stud gun and weld them into place.(THE DESIGN AND FABRICATION OF AN AUTOMATED SHEAR STUD WELDING SYSTEM, Andrew Glenn Ziegler, Research Assistant, Alexander Henry Slocum, Assistant Professor of Civil Engineering ,Massachusetts Institute of Technology Cambridge, Massachusetts 02139) Therefore one workercan do 1000 in 8hrs i.e, working day , thus per an hour he can do for 125 studs.

v) For our problem the number of studs are 64 required that is in an half an hour he can finish the work. There froe the cost of the labor is - \$37.5

The total cost is:

64 shear studs cost	@ $2.58 each price	64X $2.58= $165.12
Labor cost for half an hour is	@ $75 per hour	$ 37.5
Stud welded w/Generator for an half an hour	$550 for 24 hrs = $22.91/hour	$11.45
Consumables are there while doing welding	Price not available	---------------
Total cost for our problem for doing stud welding is		$214.07 for 5.02 m span beam
Equivalent birr amount		Birr 4709 for 5.02 m span beam.

Table 7.2 Cost calculation of the shear studs

Form work required:

For beam: Per meter length

(Width of the beam+ Depth of the beam+ Depth of the beam)*1 = m^2/m

Therefore (0.300+0.650+0.650)*1= 1.6 m^2/m

For column: Per meter length

(Width of column+ Width of column + Depth of column + Depth of column) *1 = m^2/m

Therefore (0.80+0.50+0.80+0.50)*1= 2.6 m^2/m

For slab: per meter width

Size = width X Length = m^2

=(4.52 x 1)=4.52 m^2

Comparison of Composite elements with the conventional elements:

Slab

Table 7.2 cost comparison for the slab

In calculation of slabs consider “1 m “ width

Material type and rate	Composite design quantities		R.C.C .Design Quantities	
Steel- Birr 40 /kg	11.45 kg/sq.m= Birr 458		5.64 kg/sq.m=Birr 225.6	
Concrete – Birr 2200/m^3	0.093m^3/m= Birr 204.6		0.22m^3/m=Birr 484	
Form work – Birr 100/m^2	------------------------------------		4.52x100=452	
	Total=	Birr 662.6	Total =	Birr 1161.6

Beam

Table 7.3 Cost comparison for the beam

Material type and rate	Composite design quantities		R.C.C design Quantities	
Steel- Birr 40 /kg	290.8 kg= 11632		152.5kg=6100	
Concrete – Birr 2200/m3	--------------------------------------		0.835 m^3=1837	
Form work – Birr 100/m2	---------------------------------------		1.6x100=160	
	Total=	Birr-11632	Total=	8097

Column

7.4 Cost comparison for the column

Material type and rate	Composite design quantities		R.C.C. Design quantites	
Steel- Birr 40 /kg	857.6kg=34304		245kg=9800	
Concrete – Birr 2200/m3	1.45= 3190		1.89=4158	
Form work – Birr 100/m2	2.6X100=260		2.6x100=260	
	Total=	37754	Total=	13218

Fig. 7.2 Cost comparison of the elements.

Discussion

The above tables showing the cost comparison of the composite and R.C.C structures of the building. For cost comparison only one element from slab, beam and column have taken. Because the calculations for the other elements is same. If we compare the costs for these three elements the cost for the slab in composite is less than the other two that is beam and column. In the case of the beam and column though the initial cost for the elements may be high but in overall the composite will give economy because of the following reasons.

With the use of composite columns with composite beams and composite decking it is promising to erect high rise structures in a very efficient manner. At any time there is quite a vertical spread of construction activity carried out simultaneously, with several trades working concurrently.

For example:

1) One group of workers will be erecting the steel columns and beams for one or two story's at the top of frame
2) Another group of workers two or three stories below will be fixing the metal decking for the floors.
3) Concreting the floors will be done a few story's below by another group

4) Another group as we go down the building trying the column reinforcing the bars in cages.
5) Below them yet another group is fixing the formwork placing the concrete into the column moulds etc.

Advantageous:

In a composite structure advantageous properties of both concrete and steel are efficiently utilized. Time of construction for a typical 3 to 10 stories structure, of the complex structure decreases by about 25%

The advantageous can be fully utilized as summarized below:

1) Quick return of the invested capital because of faster construction for maximum utilization of rolled and/or fabricated components(structural steel members)
2) It is advantageous based on life cycle cost analysis instead of initial cost only.
3) More usable space because of ability to cover large column free area in buildings and longer span bridges /flyovers.
4) Most effective utilization of the materials can be achieved by utilizing steel joist in tension and reinforced concrete (R.C.C) slab in compression.
5) Better seismic resistance i.e., best suited to resist repeated earthquake loadings which require a high amount of ductility and hysteretic energy of the materials/structural frame.
6) Bending stress as well as deflection is lesser Composite sections because they have higher stiffness than the corresponding steel sections (in a steel structure).
7) If span and loading unaltered a lower structural steel section (i.e. having lesser depth and weight) can be provided in composite construction, compared to the section required for non-composite construction.
8) Reduced cost of cladding in a building due to reduced beam depth reduces the story height
9) Cost of form work is lower compared to R.C.C construction
10) Due to reduced depth allows of lower cost for fire proofing of beam's exposed faces.
11) Easy structural maintenance/modification/repair
12) At the end of useful life structural steel component has considerable scrap value
13) Reductions in over all weight of structure lead to the reduction in foundation costs.
14) More use of material i.e., steel which is long-lasting fully recyclable on replacement and eco-friendly.

Conclusion

The price of the typical slab, beam and column along with the cost of the shear stud connectors were calculated. It shows that the cost of the composite steel and concrete is cheaper than the R.C slab .and the other two that is the cost of the beam and column are higher in composite steel and concrete. But this can be overcome by reducing the duration of the construction and by making the structure into service will reduce the blocking time of capital and also structurally the composite steel and concrete has the better performance.

References

1) D. R. Panchal and P. M. Marathe(2011).Comparative Study of R.C.C, Steel andComposite (G+30 Storey) Building ,institute of technology, nirma university, ahmedabad – 382 481, 08-10 December, 2011.
2) Euro code 4: Design of Composite steel and Concrete Structures, British Standards Institution, London, 1994
3) Mahbuba begum (20 December, 2012) .cost Analysis of steel concrete composite structures in Bangladesh. Asian Journal of civil Engineering (BHRC) vol.14, no.6(2013) pages 935-944.
4) http://www.reidsteel.com/brochure.htm
5) Handouts (Addis Ababa institute of technology, postgraduate program for the course “steel concrete composite structures” by Dr.Ing.Grima Z.)
6) http://www.ccanz.org.nz/images/document/CCANZ%20Briefing%20Note%201.pdf
7) EBCS 1, 2, 3 & 4 (Revised)
8) R.P.Johnson (1994) .composite structures of steel and concrete. Blackwell scientific publications,vol.1,2nd edition , Britain.muhaba et al
9) James K. Wight, James G. Macgregor. Reinforced Concrete Mechanics and Design. Upper Saddle River, New Jersey 07458, 5th edition.
10) Composite Structures of Steel and Concrete Beams, slabs, columns, and frames for buildings Third Edition R.P. JOHNSON
11) Johnson R.P, Composite structures of steel and concrete, volume 1 2004 Blackwell Scientific Press.
12) Johnson R.P, Designers' guide to Euro code 4 Design of Composite Buildings, 2nd edition. ICE.
13) Nether cot, D. Composite Construction. Spon Press
14) Johnson R. P., Composite Structures of Steel and Concrete, Volume 1, Blackwell Scientific Publications, UK, 2004.
15) Fisher J. w. Design of composite beams with formed metal deck. Eng. J,,amer, lnst ,Steel Constr,7.88-96,July 1970.
16) Dari J.Oehlers and Mark A.Bradford, (1999), ‘Elementary Behaviour of Composite Steel and Concrete Structural Members’, Butterworth and Heinmann.
17) Design Aids (for Reinforced Concrete) to IS 456 :1978 ,Special Publication SP : 16,Bureau of Indian Standards, New Delhi,1980
18) BS 5950(Part 3),Design of Simple and Continuous Beams, British Standards Institution, London
19) Handbook on Code of Practice for Design Loads (Other than Earthquake)for Buildings and Structures (IS : 875(Part 2) – 1987),Bureau of Indian Standards , New Delhi ,1989.

20) Handbook on Code of Practice for Design Loads (Other than Earthquake)for Buildings and Structures (IS : 875(Part 3) – 1987),Bureau of Indian Standards , New Delhi, 1989.
21) Handbook on Code of Practice for Design Loads (Other than Earthquake)for Buildings and Structures (IS : 875(Part 1) – 1987),Bureau of Indian Standards, New Delhi, 1989.
22) Handbook on Composite Construction-Multi-Storey Buildings-Part-3,(2002),Institute for Steel Development and Growth (INSDAG).