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Structural Design of G+10 Residential Building

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As members of the examining board of the final B.Sc. open defense, we verify that we have read and evaluated the final BSc thesis/project prepared by **Selam Alemayehu, Tigist Mulualem, Chereka Birhanu, Gacheno Gasha** entitled **Structural Design of G+10 Residential Building**, and recommended for acceptance as a fulfillment of the requirement of **B.Sc. in Civil Engineering**

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Acknowledgement

This thesis opportunity we had was a great chance for learning and professional development. Therefore, we consider ourselves as a very lucky individual as we were provided with an opportunity to be a part of it.

Bearing in mind, we are using this opportunity to express our deepest gratitude and special thanks to our advisors Mr. Msganaw Birhanu and Mr. Bulo Lema they are in spite of being extraordinarily busy with the duties, took time out to hear, guide and keep us on the correct path and allowing us to carry out our project correctly. Special thanks to our coordinator Mr. Kibert G/medhin hagos.

Summary

Now a day's construction industry plays a great role for the development of a nation in all aspects. As we all agree, behind every construction activity there must have structural analysis and design, from this consideration directly or indirectly structural analysis and design have a huge application in the development of a nation.

This thesis works on the structural analysis and design of G+10 residential building with commentaries by the new Ethiopian building code of standard ES EN 2015 based on what we have studied in the past academic years.

The thesis will have an objective to develop new skill, strength our capacity on structural analysis and design as well as integrate different discipline for specified and justified problem. Beside this, it will create awareness for others to develop this project idea for civil engineering profession.

This paper contains nine chapters; the first one is the general introduction and material properties. The second and the third chapters' deal with the design of solid slab and stairs on the structure. Chapter four and five are about the lateral i.e. Earth quake analysis. Whereas chapter six deals with the frame analysis of the building. The next two chapters concern about the design of structural members i.e. beam and column. Finally, the last chapter gives foundation.

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Background

They are so many types of structure design based on its function such as Residential, Hotel, and Mixed use…. etc. structure design. In our case Residential building structure design, it is important for today.

The building is a G+10 reinforced concrete frame structure located in Addis Ababa. The building is composed of solid slab system to support the loads due to occupancy. Form the sectional elevation view of the building, the stairs type is stairs with quarter-turn landing. The elevator shaft is located at the middle of the stairs three flights. The geometry of the roof is a duo pitch roof type. Wind load analysis is carried out for each cases of wind direction and the worst-case scenario is considered for design. In addition, galvanized steel sheet from the Kality metal products factory-manufacturing manual is used as a roof cover.

Ground floor contain shop, hotel, game zone and in first floor living and dining room, master bedroom, shower, bath room, bed room and kitchen. Above it all floors are similar to first floor.

1 Introduction

Reinforced concrete is the most widely used construction material in the world in the construction industry. It is a composite structure of construction material concrete and steel reinforcement bars. It is a concrete with steel bars embedded in it. The universal nature of reinforced concrete construction stems from the wide availability of reinforcing bars and the constituents of concrete (gravel or crushed rock, sand, water, and cement), from the relatively simple skills required in concrete construction, and from the economy of reinforced concrete compared with other forms of construction. Plain concrete and reinforced concrete are used in buildings of all sorts, underground structures, water tanks, wind turbine foundations and towers, offshore oil exploration and production structures, dams and bridges.

1.1 Mechanics of Reinforced Concrete

Concrete is very strong in compression, but weak in tension. As a result, cracks develop whenever loads, restrained shrinkage, or temperature changes give rise to tensile stresses in excess of the tensile strength of the concrete. In the plain concrete beam shown in found, the moments about point o due to the applied loads are resisted by an internal tension-compression couple involving tension in the concrete. An unreinforced beam fails very suddenly and completely when the first crack forms. In a reinforced concrete beam bars are embedded in such a way that the tension forces needed for moment equilibrium after the concrete cracks can be developed in the bars. Thus, the idea of reinforcing concrete with steel results in a material having the potential of resisting significant tensile stresses, which was hitherto impossible. Thus, the construction of load bearing flexural members, such as beams and slabs, become viable with this material (reinforced concrete). Its utility and versatility are achieved by combining the best features of concrete and steel. It can be seen from the table below that concrete and steel are less or more compatible.

Table 1-1 Commentary properties of concrete and steel

Property	Concrete	Steel
Strength in tension	Poor	Good
Strength in compression	Good	Good, but slender bars will buckle
Strength in shear	Fair	Good
Durability	Good	Corrodes if unprotected
Fire resistance	Good	Poor, loses rapid loss of strength at high temperature

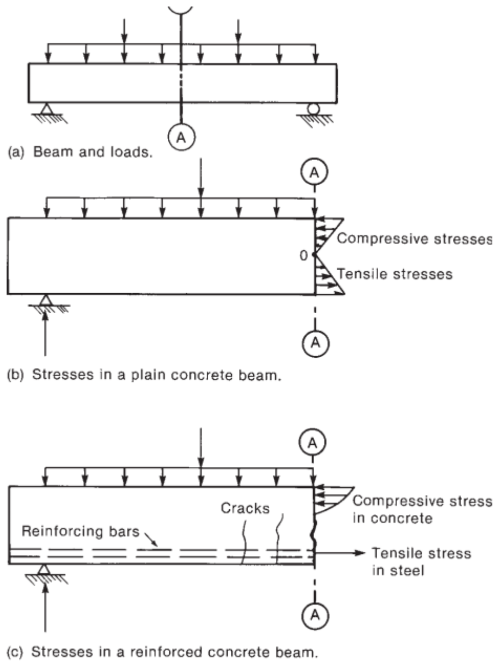


Figure 1-1 Plain and reinforcing concrete beams

The construction of reinforced concrete member involves building a form or mold in the shape of the member being built. The form must be strong enough to support the weight and hydrostatic pressure of the wet concrete, plus any forces applied to it by workers, concrete casting equipment, wind, and so on. The reinforcement is places in the form and held in place during concreting operation. After the concrete has reached sufficient strength, the forms can be removed.

1.2 Design Philosophies

Over the years, various design philosophies have evolved in different parts of the world, with regard to reinforced concrete design. A ‘design philosophy, is built up on a few fundamental premises, and is reflective of the way of thinking.

1.2.1 Working Stress Method

This was the traditional method of design not only for reinforced concrete, but also for structural steel and timber design. The conceptual basis of WSM is simple. The method basically assumes that the structure material behaves in a linear elastic manner, and that adequate safety can be ensured by suitably restricting the stress in the material induced by the expected ‘working loads’ on the structure. As the specified permissible (‘allowable’) stresses are kept well below the materials strength (i.e., the linear phase of the stress-strain curve), the assumption of linear elastic behavior is considered justifiable.

1.2.2 Ultimate Load Method (ULM)

In this method, the stress condition at the state of impending collapse of the structure is analyzed, and the non-linear stress-strain curves of concrete and steel are made use of. The design stresses used are the ultimate strength of materials and for safety the loads are magnified or scaled up by load factors, defined as the ratio of the ultimate load to working load. The ultimate load method makes it possible for different types of loads to be assigned different load factors under loading conditions, thereby overcoming the related shortcoming of WSM.

1.2.3 Limit states method (LSM)

Unlike WSM, which based calculations on service load conditions alone, and unlike ULM, which based calculations on ultimate load condition alone, LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimate loads and serviceability at working loads.

When a structure or structural element becomes unfit for its intended use, it is said to have reached a limit state. The limit states for reinforced concrete structures can be divided into three basic groups.

I. **Ultimate limit state: -** this involves a structural collapse of part or all of the structure. Such limit state should have a very low probability of occurrence, because it may lead to loss of life and major financial losses. The major ultimate limit states are loss of equilibrium of part or all of the structure as rigid body, rupture of critical parts of the structure, progressive collapse, formation of plastic mechanism, instability due to deformations of the structure, and fatigue.

II. **Serviceability limit states: -** these involve disruption of the functional use of the structure, but no collapse occurs. Because there is less danger of loss of life, a higher probability of occurrence is generally tolerated than in case of an ultimate limit state. The major serviceability limit states include excessive deformations, excessive crack widths, and undesirable vibrations.

III. **Special limit states:-** This class of limit states involve damage or failure due to abnormal conditions or abnormal loadings and includes:

- Damage or collapse in extreme earthquakes,
- Structural effects of fire, explosions, or vehicular collisions,
- Structural effects of corrosion or deterioration, and
- Long term physical or chemical instability (normally not a problem with concrete structures).

Among the three method of design, the limit state method of design (LSM) is the most widely used method of design in many countries building code standard. Our structural building is based on this method of design according to the revised Ethiopian building code standard ES EN 1992:2015 adopted from the European building code standard except for typical flat slabs. Flat slab design in ES EN 1992:2015) is based on American concrete institute (ACI).

1.3 Design Criteria According To ES EN 1992:2015

To analyze and design a structure, it is necessary to establish criteria's or requirements for determining whether a given structure is acceptable for use in a specified circumstance or for use directly as a design objective that must be met. ES EN 1990:2015 gives the following basic requirements during design of reinforced concrete or any structure for that matter.

- A structure shall be designed and executed un such a way that it will, during its intended life, with appropriate degree of reliability and in economical way sustain all actions and influences likely to occur during execution and use and remain fit for the use for which it is required (ES EN 1990:2015 section 2.1(1P).
- A structure shall be designed to have adequate structural resistance, serviceability, and durability (ES EN 1990:2015 section 2.1(2) P).
- In the case of fire, the structural resistance shall be adequate for the required period of time (ES EN 1990:2015 section 2.1(3)P).
- A structure shall be designed and executed in such a way that it will not be damaged by events such as explosion, impact, and the consequences of human

errors, to an extent disproportionate to the original cause (ES EN 1990:2015 section 2.1(4) P).

Potential damage shall be avoided or limited by appropriate choice of one or more of the followings (ES EN 1990:2015 section 2.1(5) P):

- Avoiding, eliminating or reducing the hazards to which a structure can be subjected;
- Selecting a structural form which has low sensitivity to the hazards considered;
- Selecting a structural form and design that can survive adequately the accidental removal of an individual or limited part of structure, or occurrence of acceptable localized damage;
- Avoiding as far as possible structural systems that can collapse without warning;
- Tying the structural member together.

According to ES EN 1990:2015 section 2.1(6)P the basic design criteria's or requirements listed above should be met by the choice of suitable material, by appropriate design and detailing, and by specifying control procedures for design, production, execution, and use relevant to particular project.

1.4 Design Situations

Design situations are situations that take the circumstances under which the structure is required to fulfill its function. According to ES EN 1990:2015 section 3.2 design situations are classified as follows:

I. Persistence design situations, which refer to conditions of normal use;

II. Transient design situation, which refer to temporary conditions applicable to the structure, e.g. during execution or repair;

III. Accidental situations, which refer to exceptional conditions applicable to the structure or to its exposure, e.g. to fire, exposure, impact or the consequences of localized failure;

IV. Seismic design situations, which refer conditions applicable to the structure when subjected to seismic events.

According section 3.2 of ES EN 1990:2015, Design situations shall be sufficiently Sevier and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure.

1.5 Material Properties

Two materials whose properties must be known are concrete and steel reinforcement. In case of concrete, the property with which we are primarily concerned is its compressive strength. For steel, however, it is its tensile strength capacity which is important.

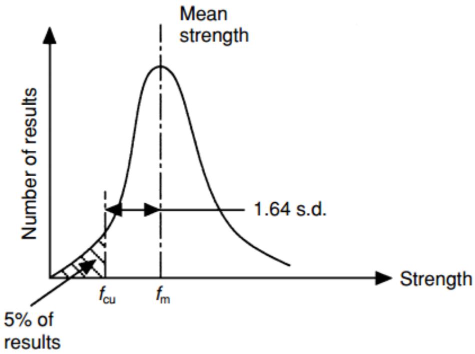


Figure 1-2 Normal frequency distribution of strengths

1.5.1 Concrete

Concrete is a very variable material, having a wide range of strength and stress-strain curves. A typical curve for concrete in compression. As the load is applied, the ratio between stressed and strains is approximately linear at first and the concrete behaves almost as an elastic material with virtually full recovery of displacement if the load is removed. Eventually, the curve is no longer linear and the concrete behaves more and more as plastic material. If the load were to be removed during the plastic range the recovery would no longer be complete and a permanent deformation would remain. The ultimate strain for most structural concrete tends to be a constant value approximately 0.0035, although this is likely to reduce for concretes with cube strengths above about 50 N/mm^2. ES EN 1992-1-1:2015 table 3.1 'design of concrete structures' recommends values for ultimate strain in such case.

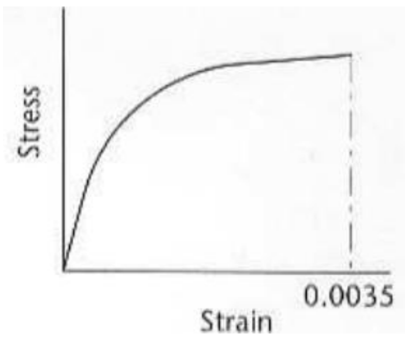


Figure 1-3 stress-strain curve for concrete under short term loading

Concrete generally increases its strength with age. This characteristic is illustrated by the graph in figure 1-4 which shows how the increase is rapid at first, becoming more gradual later. The precise relationship will depend upon the type of the cement used. Some codes of practice allow the concrete strength used in design to be varied according to the age of concrete when it supports the design load. In ES EN 1992-1-1:2015 section 3.1.2 the strength of concrete is based on the characteristic cylindrical strength f_{ck} determined at 28 days with a maximum value of C_{max} (C90/105).

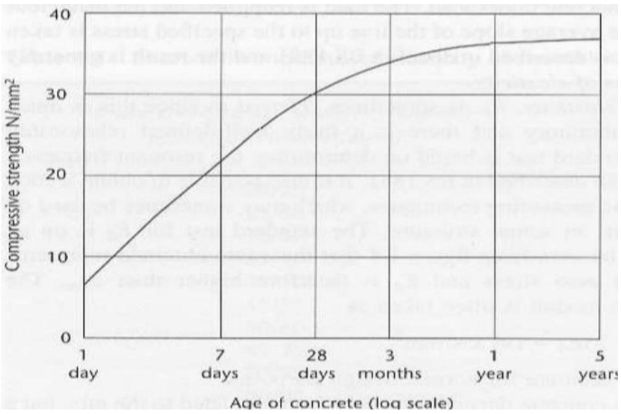


Figure 1-4 Relationship compressive strength of concrete and its age

1.5.1.1 Stress-strain curves for design of concrete cross-sections

For concrete, three possibilities are described in ES EN 1992-1-1:2015 section 3.1.7. The preferred idealization is the parabolic- rectangular diagram, but a bi-linear diagram and a rectangular diagram are also permitted.

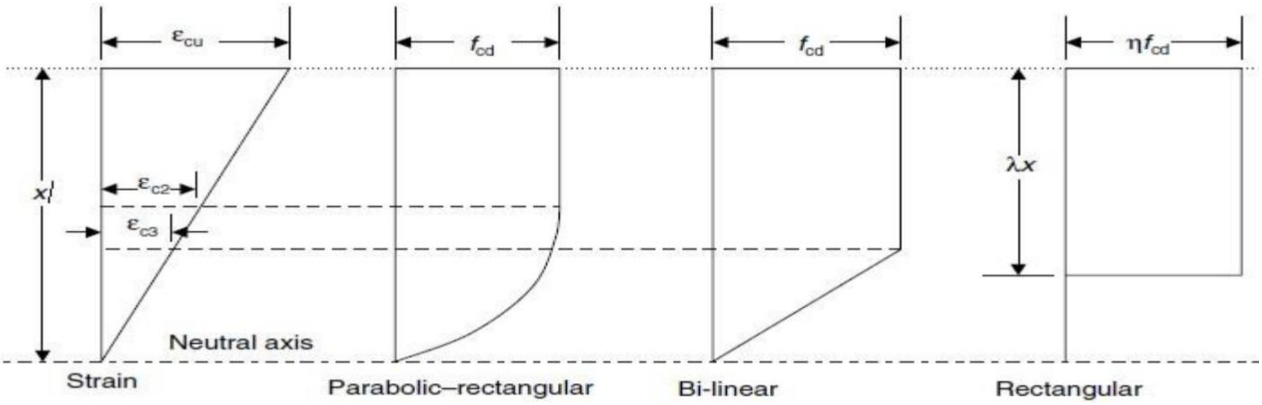


Figure 1-5 Idealized stress-strain distributions

The design concrete strength, f_{cd} is obtained by dividing the characteristic cylindrical strength, f_{ck}, of concrete by the partial safety factor for concrete, γ_c. However, the design stress is obtained by applying a reduction factor, α_{cc}, to the design strength. This is given in ES EN 1992-1-1:2015 expression 3.15 as follows:

$$fcd = \frac{\alpha_{cc} fck}{\gamma_c}$$

Where:

γ_c Is the partial safety factor for concrete, and

α_{cc} Is the coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied (recommended value is 0.85).

The introduction of the factor, α_{cc}, is not only related long-term effects on the compressive strength and of unfavorable effects resulting from the way load is applied, but also it is related to the idealization and design stress-strain diagram used for design of concrete.

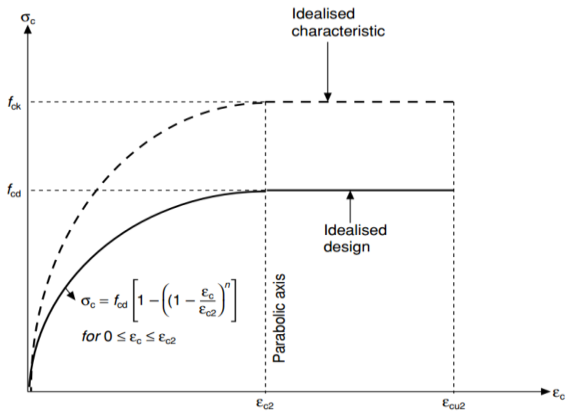


Figure 1-6 Parabolic-rectangular stress-strain diagram for concrete ($f_{ck} \leq 50$ N/mm)

The value of design tensile strength, f_{ctd}, is defined in ES EN 1992-1-1:2015 expression 3.16 as follows:

$$f_{ctd} = \frac{\alpha_{ct} * fctk, 0.05}{\gamma c}$$

Where:

γc Is the partial safety factor for concrete, and

α_{ct} Is the coefficient taking account of long term effects on the tensile strength and of the unfavorable effects, resulting from the way the load is applied

Depending on the exposure class of concrete assumed (XC1, dry or permanently wet), form ES EN 1992-1-1:2015, annex E, table E.1N, for the exposure class of concrete the minimum concrete grade of C20/25. But we have adopted a concrete grade of C25/30 for slabs, beams and columns and C30/35 for the foundation for this project.

C25/30

$$\text{Design concrete strength, fcd} = \frac{0.85 * 25}{1.5}$$

$$\text{Design concrete strength, fcd} = 14.167\ \text{MPa}$$

$$\text{Design tensile strength, } f_{ctd} = \frac{1 * 1.8}{1.5}$$

$$\text{Design tensile strenght, } f_{ctd} = 1.2\text{MPa}$$

C30/35

$$\text{Design concrete strength, fcd} = \frac{0.85 * 30}{1.5}$$

$$\text{Design concrete strength, fcd} = 17\ \text{MPa}$$

$$\text{Design tensile strength, } f_{ctd} = \frac{1 * 2}{1.5}$$

$$\text{Design tensile strenght, } f_{ctd} = 1.33\text{MPa}$$

1.5.2 Reinforcing steel

As mentioned earlier in this chapter, because concrete is weak in tension, it is reinforced with steel bars or wires that can resist the tensile stresses. The most common types of reinforcements for non pre stressed members are hot rolled deformed bars shows a typical stress-strain curve for hot rolled high yield steel (commonly used for reinforcement). Steel behaves as an elastic material, with the strain proportional to the stress up to the yield, at which point there is sudden increase in strain with no change in stress. After the yield point, this becomes a plastic material and the strain increases rapidly up to the ultimate value.

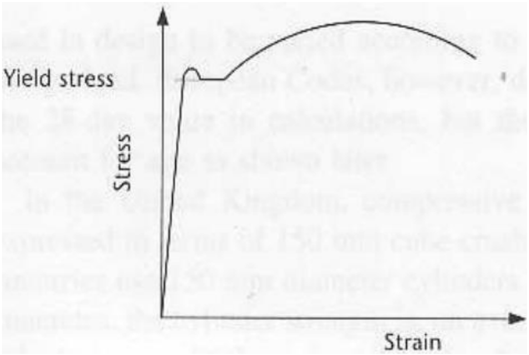


Figure 1-7 Stress-strain diagram for hot rolled high yielding reinforcement bars

1.5.2.1 Stress-strain curves for design of reinforcement

For steel, the recommended idealized and design stress-strain diagrams are given in ES EN 1992-1-1:2015 figure 3.8 as follows:

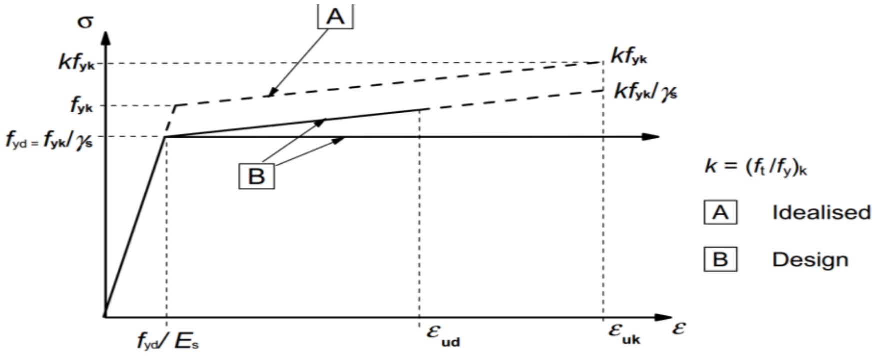


Figure 1-8 Idealized and design stress- strain diagram for reinforcement bars

The design steel stresses, f_{yd}, are derived from the idealized (characteristic) stress, f_{yk}, by dividing by the partial safety factor for steel, γ_s. This is given in ES EN 1992-1-1:2015 section 3.2.7 by assuming either of the following:

a) An inclined top branch with a strain of ε_{ud} (0.9 ε_{uk}) and maximum stress of kf_{yk}/γ_s, the recommended value of $k = (ft/fy)k$.

b) A horizontal top branch without the need to check the strain limit, the design stress should not exceed $^{fyk}/_{\gamma s}$.

In this project we have used the assumption given in B for simplicity and convenient. On our project medium ductility S400 steel grade reinforcement have been adopted.

$$\text{Design steel stress, fyd} = \frac{400}{1.15} = 347.83\text{Mpa}.$$

$$\text{Modulus of elasticity, Es} = 195\text{MPa}$$

1.6 General Description of the Building

The building is a G+10 reinforced concrete frame structure located in Addis Ababa (Figure 1-9). The building is composed of solid slab system to support the loads due to occupancy. Form the sectional elevation view of the building, the stairs type is stairs with quarter-turn landing. The elevator shaft is located at the middle of the stairs three flights.

The geometry of the roof is a duo pitch roof type. Wind load analysis is carried out for each cases of wind direction and the worst-case scenario is considered for design. In addition, galvanized steel sheet from the Kality metal products factory manufacturing manual is used as a roof cover. For the case of purlin, truss members eucalyptus members may be used since the loading is very small, but for academic purpose to practice steel structures analysis and design we have used steel members.

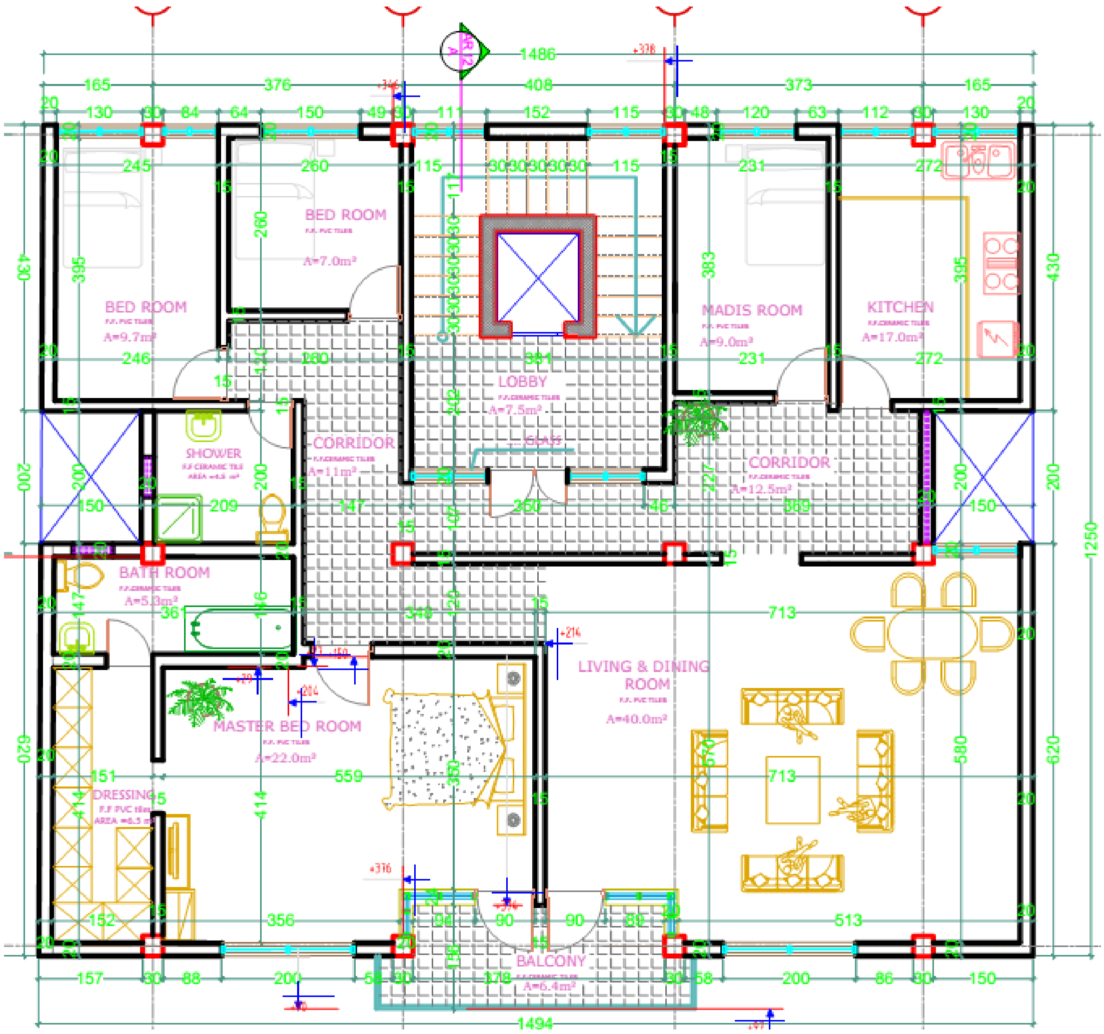


Figure 1-9 Typical floor plan of the building

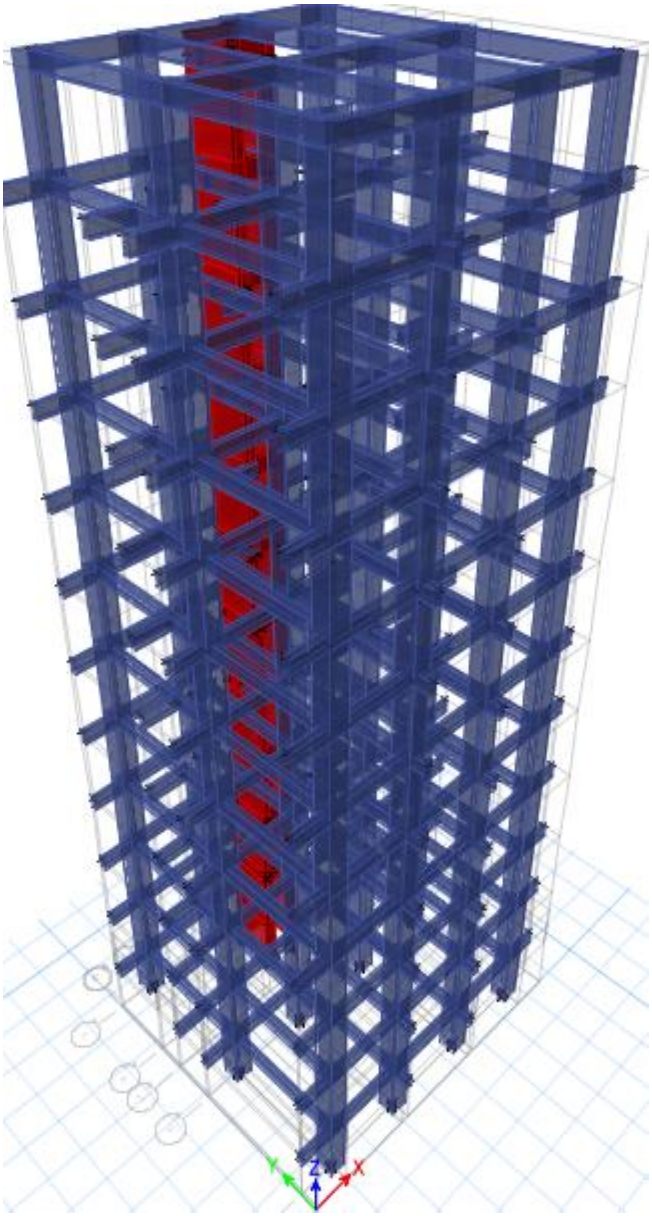


Figure 1-10 3D model of the building from ETABS 2016v16.1.0

2 Slab Analysis and Design

2.1 Introduction

A reinforced concrete slab is a broad, flat plate, usually horizontal, with top and bottom surfaces parallel or nearly so. It may be supported by reinforced beams (and is usually cast monolithically with such beams), by masonry or reinforced concrete walls, by structural steel members, directly by columns, or continuously by ground. Slabs may be supported on two opposite sides only in which case the structural action of the slab is essentially one-way, the load being carried by the slab in the direction perpendicular to the supporting beams. There may be beams in all the four sides, so that two-way slab action is obtained. Intermediate beams may be provided if the ratio of the length two width of the one panel is greater than two, most of the load is carried in the short direction to the supporting beams and one-way action is obtained in effect even though the supports are provided on all sides.

One-way slabs transfer the imposed loads in one direction only. They may be supported on two opposite sides only in which the structural action is essentially one-way, the loads being carried in direction perpendicular to the supporting beams or walls. But rectangular slabs often have such proportions and supports (e.g., relatively deep, stiff monolithic concrete beams) that result in two-way action at any point, such slabs are curved in both directions resulting in biaxial bending moments. It is convenient to think of such slabs as consisting of two sets of parallel strips, in each direction and intersecting each other. So part of the load is carried by one set and the remainder by the other.

2.2 Design and Analysis of Solid Slab

2.2.1 Type of slab

A slab subjected to dominantly uniformly distributed loads may be considered to be one-way spanning if either:

- It possesses two free (unsupported) and sensibly parallel edges, or
- It is the central part of a sensibly rectangular slab supported on four edges with a ratio of the longer to shorter span greater than 2.(ES EN 1992-1-1:2015,section 5.3(5))

The first step in the design of floor slab is classifying the slab whether it is one way or two-way slab by determining the Ly/Lx.

Where

Ly Is the longer dimension of the panel, and

Lx Is the smaller dimension of the panel.

If Ly/Lx≤2……….two way slab

If Ly/Lx >2………one way slab.

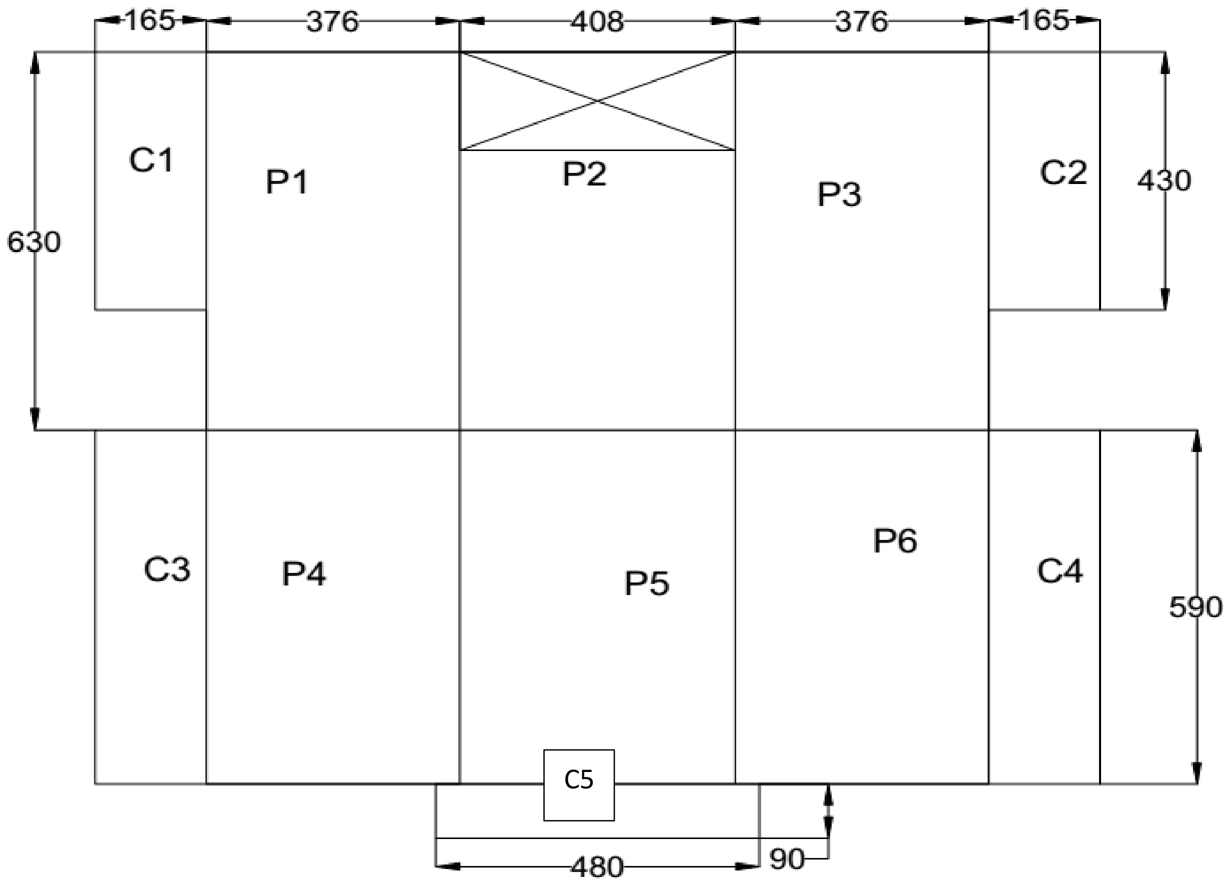


Figure 2-1 Panels with their assigned name

Table 2-1Determination of type of slab

Panel	Ly(m)	Lx(m)	Ly/Lx	Type of slab
P1	6.3	3.76	1.67	Two-way
P2	4.08	3.09	1.32	Two-way
P3	6.3	3.76	1.67	Two-way
P4	5.9	3.76	1.57	Two-way
P5	5.9	4.08	1.44	Two-way
P6	5.9	3.76	1.57	Two-way
C1	4.3	1.65	2.6	One-way
C2	4.3	1.65	2.6	One-way
C3	5.9	1.65	3.57	One-way
C4	5.9	1.65	3.57	One-way
C5	4.8	0.9	5.33	One-way

The cantilever parts of the slab are one way and they are analyzed as one-way slab by taking one-meter width, and the two-way slabs are analyzed using coefficient method.

2.2.2 Design for cover

The recommended procedure for determination of the minimum concrete cover is given by

ES EN 1992:2015 section 4 Expression (4.2).

$$C_{min} = max \begin{cases} Cmin, bond \\ Cmin, dur + \Delta cdur - \Delta cduc, st - \Delta cdur, add \\ 10mm \end{cases}$$

Where

$C_{min,bond}$ is the minimum cover due to the cover requirement,

$C_{min,dur}$ is minimum cover due to environmental conditions,

$\Delta_{c,dur}$ is additive safety element,

$\Delta_{cdur,st}$ is the reduction of minimum cover for the use of stainless steel, and

$\Delta_{cdur,add}$ is the reduction of minimum cover additional protection.

We have used

Δc,dur=0 … ES EN 1992-1-1:2015 section 4.4.1.2(6),(since we don't have additional safety elements).

Δc,duct,st=0….. ES EN 1992-1-1:2015 section 4.4.1.2(7), (since the type of reinforcement that we have used is not stainless steel).

Δcdur, add =0…ES EN 1992-1-1:2015 section 4.4.1.2(7), (since we have not used additional protection).

$$C_{min} = \max \begin{cases} Cmin, bond \\ Cmin, dur \\ 10mm \end{cases}$$

In order to transmit bond forces safely and to ensure adequate compaction the minimum cover should not be less than $C_{min,bond.}$

According to ES EN-1992-1-1:2015, Table 4.2 the value of $C_{min,bond}$ for ordinary bond requirement should be equals with the bar diameter.

N.B. if the nominal aggregate size is greater than 32mm, the value of $C_{min,bond}$ should be increased by 5mm.

The minimum cover required for bond is given by ES EN 1992-1-1:2015, Table 4.2.

$C_{min,bond}$=bar diameter

$C_{min,dur}$ is the minimum cover value for reinforcement in normal weight concrete take account of the exposure classes and the structural classes. We are going to design our building

- Design service life of 50 years.
- Normal quality control
- Maximum aggregate size of 20mm
- 1HR fire protection

We have chosen an exposure class of XC1 which is for dry environment…….ES EN 1992-1-1:2015 Table 4.1.

According to ES EN 1992-1-1:2015, section 4.4.1.2(5) NOTES, the recommended structural class for service life of 50 years is 4. And the concrete compressive strengths are given ES EN 1991-1-1:2015, Annex E.

According to ES EN 1992-1-1:2015, Annex E, Table E.1N, for exposure class of XC1 take a minimum concrete grade of C-20/25. So we have used a concrete grade of C-25/30. The diameter of reinforcement bar that we are going to use for the slab is 8mm.

I. Cover for bond and durability

According to ES EN 1992-1-1:2015, Table 4.4N for structural class of four and exposure class of xc1 the minimum cover for durability is 15mm

The type of steel on our slab system is ordinary, therefore the minimum cover for bond should be determined from ES EN 1992-1-1:2015 Table 4.2 which is equals with the bar diameter which is 8mm.

$$Cmin, b = Bar\ diameter\ (since\ the\ maximum\ aggregate\ size\ is\ less\ than\ 32mm)$$

$$Cmin, b = 8mm$$

$$Cmin = max\begin{cases} 8mm \\ 15mm \\ 10mm \end{cases}$$

$$Cmin = 15mm$$

The nominal cover should be computed according to ES EN 1992-1-1:2015, Expression 4.1 by adding the minimum cover for bond and durability ($C_{min)}$ and the allowance in design for deviation (Δc, dev).

$$Cnom = \ cmin + \ \Delta cdev$$

The required minimum cover shall be increased by the accepted negative deviation given in the standard for execution by the amount of Δc, dev. The recommended value for allowance in design for deviation is 10mm. Where fabrication is subjected to quality assurance system, in which the monitoring includes measurement of the concrete cover, the allowance in design for deviation may be reduced.

10mm≥Δc,dev≥5mm.

Where it can be assured that a very accurate measurement device is used for monitoring and non-confirming members are rejected the allowance in design for deviation, Δc,dev may be reduced.

10mm≥Δc,dev≥0mm.

For our case we are going to use the allowance in design deviation to be 10mm which is given by ES EN 1992-1-1:2015, section 4.4.13(1) P because we don't have accurate measurement devices and quality assurance system.

$$\Delta c, dev = 10mm$$

$$Cnom = \ cmin + \ \Delta c, dev$$

$$cnom = 15mm + 10mm$$

$$cnom = 25mm$$

II. Cover for 1 HR fire protection

Where mechanical resistance in the case of fire is required, concrete structures shall be designed and constructed in such a way that they maintain their load bearing function during the relevant fire exposure. (ES EN-1992-1-2:2015 section 2.1.1)

For REI=60, D_{min}=80mm<D=170mm (for the two-way slab) …. Ok! … ES EN 1992-1-2:2015, Table 5.8.

$$amin = 10mm < \ Cnom \ = 25mm$$

For REI=60, Dmin=80mm<D=210mm (for the cantilever parts). Ok! … ES EN 1992-1-2:2015, Table 5.8.

$$amin = 10mm < \ Cnom = 25mm$$

Therefore, use a concrete cover of 25mm for the slab.

2.2.3 Slab Depth Determination (Deflection requirement)

According to ES EN 1992-1-1:2015 section 7.4 the minimum depth of the slab should satisfy for the serviceability requirement

The deformation of a member or structure shall not be such that it adversely affects its proper functioning or appearance. Reinforced concrete beams or slabs in buildings are dimensioned so that they comply with the limits of span to depth ratio. Their deflections may be considered as not exceeding the limits set out in deflections that could damage adjacent parts of the structure should be limited for the deflection after construction span/500 is normally an appropriate limit for quasi-permanent loads. Other limits may be considered, depending on the sensitivity of adjacent parts.

The appearance and general utility of the structure may be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250. The sag is assessed relative to the supports. Pre-camber may be used to compensate for some or all of the deflection but any upward deflection incorporated in the formwork should not generally exceed span/250.

The limit state of deformation may be checked by either:

- By limiting the span/depth ratio, according to the equations shown below
- By comparing a calculated deflection with a limit value

The actual deformations may differ from the estimated values, particularly if the values of applied moments are close to the cracking moment. The differences will depend on the dispersion of the material properties, on the environmental conditions, on the load history, on the restraints at the supports, ground conditions, etc.

We have used the limited span to depth ratio according to equation 1 or 2 shown below the limiting span/depth ratio may be estimated using Expressions 1 and 2(ES EN 1992-1-1:2015 section 7.4.2) below and multiplying this by correction factors to allow for the type of reinforcement used and other variables. And it is given by ES EN 1992-1-1:2015 expression (7.16a).

$$\frac{l}{d} = k\left[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} + 3.2\sqrt{fck} * \left(\frac{\rho o}{\rho} - 1\right)^{\frac{3}{2}}\right] \; if \; \rho \le \rho o$$

$$\frac{l}{d} = k\left[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho - \rho'} + \frac{1}{12}\sqrt{fck} * \left(\frac{\rho'}{\rho o}\right)^{\frac{1}{2}}\right] . if \; \rho > \rho o$$

Where:

l/d Is the limit span/depth;

K Is the factor to take into account the different structural systems;

ρ0 Is the reference reinforcement ratio = $\sqrt{\text{fck}} * 10^{-3}$;

P Is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers);

ρ´ Is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers); and

Fck Is cylindrical characteristics compressive strength of the concrete in MPa units

According to ES EN 1992-1-1:2015 section 7.4.2, table 7.4N, the recommended values of k is given according to the structural system and the value of ρ for concrete slightly stressed is 0.5%

The reference reinforcement ratio (ρo) for cylindrical characteristics concrete strength of 25Mpa (c-20/25) is

$\rho o = \sqrt{25} * 10^{-3} = 5x10^{-3}$

ρo=0.5%...............ρ=ρ0, therefore use Expression (7.16a) of ES EN 1992-1-1:2015

$$\frac{l}{d} = k\left[11 + 1.5\sqrt{23} * \frac{0.5\%}{0.5\%} + 3.2\sqrt{25} * \left(\frac{0.5\%}{0.5\%} - 1\right)^{\frac{3}{2}}\right]$$

$$l/_{d} = k[11 + 1.5 * 5] = 18.5k$$

Since we are using a reinforcement bar with characteristic yield strength of 400Mpa (fyk=400Mpa), the span to depth ratio should be multiplied according ES EN 1992-1-1:2015by a factor 500/400.

$$l/_{d} = 18.5k * (\tfrac{500}{400}) = 23.124k$$

$$d = \frac{l}{23.125k}$$

Table 2-2 Effective depth of the slab with their respective panels

Slab	Structural system	K	Span length(m)	d(cm)
Panel1	End span	1.3	3.76	12.5
Panel2	End span	1.3	4.08	13.57
Panel3	End span	1.3	3.76	12.5
Panel4	End span	1.3	3.76	12.5
Panel5	Interior Span	1.5	4.08	11.76
Panel6	End span	1.3	3.76	12.5
C1	Cantilever	0.4	1.65	17.8
C2	Cantilever	0.4	1.65	17.8
C3	Cantilever	0.4	1.65	17.8
C4	Cantilever	0.4	1.65	17.8
C5	Cantilever	0.4	0.9	10.2

According to the above calculation the governing effective depth for the panels is 135.7mm, whereas for the cantilever slab the governing effective depth is 178mm.

$$Slab\ depth\ for\ the\ two\ way\ panels = d + (bar\ diameter)/2 + concrete\ cover$$

$$D = 135.7mm + 8mm/2 + 25mm$$

$$D = 164.7mm \approx 170mm$$

$$Slab\ depth\ for\ the\ cantilever = d + (bar\ diameter)/2 + concrete\ cover$$

$$D = 178mm + 8mm/2 + 25mm$$

$$D = 207mm \approx 210mm$$

2.2.4 Slab load Determination

Loads on slabs are both permanent loads and imposed loads. The permanent loads are loads that arise from the self-weight of the slab, cement screed, floor finishing material, plastering and ceiling, partition walls and partition wall finishing materials. The imposed loads are determined based on the functional use of the building and it accounts for loads that do, or can, change over time, such as people walking around the building (occupancy) or movable objects such as furniture. Imposed loads are variable as they depend on usage and capacity. However, design code ES EN 1992-1-1:2015 can provide equivalent loads for various structures based on their functional use.

1. **Permanent load (sample calculation for panel 1)**

Partition loads are wall loads that are on top of the slab and they are calculated for their thickness of the wall and their finishing material as follows;

Partition wall weight = $t_w * L * H * \gamma_{HCB} + t_p * L * H * \gamma_{pL}$

Where:

t_w	Thickness of the partition wall(m);
L	Total length of the partition wall(m);
H	Height of the partition wall(m);
γ_{HCB}	Unit weight of hallow concrete block (HCB) in KN/m^3;
t_p	Thickness of the plastering material(m); and
γ_{pL}	Unit weight of the plastering material in KN/m^3.

On panel 1 of the slab system we have a thickness of 2cm plastering on both sides of the wall and 8.5m long 15cm thick partition wall. The unit weight of the HCB walls and the plastering material (cement screed) is 14 KN/m^3 and 17 KN/m^3 respectively. Then the weight of the partition wall on panel on is calculated as:

Partition wall weight on panel 1= 8.5m*3.03m*0.15m*14 $\frac{\text{KN}}{\text{m3}}$ + 8.5m*3.03m*0.02m*2*17 $\frac{\text{KN}}{\text{m3}}$

Partition wall weight on panel 1= 71.6KN

Partition wall weight on panel 1 per m^2 of slab = 3 KN/m^2

Table 2-3Sample dead load calculation on panel 1

Material	Unit weight ($\frac{kN}{m3}$)	Thickness (m)	Load ($\frac{kN}{m2}$)
Ceramic finishing	23	0.01	0.23
Cement screed	23	0.03	0.69
Self-weight	25	0.17	4.25
Gypsum plastering	17	0.02	0.34
Partition wall	Already calculated above		3
Total load			8.51

For the rest of the panels and cantilevers of the permanent load calculation process is the same as the process for pane 1 of the slab system. The permanent loads for the rest of the panels and cantilever slab is summarized on the table below.

Table 2-4 summery calculation of dead loads on slabs

Panel	Dead load ($\frac{KN}{m2}$)
1	8.51
2	7.47
3	7.67
4	6.72
5	7.107
6	5.365
C1	6.365
C2	6.51
C3	6.51
C4	5.365
C5	6.51

2. **Imposed load**

According to ES EN 1991-1-1:2015 section 6.1.2 for the determination of the imposed loads, floor and roof areas in buildings should be sub-divided into categories according to their functional use Our building is residential G+10 building. Hence Areas for domestic and residential activities for example Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets are classified under category A according ES EN 1991-1-1:2015 table 6.1.

The categories of loaded areas, as specified in Table 6.1, shall be designed by using characteristic values qk (uniformly distributed load) and Qk (concentrated load). Where qk is intended for determination of general effects and Qk for local effects According to ES EN 1991-1-1:2015 table 6.2 Imposed loads on floors, balconies and stairs in buildings for category A is as shown on the table below

Table 2-5 Imposed loads under category A building

Load area	qk($\frac{KN}{m2}$)	Qk(KN)
Floor	1.5-2.0	2.0-3.0
Stair	2.0-4.0	2.0-4.0
Balconies	2.5-4.0	2.0-3.0

From the table above we took imposed load of 2 KN/m2 for the slab system.

3. Design load

After obtaining the permanent and imposed loads, the design loads are calculated based on ES EN 1990:2015 Annex A1 (normative) – application for buildings. Based on ES EN 1990:2015 Annex A1.3 (ultimate limit states) – design values of actions in persistent and transient design situations indicates that "the design of structural members (STR - defined as Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs (on ES EN 1990:2015 section 6.4.1(B))) not involving geotechnical actions should be verified using the design values of actions from table A1.2(B)". Therefore, the design action is given by:

$$E_d = 1.35G_k + 1.5Q_k$$

Where:

G_k Is permanent load in KN/m^2, and

Q_k Is imposed load KN/m^2.

Table 2-6 summery calculation of design loads on slabs

Panel	Permanent load (Gk)	Imposed load (Qk)	Design load (Ed)
1	8.51	2	14.48
2	7.47	2	13.08
3	7.67	2	13.35
4	6.72	2	12.07
5	7.101	2	12.6
6	5.365	2	10.24
C1	6.365	2	12.73
C2	6.51	2	13.02
C3	6.51	2	13.02
C4	6.365	2	12.73
C5	6.51	2	13.02

2.2.5 Analysis of two way slabs

The precise determination of moments in two way slabs with various conditions of continuity at the supported edges is mathematically formidable and not suited for design. Various simplified methods have been adopted for determining moments, shear and reactions. The methods that are accepted because of satisfying equilibrium conditions are:

a) Coefficient method of analysis
b) Yield line method of analysis
c) Strip method of analysis

2.2.5.1 Analysis of two-way slab using coefficient method

From the above methods the most widely used and the easiest method of analysis is the coefficient method of analysis. This method of analysis is an elastic method of analysis that uses table moment coefficient based on the support edge condition to determine the maximum design moments that apply only to the middle strips and no redistribution is to be made. In slabs where the corners are prevented from lifting, and provision for torsion is made, the maximum design moments per unit width are given by the following equations.

$$Mx = \beta x * Wd * lx^2$$

$$My = \beta y * Wd * lx^2$$

Where:

Mx and My	Bending moments in the x and y direction of slab,
βx and βy	Moment coefficient based on supported edge conditions,
Wd	Design load on the slab (KN/m^2), and
lx	Length of the shorter span (m).

Sample calculation for panel 1 (support moment)

$$Msx = \beta sx * Wd * lx^2 = 0.08412*14.48*3.76^2 = 17.22 \text{ KNm}$$

$$Msy = \beta sy * Wd * lx^2 = 0.045*14.48*3.76^2 = 9.21 \text{ KNm}$$

Sample calculation for panel 1 (span moment)

$$Mfx = \beta fx * Wd * lx^2 = 0.063*14.48*3.76^2 = 12.9 \text{ KNm}$$

$$Mfy = \beta fy * Wd * lx^2 = 0.034*14.48*3.76^2 = 6.96 \text{ KNm}$$

Table 2-7 summery calculation of unadjusted span and support bending moments

Panel	ly / lx	βsx	βfx	βsy	βfy	Wd	Msx	Mfx	Msy	Mfy
1	1.67	0.0841	0.063	0.045	0.034	14.48	17.22	12.9	9.21	6.96
2	1.54	0.0588	0.0433	0.037	0.028	13.08	12.8	9.43	14.6	6.1
3	1.67	0.0841	0.063	0.045	0.034	13.35	15.88	11.89	8.5	6.42
4	1.57	0.0805	0.0605	0.045	0.034	12.07	13.74	10.32	7.65	5.8
5	1.44	0.0562	0.0418	0.037	0.0208	12.6	11.79	8.77	7.76	5.8
6	1.57	0.0805	0.0605	0.045	0.034	10.24	11.66	8.76	6.5	4.3

2.2.6 Analysis of cantilever sabs

Cantilever slabs are slabs supported only on one side of the four side of the slab. They are analyzed as one way slabs by taking 1m wide strip of cantilever beam. The design load of the partition load at the edge of the slab can be computed using the following formula.

$$Pd = 1.35(H_{HCB} * t_{HCB} * b) * \gamma_{HCB}$$

Where:

Pd Design wall load at the edge of cantilever slab (KN),

H_{HCB} Height of the wall (m),

t_{HCB} Thickness of the wall (m),

b Width of the strip 1m, and

γ_{HCB} Unit weight of hallow concrete block (HCB) in KN/m^3.

Sample calculation for cantilever 1 (C1)

Pd =1.35(0.45m*0.2*1) *14

Pd =1.7 KN

Bending moment = 1.7*1.65 + 12.73*1.65*1.65*0.5

Bending moment = 20.133 KNm

Table 2-8 summery bending moment calculation for cantilever slabs

Panel	Bending moment (KN/m)
C1	20.133
C2	20.53
C3	20.53
C4	20.14
C5	6.803

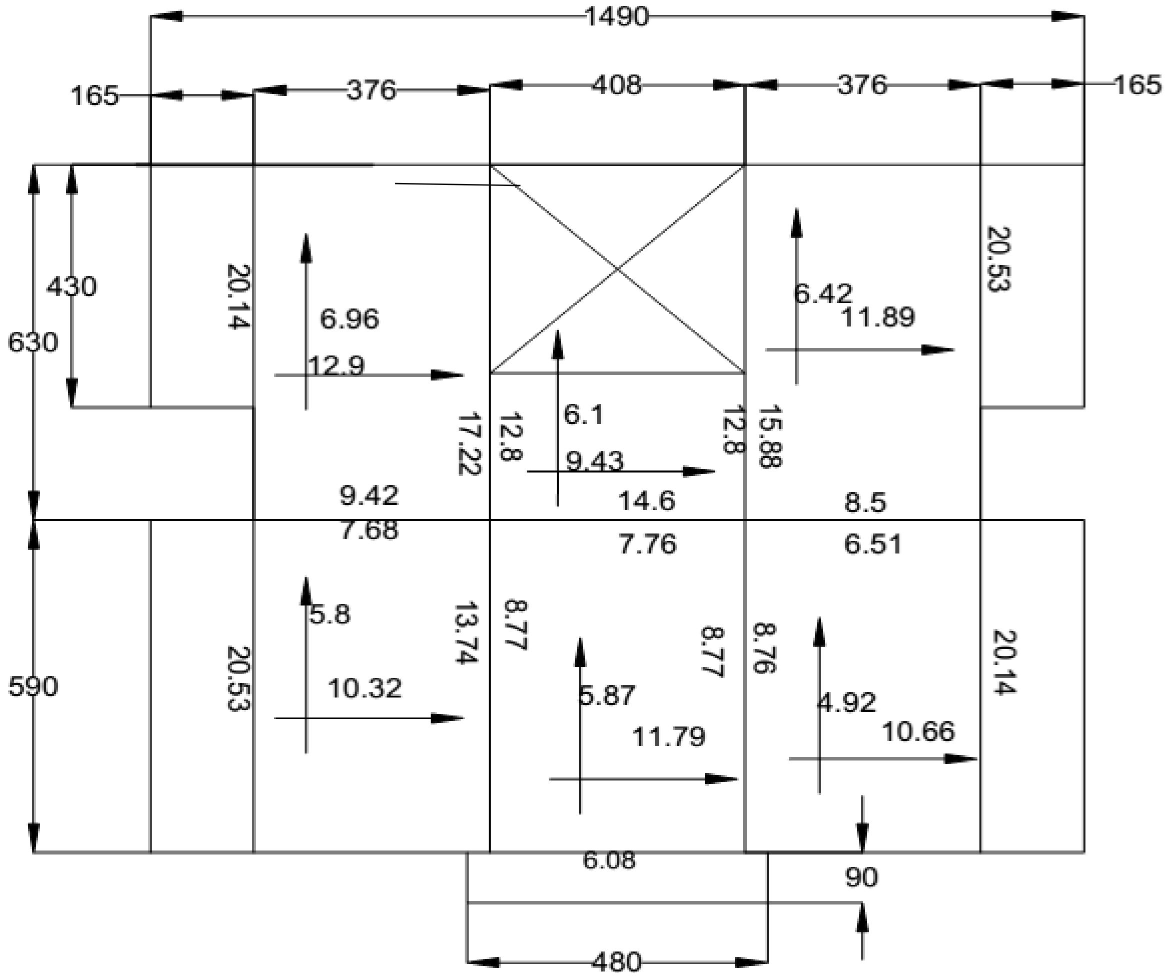


Figure 2-2 unadjusted span and support moment on the slab

2.2.7 Restrained slab with unequal conditions at adjacent panels

Bending moment at a common support, obtained by corresponded the two adjacent panels in isolation may differ significantly, because of differing edge condition at far supports or differing span length or loading.

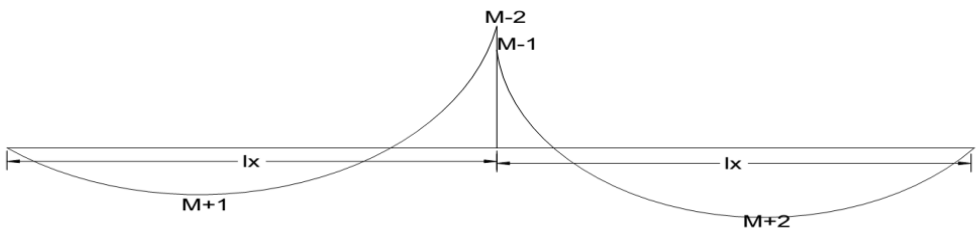


Figure 2-3 Unequal moments at adjacent panels

Obtain support moment for the adjacent panels, treating M-1 and M-2 as fixed end moments, the moments may be distributed in proportion to the stiffness of span lx in each adjacent panels thus revised bending moments M-B' may be obtained for the support over B.

The span moment in each adjacent panel should be calculated

$$M'_{+1} = (M_1 + M_{+1}) - M_{-B}$$

$$M'_{+2} = (M_2 + M_{-2} + M_{+2}) - M'_{-B} - M_{-2}$$

Moment Adjustment is required when ΔM>10%. Unless we should take the average and distribute it

On axis "B"

$$\Delta M = \frac{(17.22-12.8)}{17.22} * 100\% = 25.66\% \quad \text{and } \Delta M = \frac{(13.74-8.77)}{13.74} * 100\% = 36.17\%$$

On axis "C"

$$\Delta M = \frac{(15.88-12.8)}{15.88} * 100\% = 19.4\%, \text{ and } \Delta M = \frac{(8.77-8.76)}{8.77} * 100\% = 0.11\%$$

On axis "2"

- Between axis "A" and "B"

$$\Delta M = \frac{(9.42-7.68)}{9.42} * 100\% = 22.6\%$$

- Between axis "B" and "C"

$$\Delta M = \frac{(14.6-7.76)}{14.6} * 100\% = 46.8\%$$

- Between axis "C" and "D"

$$\Delta M = \frac{(8.5-6.51)}{8.5} * 100\% = 23.41\%$$

Moment adjustment on axis "B" between axis "2" and "3"

- $\text{stiffness(ki)} = \frac{1}{l_i}$ and $\text{stiffness(kj)} = \frac{1}{l_j}$
- $\text{Distribution factor(DFi)} = \frac{K_i}{K_i+K_j}$ and
- $\text{Distribution factor(DFj)} = K_j/(K_i + K_j)$

Table 2-9 Adjusted support moment on axis 'B'

Length(m)	3.76	4.08
Stiffness(K)	1/3.76	1/4.08
Distribution factor (Df)	0.52	0.48
Support moment (M)	17.22	12.8
change in support moment (ΔM)	4.42	
Df*ΔM	-2.3	2.12
M+Df*ΔM	14.92	14.92

Adjusted span moment between axis "2" and "3"

$$M1 = (12.9 + 17.22) - 14.92 = 15.2KNm.$$

$$M2 = (9.43 + 12.8) - 14.92 = 7.31KNm.$$

Table 2-10 Moment adjustment on axis "C" between axis "2" and "3"

Length(m)	4.08	3.76
Stiffness(k)	1/4.08	1/3.76
Distribution factor(Df)	0.48	0.52
Support moment (M)	12.8	15.88
change in support moment (ΔM)	3.08	
Df*ΔM	1.48	-1.60
M+Df*ΔM	14.28	14.28

Adjusted span moment between axis "2" and "3"

$$M1 = (9.43 + 12.8) - 14.28 = 7.31KNm.$$

$$M2 = (11.89 + 15.88) - 14.28 = 13.49KNm.$$

By following the same procedure, we have computed the adjusted support and span moment. And the results are tabulated as shown below.

Table 2-11 Adjusted support moment

Axis	Adjusted support moment in KNm
On axis "B" between axis "1" and "2"	11.16
On axis "C" between axis "1" and "2"	8.77
On axis "2" between axis "A" and "B"	8.59
On axis "2" between axis "B" and "C"	11.32
On axis "2" between axis "C" and "D"	7.55

Table 2-12 Adjusted span moment

Axis	M1(KNm)	M2(KNm)
On axis “B” between axis “1” and “2”	12.9	9.4
On axis “C” between axis “1” and “2”	9.4	10.66
On axis “2” between axis “A” and “B”	7.79	4.89
On axis “2” between axis “B” and “C”	9.38	2.31
On axis “2” between axis “C” and “D”	7.37	3.88

Where M1 and M2 stands for the span moments on the left and right of the support respectively

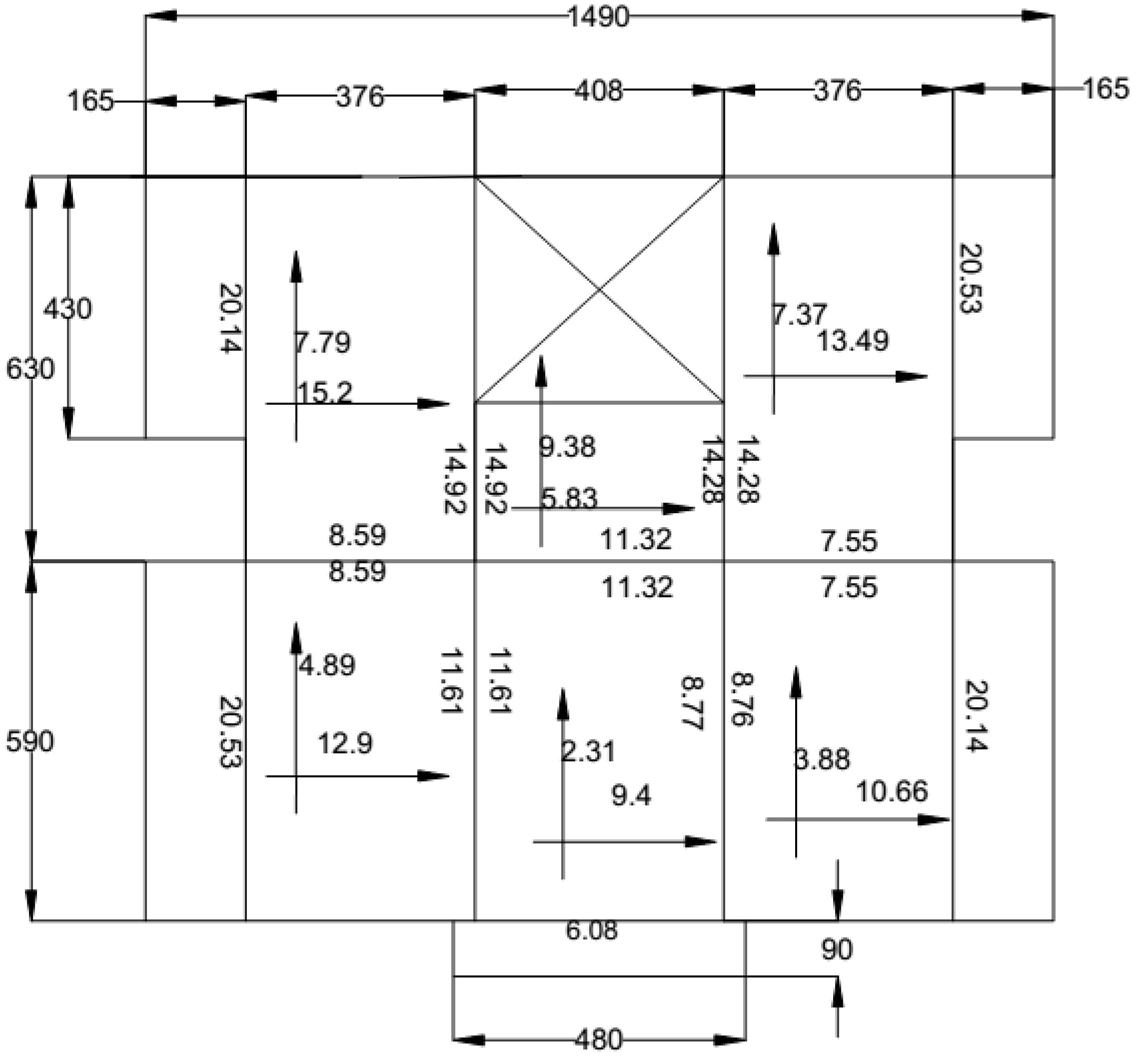


Figure 2-4 Adjusted span and support moment on the slab

2.3 Flexural Reinforcement Design of The Slab

The total depth of the slab for the two-way panels and for the cantilever part of the slab is 170mm and 210mm respectively. From this the effective depth of the slab, which is the depth measured from the compression face of the slab to the center of the reinforcement on the tension side of the slab can be calculated as follows:

$$d_x = D - \frac{\phi}{2} - cover$$

$$d_y = D - \phi - \frac{\phi}{2} - cover$$

Where

D is total depth of the slab,

d_x is the effective depth of main reinforcement,

d_y is the effective depth of transverse reinforcement, and

Φ is bar diameter.

The nominal concrete cover is 25mm, a reinforcement bar having 8mm is used.

The effective depth for the two-way panels will be:

$d_x = 170mm - \frac{8mm}{2} - 25mm$=141mm

$d_y = 170 - 8mm - \frac{8mm}{2} - 25mm = 133$mm

The effective depth for the cantilever parts of the slabs will be:

$d_x = 210mm - \frac{8mm}{2} - 25mm$ =181mm

$d_y = 210mm - 8mm - \frac{8mm}{2} - 25mm = 173$mm

2.3.1 Main reinforcement flexural design

M_{sd} =15.2KNm

dx =141mm

$\mu sd = Msd/(fcd * b * d^2)$.

Where

M_{sd} Is the design bending moment,

f_{cd} Is the design compressive strength of concrete, b Is width (for slab 1m width is taken), and

D Is effective depth of the slab

$\mu_{sd} = (15.2 * 10\text{^}6\text{Nmm})/(14.167\text{N/mm2} * 1000\text{mm} * (141\text{mm})^2)$.

μ_{sd}=0.054

K_z=0.969 …………...ES EN 1992-1-1:2015, Table 2.2

$$\text{Ast1} = \frac{\text{Msd}}{\text{fyd}*\text{Z}}.$$

Where

f_{yd} Is the design tensile strength of reinforcement bar,

Z Is the moment arm which is the product of Kz and d, and

A_s Is the area of reinforcement bar on the tension side

$A_s = 15.2 * 10^6 * Nmm/(347.83\frac{N}{mm^2} * 0.969 * 141mm)$= 319.84mm^2

The area of the reinforcement should not be taken less than the minimum area of reinforcement given by ES EN 1992-1-1:2015, Expression 9.1N in order to control shrinkage, and cracking. And should not be taken greater than the maximum area of reinforcement provided by the code because if it is greater than the maximum value the ductility will be altered, and there will be congestion of reinforcement bar.

$$A_{s,min} = max\begin{cases}\dfrac{0.26 * fctm * bt * d}{fyk} \\ 0.0013bt * d\end{cases}$$

Where

b_t Is the mean width of the tension zone, and

f_{ctm} Is mean value of axial tensile strength of concrete

The value of f_{ctm} according to EN-1992-1-1, Table 3.1 for characteristic tensile strength of C-25/30 is 2.6Mpa.

$$A_{s,min} = max\begin{cases}\dfrac{0.26 * 2.6\frac{N}{mm^2} * 1000mm * 141mm}{400\frac{N}{mm^2}} = 238.29mm^2 \\ 0.0013 * 1000mm * 141mm = 183.3mm^2\end{cases}$$

$As, min \ = 238.29mm2 \ < \ Ast1 = 319.84mm2$

The maximum area of reinforcement should be determined according to ES EN 1992-1-1:2015 section 9.2.1.1(3).

$A_{s,max} = 0.04A_C$, where A_c is the area of concrete

$A_{s,max} = 0.04 * 170mm * 1000mm$

$A_{s,max} = 6800mm^2$ ……………...for the two-way panels

$A_{s,max} = 0.04 * 210mm * 1000mm$

$A_{s,max} = 8400mm^2$ ………………for the cantilever part of the slab

The minimum area of reinforcement is less than the actual area of reinforcement and the maximum area of reinforcement bar is greater than the actual area of reinforcement. Therefore, take

$\text{Ast, provided} = 319.84\text{mm2}$

$$S = \frac{a_s * b}{ASt, provided}$$

Where

S Is the spacing of the reinforcement bars,

a_s Is the area of single reinforcement bar, and

A_{st}, Is the area of the provided reinforcement bar.

$S = ((\pi * \frac{(8mm)^2}{4}) * 1000mm)/319.84mm^2$

S= 157.16mm, take $S_{,provided}$=150mm

The spacing of the reinforcement bar should not exceed the maximum spacing (S_{max}) given ES EN 1992-1-1:2015 section 9.3.1(5).

$$\text{Smax} = \min \begin{cases} 3\text{h} \\ 40\text{mm} \end{cases}$$

h is the total depth of the slab

$\text{Smax} = \min \begin{cases} 3 * 170\text{mm} = 510\text{mm} \\ 400\text{mm} \end{cases}$

$\text{Smax} = 400\text{mm}$

$\text{S, provided} \ = 150\text{mm} < \text{S, max} = 400\text{mm}$

Since the provided spacing of the reinforcement bars is less than the maximum value of the spacing take the spacing of the reinforcement bars to be 150mm.

Following the same procedure, the area of the reinforcement and the spacing of the reinforcement bars can be computed as shown in Appendix A.

2.4 Design for Shear

According to ES EN 1992-1-1:2015 section 9.3.2(1) a slab in which shear reinforcement is provided should have a depth of at least 200mm. since the depth of the two-way panels in our slab system is less than 200mm which is 170mm. Therefore, no provision of shear reinforcement is required.

For the cantilever parts of the slab the slab with a depth of 210mm the provision of shear reinforcement should may be required.

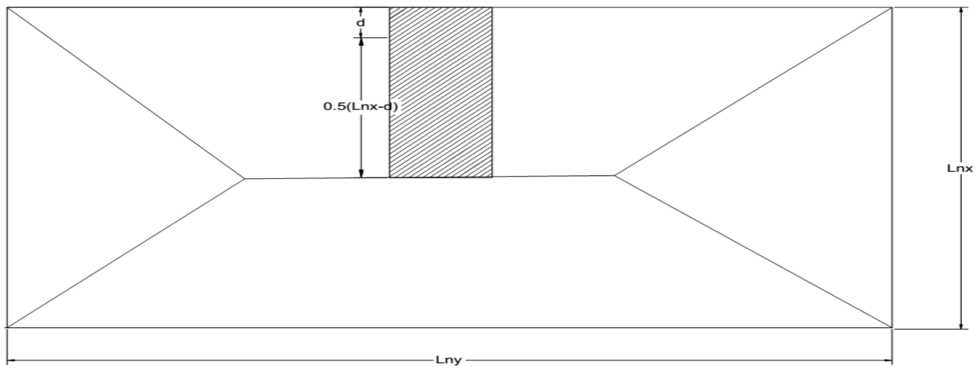


Figure 2-5 strips for shear design

$$V_{Ed} = w(0.5lnx - d) * bw$$

Where

V_{Ed}	Is the shear force,
b_w	Is the smallest width of the cross-section in the tensile area(m),
l_{nx}	Is the smaller clear span length,
D	Is effective depth, and
W	Is the areal distributed design load over the slab.

The design shear resistance of concrete without shear reinforcement $\mathrm{VRd,c}$ should be determined using ES EN 1992-1-1:2015 expression (6.2a) or ES EN 1992-1-1:2015 expression (6.2b) respectively.

$V_{Rd,c} = (C_{Rd,c} * k(100 * \rho * fck)^{\frac{1}{3}} + (k1\sigma cp))b_w d$ or

$V_{Rd,c} = \big(V_{min} + (k1\sigma cp)\big)b_w d$

$$V_{Rd,c} = \max\begin{cases}(C_{Rd,c} * k(100 * \rho_l * fck)^{\frac{1}{3}} + (k1\sigma cp))b_w d \\ \big(V_{min} + (k1\sigma cp)\big)b_w d\end{cases}$$

$V_{min} = 0.035 * k^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}}$ ………ES EN-1992-1-1:2015 (1) Note

Where

ρ_l Is the flexural reinforcement ratio,

$\rho_l = Asl/bwd \leq 0.02$ ....…….. ………………….. ES EN 1992-1-1:2015 (1)

σ_{cp} Is ratio of axial load to area of the concrete,

$\sigma_{cp} = \min\begin{cases}NEd/Ac \\ 0.2fcd\end{cases}$ ………………………….. ES EN 1992-1-1:2015 (1)

N_{Ed} is the axial force in the cross-section due to loading or pre-stressing in newton's (NEd>0 for compression).

A_c Is the area of concrete cross section 1992 (mm2),

A_{sl} Is the area of the tensile reinforcement, and

b_w Is the smallest width of the cross-section in the tensile area (mm).

$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ d in mm……………………………… ES EN 1992-1-1:2015 (1)

$C_{Rd,c} = 0.18/\gamma c$ ………………………………………… ES EN -1-1:2015 (1)Note

Taking the area of reinforcement ϕ8c/c 150 and effective depth of 181mm the distributed design load over the strip is 6.51KN/m^2.the clear distance of 1.5m and having reinforcement area of 334.11mm^2.

$V_{Ed} = 6.51\frac{KN}{m^2}(0.5 * 1.5m - 0.181m) * 1m$

$VEd = 3.7KN$

$V_{Rd,c} = \max\begin{cases}(C_{Rd,c} * k(100 * \rho_l * fck)^{\frac{1}{3}})b_w d \\ (V_{min})b_w d\end{cases}$, the component $k_1\sigma_{cp}$ is zero because we don't have any axial load supported on the slab.

$C_{Rd,c} = \frac{0.18}{1.5}$ =0.12

$k = 1 + \sqrt{\frac{200}{181}}$ =2.05…. Take k=2

$\rho_l = \frac{334.11mm^2}{1000mm*181mm} = 1.846x10-3$

$$V_{Rd,c} = \max\begin{cases}(0.12)*2(100*1.846*10^{-3}*25)^{\frac{1}{3}})1000mm*181mm \\ \left(0.035*2^{\frac{3}{2}}*25^{\frac{1}{2}}\right)1000mm*181mm\end{cases}$$

$$V_{Rd,c} = \max\begin{cases}72.32KN \\ 89.6KN\end{cases}$$

Take $V_{Rd,c} = 89.6KN$

Since the value of acting shear which is 3.7KN is less than the design shear resistance of the sab without shear reinforcement which is 89.6KN no provision of shear reinforcement is required

2.5 Load Transfer from Slab to Beam

The supporting beams will carry loads transferred from the slabs of triangular or trapezoidal patterns. To simplify structural analysis of beams by transforming triangular and trapezoidal slab loads to equivalent uniform distributed load. The loads that we have transferred to the supporting beams are un-factored and the dead load and live loads are transferred separately.

Figure 2-6 Trapezoidal load distribution and its equivalent rectangular load distribution

Sample calculation

$$v = vsx, \text{when } l = ly \text{ and } v = vsy \text{ when } l = lx$$

For panel 1

$V_{x,c} = \beta_{vx,c} * w * l_x$

$V_{x,c} = 0.56 * 8.51\frac{KN}{m^2} * 3.76m$ =17.92 KN/m

$V_{x,d} = \beta_{vx,d} * w * l_x$

$V_{x,d} = 0.37 * 8.51 \frac{KN}{m^2} * 3.76m$ =11.84 KN/m

$V_{y,c} = \beta_{vy,c} * w * l_x$

$V_{y,c} = 0.4 * 8.51 \frac{KN}{m^2} * 3.76m$ =12.8 KN/m

$V_{y,d} = \beta_{vy,d} * w * l_x$

$V_{y,d} = 0.26 * 8.51 \frac{KN}{m^2} * 3.76m = 8.32 \text{ KN/m}$

Table 2-13 Un-factored dead load transferred from two-way panels of the slab to beam

1st to 10th Floor				Shear force coefficient					Shear force			
Panel	LY	LX	LY/LX	βvx,c	βvx,d	βvy,c	βvy,d	Dead load (KN/m2)	Vx,c	Vx,d	Vy,c	Vy,d
1	6.3	3.76	1.68	0.56	0.37	0.4	0.26	8.51	17.92	11.84	12.80	8.32
2	4.08	2.6	1.57	0.52	0.34	0.36	0	7.47	10.10	6.60	6.99	0.00
3	6.3	3.76	1.68	0.56	0.37	0.4	0.26	7.67	16.15	10.67	11.54	7.50
4	5.9	3.76	1.57	0.55	0.36	0.4	0.26	6.72	13.90	9.10	10.11	6.57
5	5.9	4.08	1.45	0.46	0	0.36	0.24	7.11	13.34	0.00	10.44	6.96
6	5.9	3.76	1.57	0.55	0.36	0.4	0.26	5.37	11.09	7.26	8.07	5.24

Table 2-14 Un-factored live load transferred from two-way panels of the slab to beam

1st to 10th floor				Shear force coefficient					Shear force			
Panel	LY	LX	LY/LX	βvx,c	βvx,d	βvy,c	βvy,d	Live load (KN/m2)	Vx,c	Vx,d	Vy,c	Vy,d
1	6.3	3.76	1.68	0.56	0.37	0.4	0.26	2.00	4.21	2.78	3.01	1.96
2	4.08	2.6	1.54	0.52	0.34	0.36	0	2.00	4.24	2.77	2.94	0.00
3	6.3	3.76	1.68	0.56	0.37	0.4	0.26	2.00	4.21	2.78	3.01	1.96
4	5.9	3.76	1.57	0.55	0.36	0.4	0.26	2.00	4.14	2.71	3.01	1.96
5	5.9	4.08	1.45	0.46	0	0.36	0.24	2.00	3.75	0.00	2.94	1.96
6	5.9	3.8	1.57	0.55	0.36	0.4	0.26	2.00	4.14	2.71	3.01	1.96

2.6 Load Transfer from The Cantilever Parts of The Slab to Beam

Sample calculation

Load transferred from “C1” and “C2”

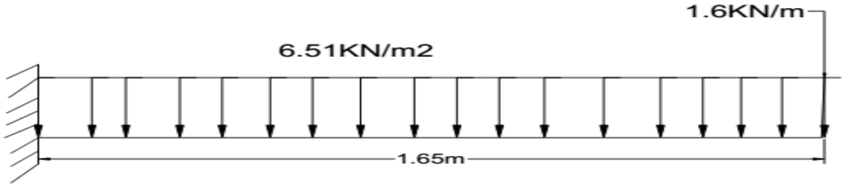


Figure 2-7 Un-factored dead load on C1 and C2

$$\text{unfactored load from slab(p)} = \text{distributed load} * \text{span length} + \text{concentrated load}$$

$$P_d = 6.51\frac{KN}{m^2} * 1.65m + 1.7\ KN/m$$

$$P_d = 12.44KN/m$$

Following the same procedure, we have determined the loads transferred from the cantilever parts to the supporting beams. And they are tabulated as shown below.

Table 2-15 Un-factored dead and live loads from the cantilever part of the slab to the supporting beams

Panels	Dead load(KN/m)	Live load(KN/m)
C1	12.44	3.3
C2	12.2	3.3
C3	12.44	3.3
C4	12.2	3.3
C5	7.43	1.8

3 Analysis and Design of staircase

3.1 Introduction

Stair is set of steps leading from one floor to another and provided in building to afford a means of communication between the various floors. And steps arranged in series and placed in an enclosed is called staircase.

Primary function of stair

- To provide a means of circulation between floor levels.
- Establish a safe and easy means of travel between floor levels.
- Provide a means of conveying fitting and furniture between floor levels.

Technical terminologies

Step: is a portion of stair which permits ascent and descent. And it is comprised of a tread and a riser.

Tread: is the horizontal member of stair.

Going: is the horizontal distance between the nosing or front edges of two consecutive steps.

Riser: is the vertical member of stair.

Rise: is the vertical distance between the upper surfaces of two consecutive steps.

Nosing: it is the projecting part of beyond the face of the riser.

Flight: is a continuous set of steps between floors and /or landing.

Landing: is a platform between two flights.

Baluster: is the vertical member which supports the hand rail.

Hand rail: is a rounded or molded member of wood or metal fixed on the top of baluster.

Head room: is the minimum clear vertical distance between the tread and overhead structure.

Soffit: it is the underside of the stair.

Run: it is the total length of the stair in a horizontal plane, including landing.

Newel post: is the vertical member which is placed at the end of flight to connect the end sting and hand rail.

Winder: are tapering step which are provided for changing the direction of stair.

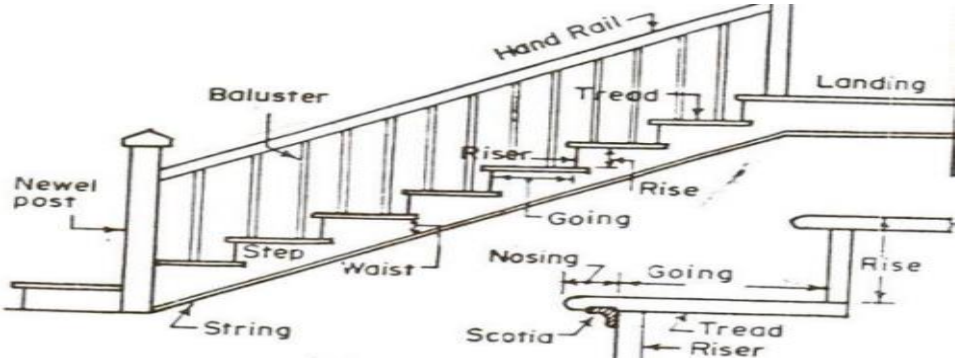


Figure 3-1 Staircase and its component

3.2 Classification of staircase

Stair case are classified based on

- Geometric configuration
- Structural classification

3.2.1 Based on Geometric configuration

I. **Straight stairs**: these stairs run straight between the two floors. And it is used for small houses where there are restrictions in available width.

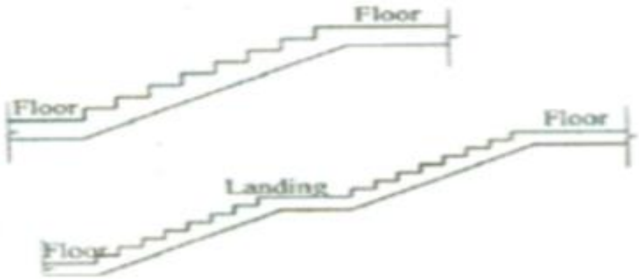


Figure 3-2 Straight stairs

II. **Turning stairs:** includes quarter turn stair, half turn stair, three quarter turn stairs and bifurcating stairs.

  a) Quarter turn stair: is the one which changes its direction either to the left or to the right. And the turn being affected by introducing a quarter space landing or by providing winders.

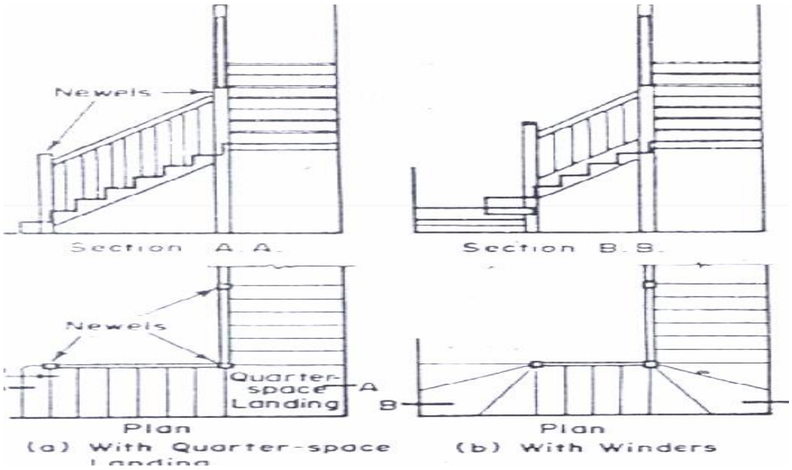


Figure 3-3 Quarter turn stair

B) Half turn stair: is the one which has its direction reversed or changed for180^0.

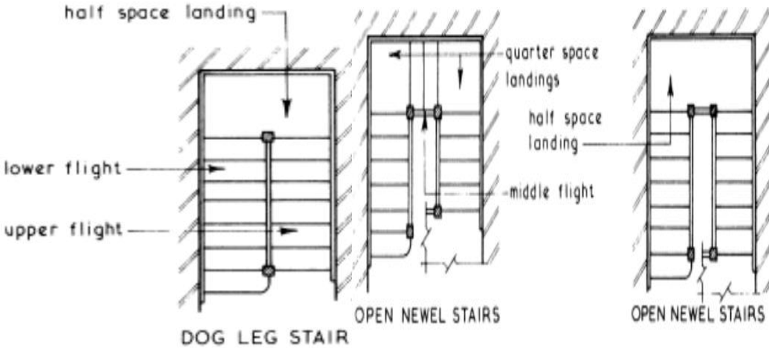


Figure 3-4 Half turn stair

C) Three quarter Turn stairs: Have its direction changed three times with its upper flight crossing the bottom one .and such type of stair is used when the length of the stair room is limited and when the vertical distance between the two floors is quite large.

III. Bifurcated stairs: the stair has a wider at the bottom, which bifurcates into two narrow flight one turning to the left and the other to the right.

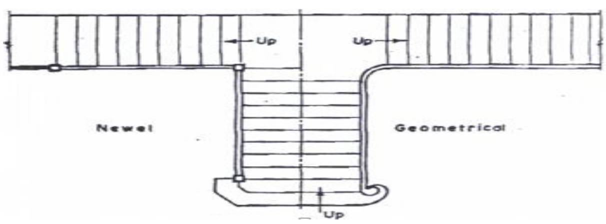


Figure 3-5 Bifurcated stairs

3.2.2 Based on structure

I. **Stair slab spanning longitudinally**: Here, one or more supports are provided parallel to the rise for slab bending longitudinally show different support arrangements of a two flight.

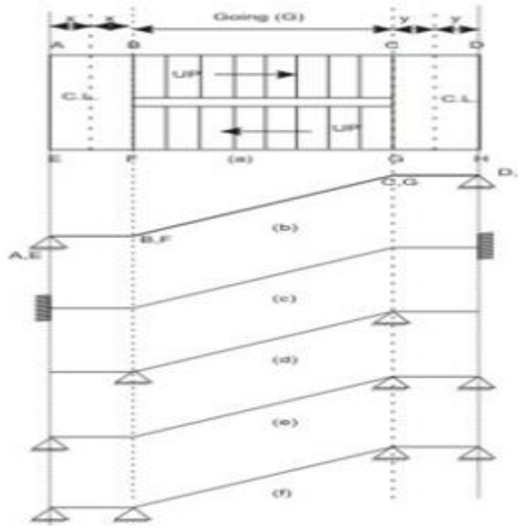


Figure 3-6Stair slab spanning longitudinally

II. **Stair slab spanning transversally:** Here, either the waist slabs or the slab components of isolated tread-slab and trade-riser units are supported on their sides or are cantilevers along the width direction from a central beam. The slabs thus bend in a transverse vertical plane. The following are the different arrangements:

a) Slab supported between two stringer beams or walls.

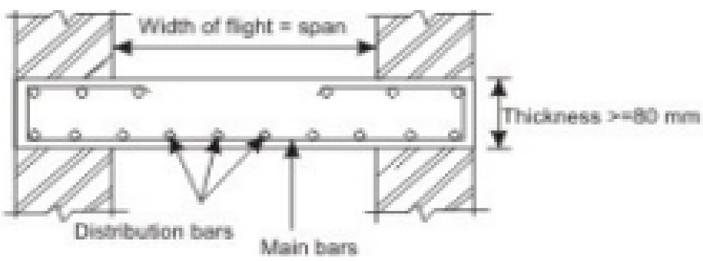


Figure 3-7 Slab supported between two stringer beams or walls

Cantilever slabs from a spandrel beam or wall.

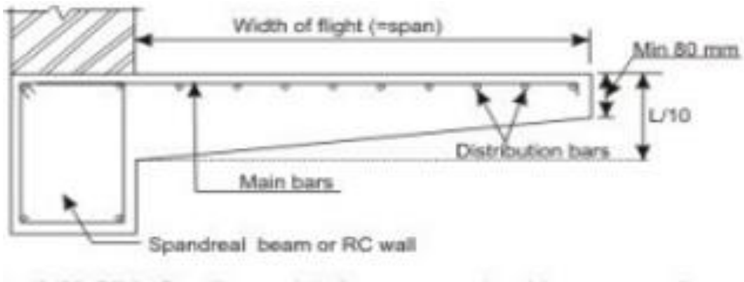


Figure 3-8 Cantilever slabs from a spandrel beam or wall

Doubly cantilever slabs from a central beam

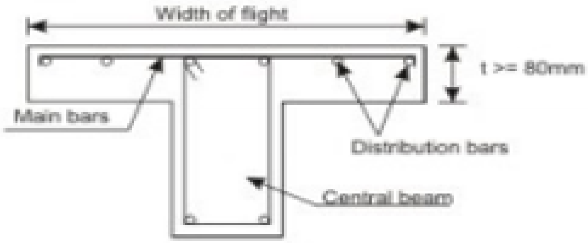


Figure 3-9 doubly cantilever slabs from a central beam

3.3 Detailing of landing going junction

At the junction there are two cases as shown in figure 3-10

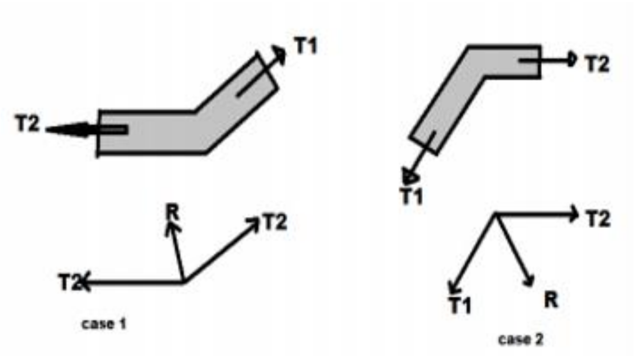


Figure 3-10 Resultant tensional force at the junction

Case 1: there is no problem due to resultant force due to tension

Case 2: there is problem due to resultant force due to tension; correct detailing procedures must be followed to avoid the failure of concrete due to resultant force.

3.4 Analysis and Design of Staircase

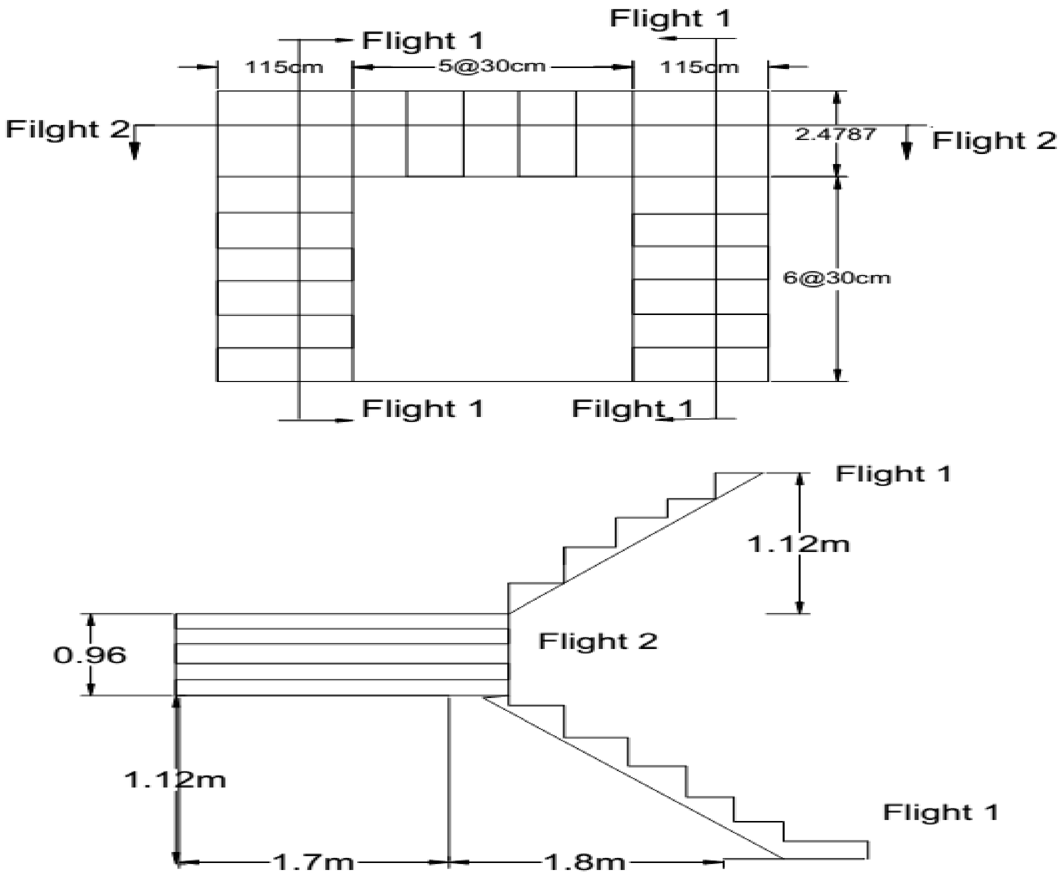


Figure 3-11 Top and sectional view of staircase

Number of riser(N) = (tread + 1) =(6 + 1)=7

Height of riser = H/N =$1.12m/7$= 16cm

3.5 Depth of deflection

Our stair is treated as simply supported solid slab than according to ES EN 1992-1-1:2015 section 7.4 Table 7.4N k =1 for simply supported beam or slab.

$$l/d = 1*\left[11+1.5\sqrt{25}*0.005/0.005\right]*500/400$$

$$l/d = 23.125$$

For flight 1 ($l = 2.97m$,)

$$d = l/23.125 = 2.97m/23.125 = 128.43mm$$

For flight 2($l = 1.5m$)

$$d = l / 23.125 = 1.5m / 23.125 = 64.86mm$$

Taking the maximum deflection depth

$$\mathrm{d} = \max \begin{cases} 128.43\mathrm{mm} \\ 64.86\mathrm{mm} \end{cases}$$

$\mathrm{d} = 128.43\mathrm{mm}$ Then

$\mathrm{D} = \mathrm{d} + {}^{1}/_{2}\, Ø + \mathrm{cover}$

Where $\mathrm{d} = 128.43\mathrm{mm}$, $Ø = 10\mathrm{m}$ $\quad \mathrm{cover} = 25\mathrm{mm}$

$$\mathrm{D} = 128.43\mathrm{mm} + {}^{1}/_{2}\, 10\mathrm{mm} + 25\mathrm{mm} = 158.43\mathrm{mm} \cong 160\mathrm{mm}$$

3.6 Stairs loading

Table 3-1 Material used with their unit weight and thickness

Material type	Unit weight(KN/m^2)	Thickness (cm)
Cement screed	23	3
Concrete	25	16
Plastering	17	2
Marble	27	2

Calculation of dead load using 1m width Flight 2

Step dead load

$$\text{Dead load of cement screed} = t_{cs} * \gamma_{cs} = 0.03m * 23\frac{kN}{m2} = 0.69kN/m$$

$$\text{Dead load of marble} = t_{mar} * \gamma_{mar} = 0.02m * 27\frac{kN}{m^2} = 0.54kN/m$$

$$\text{Dead load of concrete} = t_{con} * \gamma_{con} = 0.16m * 0.5 * 25\frac{KN}{m^2} = 2kN/m$$

$$\text{Dead load of step} = \sum(Dl_{cs}, Dl_{mar}, Dl_{con})$$

$$= 0.69kN/m + 0.54KN/m + 2kN/m$$

$$\text{Dead load of step} = 3.23\mathrm{kN/m}$$

Riser dead load

Dead load of cement screed $= \frac{N*(t_{cs}*h_{cs}*\gamma_{cs})}{\text{projected length}} = \frac{6*(0.16*0.03*23)}{1.5} = 0.4416\text{ kN/m}$

Dead load of marble $= \frac{N*(t_{mar}*h_{mar}*\gamma_{mar})}{\text{projected length}} = \frac{6*(0.16*0.02*27)}{1.5} = 0.3456\text{ kN/m}$

Riser dead load = $\sum(Dl_{cs}, Dl_{mar})$

Riser dead load = $0.4416\text{kN/m} + 0.3456\text{kN/m}$

Riser dead load=0.7872kN/m

Waist dead load

$\tan \theta = \frac{96\text{cm}}{150\text{cm}}, \theta = \tan^{-1}\left(\frac{96}{150}\right) = 32.62^0$

$l_{inc} = \frac{96}{\sin 32.62^0} = 1.78\text{m}$

Dead load of concrete $= \frac{t_{con}*\gamma_{con}*l_{inc}}{\text{projected length}} = \frac{0.16*1.78*25}{1.5} = 4.747\text{kN/m}$

Dead load plastering $=\frac{t_{pl}*\gamma_{pl}*l_{inc}}{\text{projected length}} = \frac{0.02*1.78*17}{1.5} = 0.356\text{kN/m}$

Waist dead load =$\sum(Dl_{con}, Dl_{pl})$

Waist dead load= $4.747\frac{KN}{m} + 0.356\frac{KN}{m}$

Waist dead load $=$ 5.103 kN/m

Total dead load on the stairs slab

$DLI_T = DL_{step} + DL_{riser} + DL_{waist}$

$DLI_T = 3.23\text{kN/m} + 0.7872\text{kN/m} + 5.103\text{kN/m}$

Total dead load on the stairs slab = $DLI_T = 9.1202\text{ kN/m}$

Imposed load on flight 2

According to ES EN 1991-1-1:2015 table 6.2 for category A live load for stair is 4 kN/m^2.

$L.L = 4\frac{kN}{m^2} * 1m = 4\text{kN/m}$

Design load and moment for flight 2

$$\text{Design load}, P_d = 1.35 * DLI_T + 1.5 * L.L$$

$$P_d = 1.35 * 9.12 + 1.5 * 4 = 18.312\ kN/m$$

$$\text{Design moment}, M_{max} = M_{design} = M_D = 18.312 * \frac{1.5}{2} * \left(\frac{1.5}{4} + \frac{1.15}{2}\right) kNm$$

$$M_D = 13.0473kNm$$

Load transfer from flight 2 to flight 1

- From live load, $W = \frac{R_A}{1.15} = \frac{\left(\frac{4*1.5}{2}\right)}{1.15m} = 2.609kN/m$
- From dead load, $W = \frac{R_A}{1.15} = \frac{\left(\frac{9.12*1.5}{2}\right)}{1.15m} = 5.948kN/m$

Calculation of dead load using 1m width Flight 1

Step dead load

$$\text{Dead load of cement screed} = t_{cs} * \gamma_{cs} = 0.03m * 23\frac{kN}{m^2} = 0.69kN/m$$

$$\text{Dead load of marble} = t_{mar} * \gamma_{mar} = 0.02m * 27\frac{kN}{m^2} = 0.54kN/m$$

$$\text{Dead load of concrete} = t_{con} * \gamma_{con} = 0.16m * 0.5 * 25\frac{kN}{m^2} = 2kN/m$$

$$\text{Dead load of step} = \sum(Dl_{cs}, Dl_{mar}, Dl_{con})$$

$$= 0.69kN/m + 0.54kN/m + 2kN/m$$

$$\text{Dead load of step} = 3.23kN/m$$

Riser dead load

$$\text{Dead load of cement screed} = \frac{N*(t_{cs}* h_{cs}* \gamma_{cs})}{\text{projected length}} = \frac{7*(0.16*0.03*23)}{1.8} = 0.429kN/m$$

$$\text{Dead load of marble} = \frac{N*(t_{mar}* h_{mar}* \gamma_{mar})}{\text{projected length}} = \frac{7*(0.16*0.02*27)}{1.8} = 0.336kN/m$$

$$\text{Riser dead load} = \sum(Dl_{cs}, Dl_{mar})$$

$$= 0.429kN/m + 0.336kN/m$$

$$\text{Riser dead load} = 0.765kN/m$$

Waist dead load

$\tan \theta = \frac{1.12\text{cm}}{180\text{cm}}, \theta = \tan^{-1}\left(\frac{1.12}{180}\right) = 31.89^0$

$l_{inc} = \frac{1.12}{\sin 31.89^0} = 2.12\text{m}$

$\text{Dead load of concrete} = \frac{t_{con} * \gamma_{con} * l_{inc}}{\text{projected length}} = \frac{0.16*2.12*25}{1.8} = 4.711\text{kN/m}$

$\text{Dead load plastering} = \frac{t_{pl} * \gamma_{pl} * l_{inc}}{\text{projected length}} = \frac{0.02*2.12*17}{1.8} = 0.4\text{kN/m}$

$\text{Waist dead load} = \sum(Dl_{con}, Dl_{pl}) = 4.711\frac{\text{kN}}{\text{m}} + 0.4\frac{\text{kN}}{\text{m}}$

$\text{Waist dead load} = 5.11\ \text{kN/m}$

Total dead load on stairs slab

$DL_T = DL_{step} + DL_{riser} + DL_{waist}$

$DL_T = 3.23\text{kN/m} + 0.7651\text{kN/m} + 5.11\text{kN/m}$

Total dead load on the inclined slab = $DLI_T = 9.105\ \text{kN/m}$

Landing dead load

$\text{Total dead load of landing} = DLL_T = t_{cs} * \gamma_{cs} * t_{mar} * \gamma_{mar} * t_{con} * \gamma_{con} * t_{pl} * \gamma_{pl} + \text{W dead load from flight 2}$

$= 27 * 0.02 + 0.03 * 23 + 0.16 * 25 + 17 * 0.02 + 7.93 = 11.52\text{kN/m}$

$\text{Total dead load of landing} = DLL_T = 11.52\text{KN/m}$

Imposed load

$\text{Total imposed on the stairs slab} = \text{L.L} = 4\frac{\text{kN}}{\text{m}^2} * 1\text{m} = 4\text{kN/m}$

$\text{Total imposed on landing} = LL_L = \text{L.L} + \text{W live load from flight 2}$

$= 4\ \text{kN/m} + 2.609\ \text{kN/m}$

$\text{Total imposed load on landing} = LL_L = 6.609\ \text{kN/m}$

Design load and moment for the flight

Design load for the stairs slab

Design load, $P_d = 1.35 * DLI_T + 1.5 * L.L$

$P_d = 1.35 * 9.105 + 1.5 * 4$

$P_d = 18.292 kN/m$

Design load for landing

Design load, $P_d = 1.35 * DLL_T + 1.5 * LL_L$

$P_d = 1.35 * 11.52 + 1.5 * 6.609$

$P_d = 25.47\ kN/m$

Design moment for flight 1

Design moment, $M_{max} = M_{design} = M_D = 22.696 kNm$

$$M_D = 22.696 kNm$$

3.7 Design of Staircase for Flexure

Material used and geometry

fyd = 347.83Mpa

fcd =14.167Mpa

$d = 128.43mm$

$b_t = 1000mm$

Using reinforcement Ø 10 then $a_s = \frac{3.14 * \emptyset^2}{4} = 78.5 mm^2$

3.7.1 Flight 2, Design for main reinforcement bar (principal reinforcement)

$$M_D = M_{sd} = 13.0473 kNm$$

From general design chart and design table to ES EN 1992-1-1:2014 Table 2.2 design table for C-12/15 – C-50/60

$$\mu_{sd} = \frac{M_{sd}}{fcd * b_t * d^2} = \frac{13.0473 * 10^6 Nmm}{14.167 * 1000 * 128.43^2 Nmm} = 0.5586$$

Using $\mu_{sd} = 0.5586$,$k_z = 0.969$ …. from design table 2.2 for $C12/25 - C50/60$

$$Z = k_z * d = 0.969 * 128.43mm = 124.40mm$$

$$A_s = \frac{M_{sd}}{fyd * Z} = \frac{13.0473 * 10^6 Nmm}{387.83 \frac{N}{mm^2} * 124.4mm} = 270.28\mathrm{mm}^2$$

$$A_s = 270.28mm^2$$

3.7.2 Check for minimum and maximum reinforcement area

Minimum requirement

$$\mathrm{A_{s,min}} = 0.26 \frac{2.6}{387.83 \frac{\mathrm{N}}{\mathrm{mm}^2}} * 1000\mathrm{mm} * 128.4\mathrm{mm}$$

$$\geq 0.0013 * 1000\mathrm{mm} * 128.4\mathrm{mm}$$

$$\mathrm{A_{s,min}} = 217.0467\mathrm{mm}^2 \geq 166.959\mathrm{mm}^2$$

$$\mathrm{A_{s,min}} = 217.0467\mathrm{mm}^2$$

$$\mathrm{A_{s,min}} = 217.0467\mathrm{mm}^2 < \mathrm{A_s} = 270.28\mathrm{mm}^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\mathrm{ok}$$

Because According to EN EN 1992-1-1:2015 section 9.2 Sections containing less reinforcement than $\mathrm{A_{s,min}}$ should be considered as unreinforced.

Maximum requirement

$$\mathrm{A_{s,max}} = 0.004 * 1000\mathrm{mm} * 160\mathrm{mm}$$

$$\mathrm{A_{s,max}} = 6400\mathrm{mm}^2$$

$$\mathrm{A_{s,max}} = 6400\mathrm{mm}^2 > \mathrm{A_s} = 270.28\mathrm{mm}^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\mathrm{ok!}$$

Because According to ES EN 1992-1-1:2015 section 9.2 the cross-sectional area of tension or compression reinforcement should not exceed $\mathrm{A_{s,max}}$ outside lap locations.

Therefore, provide $\mathrm{A_s} = 270.28\mathrm{mm}^2$

$$\mathrm{A_{s,provided}} = 270.28\mathrm{mm}^2$$

3.7.3 Flight 2, Secondary transvers reinforcement

According to ES EN 1992-1-1:2015 section 9.3.1.1 Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs. In areas near supports transverse reinforcement to principal top bars is not necessary where there is no transverse bending moment. Staircase is treated as one-way slab. Therefore, secondary transverse reinforcement for staircase is

$$A_{st} = 20\% A_{s,provided} \ .$$

Where

A_{st} Is area of Secondary transverse reinforcement, mm^2, and

$A_{s,provided}$ Is area of principal reinforcement provided, mm^2

$A_{st} = 0.2 * 270.28mm^2$

$A_{st} = 54.056mm^2$

3.7.4 Check for minimum and maximum reinforcement area

Minimum requirement

$A_{st} = 54.056mm^2 > A_{s,min} = 217.0467mm^2 \dots \dots \dots$ not ok. Therefore, use area minimum as secondary reinforcement.

$A_{st,provided} = 217.0467mm^2$

Calculation for spacing

Spacing for principal reinforcement

$S = \frac{b_t * a_s}{A_{s,provided}} = \frac{1000mm * 50.26mm^2}{270.28mm^2} = 185.98mm$

$S = 180mm$

Spacing for secondary reinforcement

$S = \frac{b_t * a_s}{A_{st,provided}} = \frac{1000mm * 50.26mm^2}{217.0467mm^2} = 231.56mm$

$S = 230mm$

Check for maximum spacing

According to ES EN 1992-1-1:2015 section 9.3.1.1 the spacing of bars should not exceed $S_{max,slabs}$.

For principal reinforcement

$$S_{max,slabs} = \min\begin{cases} 3h \\ 400 \end{cases} mm$$

$$S_{max,slabs} = \min\begin{cases} 3*160 \\ 400 \end{cases} mm = \min\begin{cases} 480 \\ 400 \end{cases} mm$$

$$S_{max,slabs} = 400mm > S = 180mm \ldots\ldots\ldots.ok$$

$$S_{provided} = 180mm$$

For secondary reinforcement

$$S_{max,slabs} = \min\begin{cases} 3.5h \\ 450 \end{cases} mm$$

$$S_{max,slabs} = \min\begin{cases} 3.5*160 \\ 450 \end{cases} mm = \min\begin{cases} 560 \\ 450 \end{cases} mm$$

$$S_{max,slabs} = 450mm > S = 230mm \ldots\ldots\ldots.ok$$

$$S_{provided} = 230mm$$

Therefore, provide Ø10 c/c 180mm for principal reinforcement bar and provide Ø10 c/c 230mm for secondary reinforcement bar.

3.7.5 Flight 1, Design for main reinforcement bar (principal reinforcement)

$$M_D = M_{sd} = 22.696KNm$$

$$\mu_{sd} = \frac{M_{sd}}{fcd * b_t * d^2} = \frac{22.696 * 10^6 Nmm}{14.167 * 1000 * 128.43^2 Nmm} = 0.097$$

Using $\mu_{sd} = 0.097$,$k_z = 0.9475$ ….from design table 2.2 for C12/25 − C50/60

$$Z = k_z * d = 0.9475 * 128.43mm = 121.687mm$$

$$A_s = \frac{M_{sd}}{fyd * Z} = \frac{13.0473 * 10^6 Nmm}{387.83 \frac{N}{mm^2} * 121.687mm} = 481.406mm^2$$

$$A_s = 481.406mm^2$$

Check for minimum and maximum reinforcement

Minimum requirement

$$A_{s,min} = 217.0467mm^2 < A_s = 481.406mm^2 \ldots\ldots\ldots.\text{ok}$$

Maximum requirement

$$A_{s,max} = 6400mm^2 > A_s = 481.406mm^2 \ldots\ldots\ldots.\text{ok}$$

Therefore, $A_{s,provided} = 481.406mm^2$

Design for secondary reinforcement bar

$$A_{st} = 0.2 * 481.406mm^2$$

$$A_{st} = 96.2812mm^2$$

Check for minimum reinforcement

Minimum requirement

$$A_{s,min} = 217.0467mm^2 > A_{st} = 96.2812mm^2 \ldots\ldots\ldots\text{not ok}$$

$$A_{st,provided} = 217.0467mm^2$$

Check for spacing of bars

Spacing for principal reinforcement

$$S = \frac{b_t * a_s}{A_{s,provided}} = \frac{1000mm * 78.5mm^2}{481.406mm^2} = 160mm$$

$$S = 160mm < S_{max,slabs} = 400mm$$

$$S_{provided} = 160mm$$

Spacing for secondary reinforcement

$$S = \frac{\mathbf{b_t * a_s}}{\mathbf{A_{st,provided}}} = \frac{1000mm * 50.26mm^2}{217.0467mm^2} = 230mm$$

$$S = 230mm < S_{max,slabs} = 450mm$$

$$S_{provided} = 230mm$$

3.8 Design of staircase for shear

According to ES EN 1992-1-1:2015 section 6.2.1 For member's subject to predominantly uniformly distributed loading the design shear force need not to be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition, it should be verified that the shear at the support does not exceed$V_{Rd,max}$.

3.8.1 Check if the $V_{Rd,max}$ greater than V_{Ed} at the support

According ES EN 1992-1-1:2015 section 6.2.3 equation 6.9 the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts is calculated as follows:

$$V_{Rd,max} = \frac{\alpha_{cw} * b_w * z * v_1 * fcd}{(\cot\theta + \sin\theta)}$$

Where

α_{cw}	is a coefficient taking account of the state of the stress in the compression chord
b_w	1m width of the slab to be analysis, mm
v_1	is a strength reduction factor for concrete cracked in shear
Θ	is the angle between the concrete compression strut and the beam axis perpendicular to the shear force
Z	is the inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration. In the shear analysis of reinforced concrete without axial force.mm

$$\alpha_{cw} = 1 \text{ for non} - \text{prestressed structures}$$

$$z = 0.9d = 0.9 * 128.43\text{mm} = 115.587\text{mm}$$

$$v_1 = 0.6\left(1 - \frac{fck}{250}\right) = 0.6\left(1 - \frac{25}{250}\right) = 0.54$$

$$b_w = 1000\text{mm}$$

$$\theta = 21.8^0$$

V_{Ed} at the support

For flight 2 $V_{Ed} = 13.734\text{KN}$

For flight 1 $V_{Ed} = 33.91KN$

$$V_{Rd,max} = \frac{1 * 1000 * 115.587mm * 0.54 * 14.167N/mm^2}{(2.5 + 0.4)}$$

$$V_{Rd,max} = 304.92KN$$

$$V_{Rd,max} = 304.92KN > V_{Ed} = 13.734KN \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots ok$$

$$V_{Rd,max} = 304.92KN > V_{Ed} = 33.91KN \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots.. ok$$

According to ES EN 1992-1-1:2015 in regions of the member where $\mathbf{V_{Rd,c}} \geq \mathbf{V_{Ed}}$ no calculated shear reinforcement is necessary

3.8.2 Check if $V_{Rd,c}$ is greater than V_{Ed} d distance from the face of the support

According to ES EN 1992-1-1:2015 section 6.2.2 equation (6.2.a and 6.2.b)

$$V_{Rd,c} = \max\begin{cases} \left[c_{Rd} * k * (100 * \rho_1 * f_{ck})^{\frac{1}{3}} * k_1 * \sigma_{cp}\right] * b_w * d \\ \left(v_{min} + k_1 * \sigma_{cp}\right) * b_w * d \end{cases}$$

$$c_{Rd} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$k = 1 + \sqrt{\frac{200}{128.43}} = 2.25 \leq 2.0$$

$$k = 2.0$$

$$\rho_1 = \frac{A_s}{b_w * d} \leq 0.02$$

For flight 2($A_s = 481.406mm^2$)

$$\rho_1 = \frac{481.406mm^2}{1000mm * 128.43mm} = 0.00374$$

$$\rho_1 = 0.00374$$

For flight 2($A_s = 270.28mm^2$)

$$\rho_1 = \frac{270.28mm^2}{1000mm * 128.43mm} = 0.0021$$

$$\rho_1 = 0.0021$$

$\sigma_{cp} = \frac{N_{ed}}{A_c} < 0.2f_{cd} = 0$……………………….because $N_{ed} = 0$

$$v_{min} = 0.035 * k^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}}$$

$$v_{min} = 0.035 * 2^{\frac{3}{2}} * 25^{0.5}$$

$$v_{min} = 0.495$$

V_{Ed} from d distance from the face of the support.

For flight 1, $V_{Ed} = 4.10946 + \left(29.79 * \frac{0.8916}{1.17}\right) = 26.81KN$

For flight 2, $V_{Ed} = 11.408KN$

For flight1, $V_{Rd,c}$

$$= \max\begin{cases}\left[0.12 * 2 * (100 * 0.00374 * 25)^{\frac{1}{3}} * 1 * 0\right] * 1000 * 128.43mm^2 \\ (0.495 + 1 * 0) * 1000mm * 128.43mm\end{cases}$$

$$V_{Rd,c} = \max\begin{cases}64.935KN \\ 63.57KN\end{cases}$$

$$V_{Rd,c} = 64.935KN$$

$> 26.81KN \ldots\ldots$ ok(no calculated shear reinforcement is necessary)

For flight 2, $V_{Rd,c}$

$$= \max\begin{cases}\left[0.12 * 2 * (100 * 0.0021 * 25)^{\frac{1}{3}} * 1 * 0\right] * 1000mm * 128.43mm \\ (0.495 + 1 * 0) * 1000mm * 128.43mm\end{cases}$$

$$V_{Rd,c} = \max\begin{cases}56KN \\ 63.57KN\end{cases}$$

$$V_{Rd,c} = 63.57KN$$

$> 11.408KN \ldots\ldots$ ok(no calculated shear reinforcement is necessary)

Therefore, no need of shear reinforcement and minimum reinforcement is required for our staircase because according to ES EN 1992-1-1:2015 section 6.2.1. If $\mathbf{V_{Rd,c} \geq V_{Ed}}$ no shear reinforcement is required and the minimum shear reinforcement may be omitted in members such as slabs (solid, ribbed, or hollow core slabs) where transverse redistribution of loads is possible.

3.9 Load transfer from staircase to beam

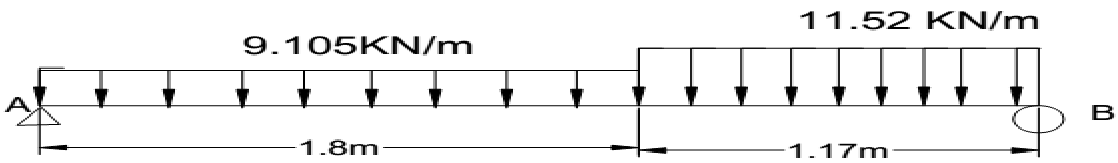


Figure 3-12 Load transferred from staircase to beam from due to dead load

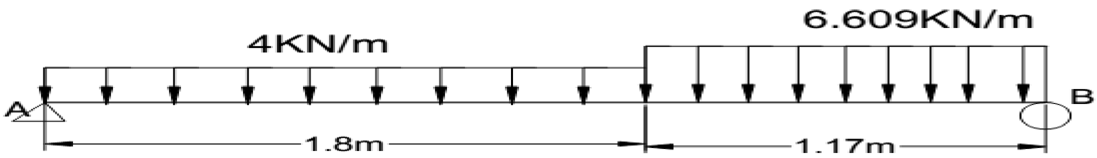


Figure 3-13 Load transferred from stair to beam due to live load

Table 3-2 Loads transferred from stair to beam

Load type	R_A(KN)	R_B(KN)
Dead load	6.54	8.39
Live load	14.08	15.8

4 WIND LOAD ANALYSIS AND ROOF DESIGN

4.1 Introduction

Wind is pressurized motion of air. They act directly on the external surface, also act indirectly on the internal surface. Structure gives response by resisting the wind with insignificant or small deflection. They may also directly affect the internal surface of open structures. Pressure acts on the area of the surface producing forces normal to the surface for the structure or for individual cladding component. Since the building is located in Addis Ababa the earthquake is the governing factor. Due to this reason we have not analyzed the building frame system for wind load. We have analyzed the wind load on the roof in order to determine the total load comes from the roof into the top tie beam.

4.2 Wind Load on Roof

Wind produces dynamic loads on a structure at highly variable magnitudes. The Variation in

Pressures at different locations on a building are complex to the point that pressures may become too analytically intensive for precise consideration in design. To simplify the complexity in analysis of wind load different codes provides specifications for wind load by considering basic static pressure zones on a buildings representative of peak loads that are likely to be experienced. Wind forces act directly or indirectly on the internal and external surface of structures. Wind loads fluctuate with time. A wind load produces static, dynamic and aerodynamic effects on structures. The wind load on the building is analyzed using the steps mentioned in EN-1991-1-4:2004.

Steps to analyze wind load on roof

i. Determination the basic wind velocity
ii. Determination the mean wind velocity
iii. Determination the wind turbulence
iv. Determination of peak velocity pressure
v. Determination of wind pressure on the roof

4.3 Analysis of wind load on the roof

4.3.1 Wind parameter

4.3.1.1 Basic wind velocity

The basic wind velocity $V_{b,}$ should be determined according to ES EN 1991-1-4:2015 expression 4.1.

$$V_b = C_{dir} * C_{season} * V_{b,o}$$

Where:

$V_{b,o}$	Is the fundamental value of the basic wind velocity;
C_{dir}	is the directional factor, its recommended value is 1. ES EN-1991-1-4:2015 section 4.2 NOTE2; and
C_{season}	is the season factor, its recommended value is 1 ES EN-1991-1-4:2015 section 4.2 NOTE3

$V_{b,o}$=Vb,map*Calt…………..British standard

$$C_{alt} = \begin{cases} 1 + 0.001A, & Z_S < 10M \\ 1+0.001A(\frac{10}{ZS})^{0.2} & Z_S \geq 10M \end{cases}$$

Where

Calt is Altitude factor

An average altitude is 2400m for Addis Ababa

Vb,map map wind velocity which is 15.1Km/hr=4.38m/sec for Addis Ababa from online sources(http://www.worldonlineweather.com)

Z_S=h*0.6

=42.84*0.6

= 25.704m

$$Calt=1 + 0.001 * 2400\left(\frac{10}{25.704}\right) 0.2$$

=2.98

V_b=2.98*4.38 m/sec

=13.0524 m/sec

4.3.1.2 Mean wind velocity

The mean wind velocity Vm(z) at a height Z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, Vb, and should be determined using ES EN 1991-1-4:2015 section 4.3.1(1) P, expression (4.3)

$$Vm(z) = Cr(z) \times Co(z) \times Vb$$

Where

Cr(z) Is the roughness factor, and

Co(z) Is the orography factor.

According to ES EN 1991-1-4:2015 section 4.3.1 the orography factor can be taken as 1 unless specified

The roughness factor accounts for the variability of the mean wind velocity of the size of the structure due to

- Height above ground level
- The ground roughness of the terrain up wind of the structure on the direction considered (ES EN 1991-1-4:2015 section 4.3.2(1))

The recommended procedure for determination of roughness factor $C_{o(Z)}$ at height Z is given by ES EN 1991-1-4:2015 expression (4.4) it is based on logarithmic velocity profile

$$Cr(z) = Kr * ln\left(\frac{z}{zo}\right) \quad for\ Zmin \le Z \le Zmax\ or\ \ Cr(z) = Cr(zmin) \quad for\ z \le zmin$$

Where

$\mathrm{k_r}$ is the terrain factor depending on roughness length $\mathrm{z_o}$.

The terrain factor K_r should be determined using ES EN 1991-1-4:2015 expression (4.5).

$$Kr = 0.19\left(\frac{Zo}{Z0,II}\right)^{0.07}$$

Where

$Z_{0,II}$ Is 0.05 (for terrain category II, ES EN 1991-1-4:2015,Table 4.1),

Z_{min} Is minimum height depend on terrain category, and

Z_{max} Is maximum height is to be taken 200m unless otherwise defined

Z_o Is Roughness length depends on terrain category.

We have categorized Addis Ababa under terrain category IV. According to ES EN 1991-1-4:2015, Table 4.1 the values of roughness length (z_o) and minimum height (z_{min}) are 1m and 10m respectively

The total height of our building which is from the ground to the top of the roof is 42.84m.

$Kr = 0.19\left(\frac{Zo}{Z0,II}\right)^{0.07}$

$K_r = 0.19\left(\frac{1}{0.05}\right)^{0.07} = 0.234$

$C_{r(z)} = k_r\, ln\frac{z}{z_o}$

$C_{r(z)} = 0.234 \times ln\left(\frac{42.84}{1}\right) = 0.88$ Where $10\text{m} < 42.84\text{m} < 200\text{m}$

$Vm(z) = Cr(z) \times Co(z) \times Vb$

$Vm(z) = 0.846 \times 1 \times 13.0524 m/sec$

$Vm(z) = 11.487 m/sec$

4.3.2 Wind turbulence

According to ES EN 1991-1-4:2015, section 4.4(11) is the turbulence intensity IV (z) at height z is defined as the standard deviation of the turbulence divided by mean wind velocity. And it should be determined using ES EN 1991-1-4:2015 expression (4.7).

$$l_{v(z)} = \frac{\sigma_v}{v_{m(z)}} = \frac{K1}{(c_{o(z)} * ln(\frac{zo}{z0,II})} \quad \text{for } Z_{min}<Z<Z_{max}$$

$$l_{v(z)} = l_{v(zmin)} \qquad \text{for } Z<Z_{min}$$

Where

k_1 Is the turbulence factor its recommended value is 1,

$c_{o(z)}$ Is orography factor ,

z_o Is roughness length, and

σ_v Is the standard deviation.

The recommended procedure for determination of the standard deviation is given by ES EN 1991-1-4:2015 expression 4.6.

$$\sigma_v = Kr * Vb * Kl$$

$\sigma_v = 0.234 \times 13.0524 m/sec \times 1$

$\sigma_v = 3.05$

$l_{v(z)} = \frac{1}{1 * In(42.84)}$

=0.27

4.3.3 Determine the peak velocity pressure

The peak velocity pressure $q_{p(2)}$at height z, which includes mean and short term velocity fluctuations, should be determined by ES EN 1991-1-4:2015 Expression (4.8)

$$qp(z) = [1 + 7lv(z)] * \frac{1}{2} * vm(z)^2 * \rho = Ce(z)qb$$

Where

ρ Is the air density which depends on the attitude, temperature, and barometric pressure to be expected in the region during wind storms, and

$c_{e(z)}$ Is exposure factor

The exposure factor $C_{e(z)}$ should be determined by ES EN 1991-1-4:2015 expression (4.9)

$$.C_{e(Z)} = q_{p(z)}/q_b$$

Where q_b is the basic velocity pressure, the recommended procedure for its determination is given by ES EN 1991-1-4:2015 expression 4.9.

$$q_b = \frac{1}{2} * \rho * v_b^2$$

According to ES EN 1991-1-4:2015, the recommended value of air density is 1.25 kg/m^3.

$$q_{p(42.84)} = 0.5\left[1 + 7 * l_{v(42.84)}\right] * \rho * V_{m(42.84)}^2$$

$$q_{p(42.84)} = [1 + 7 * 0.27] * 0.5 * 1.25\ kg/m^3 * \left(11.487 \frac{m}{sec}\right)^2)$$

$$q_{p(42.84)} = 0.238\ KN/m2$$

4.4 Wind pressure on surfaces

4.4.1 External wind pressure

The wind pressure acting on external surfaces, W_e, should be obtained using ES EN 1991-1-4:2015 expression 5.1.

$$We = q_p(Ze) * Cpe$$

Where

$q_p(Ze)$ Is the peak velocity pressure,

Cpe Is the reference height for the external pressure, and

Ze Is the pressure coefficient for the external pressure

The external pressure coefficients c_{pe} for buildings and parts of buildings depend on the size of the loaded area A, which is the area of the structure that produces the wind action in the section to be calculated. The external pressure coefficients are given for loaded areas A of 1 m^2 and 10 m^2 in in ES EN 1991-1-4:2015 table 7.4 for the appropriate building configurations as cpe,1, for local coefficients, and cpe,10, for overall coefficients, respectively (ES EN 1991-1-4:2015 section 7.2.1 (1)).

Note: the recommended procedure for calculating external pressure coefficient c_{pe} for building with loaded areas of between 1m^2 and 10m^2 is given in ES EN 1991-1-4:2015 figure 7.2 as follows: -

$$Cpe = Cpe,1 - (Cpe,1 - Cpe,10)\log_{10} A$$

Where

$Cpe,1$ Is external pressure coefficient for loaded area A of 1 m^2 ,

$Cpe,10$ Is external pressure coefficient for loaded area A of 10 m^2, and

A Is loaded area A.

The type of roof on our building is dual pitch roof. According ES EN 1991-1-4:2015 section 7.2.5 for dual patch roofs:

i. The roof, including protruding parts, should be divided in zones as shown in ES EN 1991-1-4:2015 figure 7.8.
ii. The reference height Z_e should be taken as h
iii. The pressure coefficients for each zone that should be used are given in ES EN 1991-1-4:2015 table 7.4.

Table 4-1 wind condition parameter

Wind direction at Θ = 0		Wind direction at Θ = 90	
Alpha (α)	15^0	Alpha (α)	15^0
cross wind dimension (b)	14.9m	cross wind dimension (b)	13.1m
Height of the building (h)	37.285m	Height of the building (h)	37.285
Truss span	15m	-	-

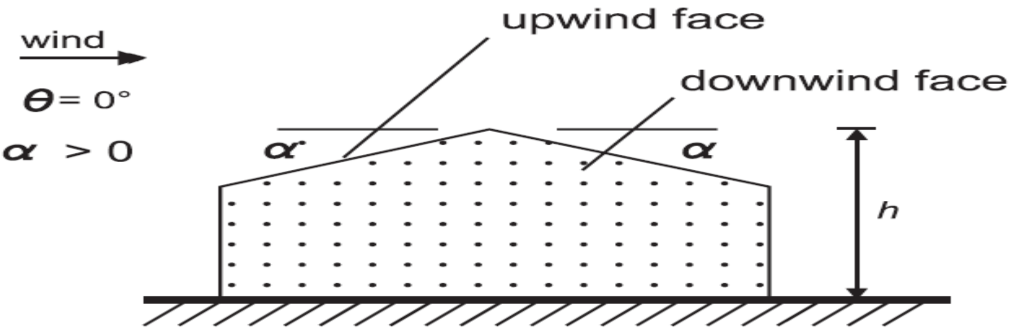
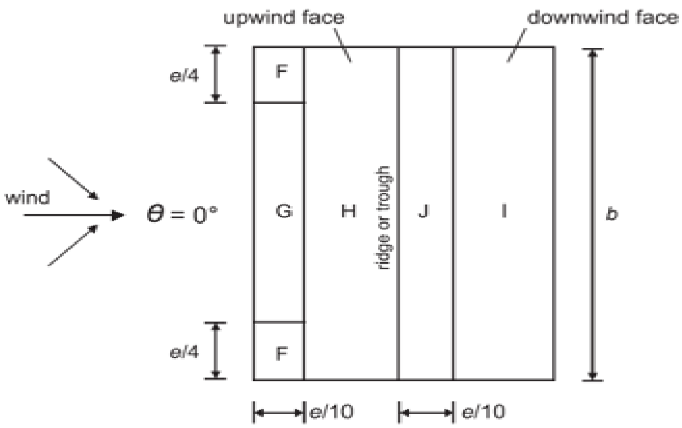


Figure 4-1 General for duo pitch roof when pitch angle is positive



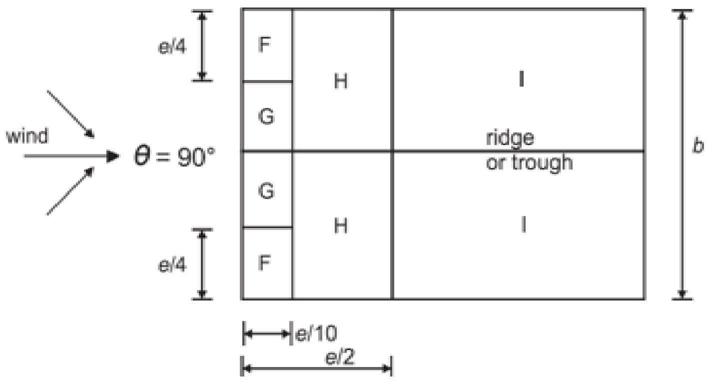
b: cross wind dimension = 14.9m

$e = \min(b, 2h)$

$e = \min(14.9m, 74.57m)$

$e = 14.9m$

Figure 4-2 Zones for wind direction Θ = 0



b: cross wind dimension= 13.1m

$e = \min(b, 2h)$

$e = \min(13.1m, 74.57m)$

$e = 13.1m$

Figure 4-3 Zones for wind direction Θ = 90

Table 4-2 External pressure coefficients, wind direction Ө = 0

Zone	F	G	H	J	I
Area (m^2)	5.55	11.1	75.4	22.2	75.4
Cpe,1	-1	-	-	-	-
	+0.2	-	-	-	-
Cpe,10	-0.9	-0.8	-0.3	-1	-0.4
	+0.2	+0.2	+0.2	0	0
Cpe	-1.18	-0.8	-0.3	-1	-0.4
	+0.2	+0.2	+0.2	0	0

Table 4-3 External pressure coefficients, wind direction Ө = 90

Zone	F	G	H	I
Area (m^2)	4.3	4.3	34.32	54.7
Cpe,1	-2	-2	-1.2	-0.5
Cpe,10	-1.3	-1.3	-0.6	-0.5
Cpe	-1.55	-1.55	-0.6	-0.5

Table 4-4 External wind pressure, wind direction Ө = 0

Zone	Cpe	q(zo) (KN/m^2)	We(KN/m^2)
F	-1.18	0.6357	-.0.75
	+0.2	0.6357	+0.127
G	-0.8	0.6357	-0.5
	+0.2	0.6357	+0.127
H	-0.3	0.6357	-0.19
	+0.2	0.6357	+0.127
I	-0.4	0.6357	-0.25
	0	0.6357	0
J	-1	0.6357	-0.6357
	0	0.6357	0

Table 4-5 External wind pressure, wind direction ϴ = 90

Zone	Cpe	q(ze) KN/m^2)	We (KN/m^2)
F	-1.55	0.6357	-0.98
G	-1.55	0.6357	-0.98
H	-0.6	0.6357	-0.38
I	-0.5	0.6357	-0.32

4.4.2 Internal wind pressure

The internal wind pressure is the wind pressure acting on the internal surfaces of a building which can calculate using ES EN 1991-1.4:2015 expression 5.2.

$$Wi = q_p(Z_i) * Cpi$$

Where

$q_p(Z_i)$ Is the peak velocity pressure,

Cpi Is the pressure coefficient for the internal pressure, and

Z_i Is the reference height for internal pressure

The internal pressure coefficient, c_{pi}, depends on the size and distribution of the openings in the building envelope (ES EN 1991-1.4:2015 section 7.2.9(1) P). The openings of a building include small opening such as: open windows, ventilators, chimneys, etc. as well as background permeability such as air leakage around doors, windows services and through the building envelope.

A face of a building should be regarded as dominant when the area of the opening at that face is at least twice the area of opening and leakages in the remaining faces of the building considered (ES EN 1991-1.4:2015 section 7.2.9(4). And for a building with a dominant face the internal pressure should be taken as a fraction of the external pressure at the openings of the dominant face. But according to ES EN 1991-1.4:2015 section 7.2.9(6) for buildings without a dominant face, the internal pressure coefficient c_{pi} should be determined from ES EN 1991-1.4:2015 figure 7.13, and is a function of the ratio of the height and the depth of the building, h/d, and the opening ratio μ for each wind direction θ, which should be determined from ES EN 1991-1.4:2015 expression (7.4).

$$\mu = \frac{\left(\sum area\ of\ openings\ where\ c_{pe}\ is\ negative\ or - 0.0\right)}{\sum area\ of\ all\ openings}$$

Where it is not possible, or not considered justified (which is our case), to estimate μ for particular case we can take c_{pi} as more onerous of +0.2 and -0.3.

Table 4-6 Internal wind pressure, wind direction Ө = 0

Zone	c_{pi}	$q_p(Z_i)$ (KN/m^2)	Wi (KN/m^2)
F	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19
G	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19
H	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19
I	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19
J	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19

Table 4-7 Internal wind pressure, wind direction Ө = 90

Zone	c_{pi}	$q_p(Z_i)$ (KN/m^2)	Wi (KN/m^2)
F	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19
G	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19
H	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19
I	+0.2	0.6357	+0.127
	-0.3	0.6357	-0.19

4.4.2.1 Net wind pressure

The net wind pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative (ES EN 1991-1.4:2004 article 5.2 (3)). Examples are show on the figure below

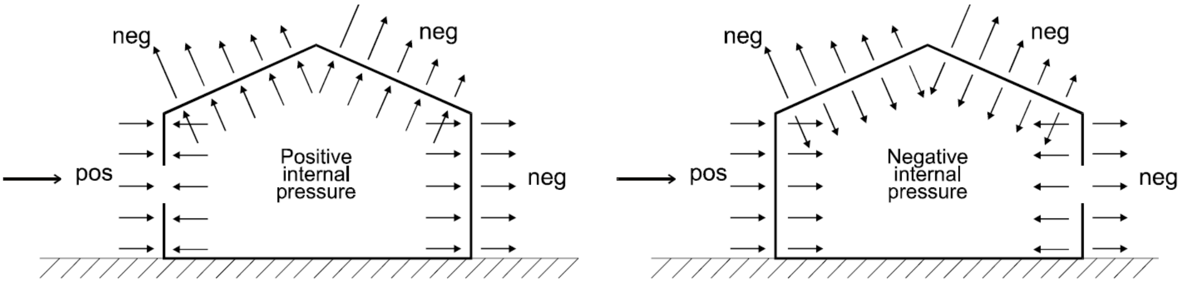


Figure 4-4 wind pressure on surfaces

Table 4-8 Net wind pressure, wind direction Ɵ = 0

Zone	We (KN/m^2)	Wi (KN/m^2)	Wnet (KN/m^2)
F	-0.75	+0.127	-0.88
	+0.127	-0.19	+0.317
G	-0.5	+0.127	-0.627
	+0.127	-0.19	+0.317
H	-0.19	+0.127	-0.317
	+0.127	-0.19	+0.317
I	-0.25	+0.127	-0.377
	0	-0.19	+0.19
J	-0.6357	+0.127	-0.763
	0	-0.19	+0.19

Table 4-9 Net wind pressure, wind direction Ɵ = 90

Zone	We (KN/m^2)	Wi (KN/m^2)	Wnet (KN/m^2)
F	-0.98	+0.127	-1.107
		-0.19	-0.79
G	-0.98	+0.127	-1.107
		-0.19	-0.79
H	-0.38	+0.127	-0.507
		-0.19	-0.19
I	-0.32	+0.127	-0.449
		-0.19	-0.13

The maximum wind surface pressure on the roof will be:

Compression = +0.377 KN/m^2

Suction = -1.107 KN/m^2

4.5 Analysis and Design of Purlin

Purlins are beams used on trusses to support the sloping roof system between the adjacent trusses. RHS, Channels, angle sections, and cold formed C- or Z-sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses. To avoid bending in the top chords of roof trusses, it is theoretically desirable to place purlins only at panel points.

A purling having a length of 4.8m and 3.76m are used in our roof system. And the spacing between the purlins is 1375mm.But we have designed the lattice purlin having a length of 4.08m because of having greater deflection than the purlin having a length of 3.76m. In addition to this the depth of the lattice purlin is 251mm and using 290mm the spacing between the reinforcement bars.

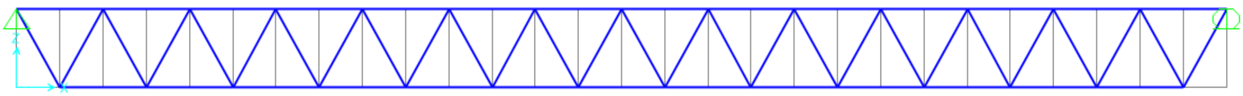


Figure 4-5 Lattice purlin

The analysis and design of the lattice purlin is carried out using SAP2000v19.2.0. And we have used atrial section of RHS 30x30x3 for the external cords from the kality manual specification. And a reinforcement bar having a diameter of 12mm and characteristic tensile strength of 400Mpa for the diagonal members The type of steel used form the specification for the external members is hot-rolled carbon steel sheets having yield strength, ultimate strength, and elongation of 313 KN/mm^2 ,393KN/mm^2, and 30mm respectively. The thickness of the still section is from 2mm to 6mm for structural hallow section.

Table 4-10 Mechanical properties of the steel section

Material for external members					
Type of steel	Weight of zinc coating	Yield strength(Mpa)	Tensile strength(Mpa)	Elongation (mm)	Thickness and purpose
Hot-rolled carbon steel	-	313	393	30	Frm2-6mm for pressed products, structural

sheets					hollow sections and general use

4.5.1 Purlin loading

4.5.1.1 Wind load on purlin

The wind load on purlin is the wind that is transferred from the roof cover; this is calculated as surface wind pressure (both external and internal surface pressures) on the wind analysis section of this document. And they were found as follows:

Maximum wind Compression pressure = +0.377 KN/m^2

Maximum wind Suction pressure = -1.107 KN/m^2

4.5.1.2 Dead load on purlin

In purling the dead load arises from the weight of the EGA sheet roof cover and the self-weight of the purling itself. The weight form the EGA sheet covering is calculated below after appropriate selection of EGA sheet but the self-weight of the purling is considered in SAP 2000 structural design software.

Selection EGA sheet

The EGA sheet is the top cover of the roof and is selected from the products catalogue of kality metal products factory. Taking a maximum wind surface load of 1.107 KN/m^2 and purling spacing of 1.375m, the load carrying capacity of EGA 300 having thickness of 0.35mm is 1.17 KN/m^2 which is greater than the maximum wind surface load on our roof. Therefor EGA 300 corrugated sheet is chosen as the roof cover our building.

Table 4-11 selected EGA sheet 300

Parameter	Purling spacing(m)	Thickness(mm)	Area (mm^2)	Weight (KN/m^3)
Value	1.375	0.35	350	77.08

$$\text{Dead load} = \text{weight of EGA sheet 300} * \text{thickness}$$

$$\text{Dead load} = 77.08\ \text{KN/m}^3 * 0.00035\text{m}$$

$$\text{Dead load} = 0.2698\text{KN/m}$$

4.5.1.3 Imposed load on purlin

In ES EN 1991-1.1:2015 table 6.9 roofs are categorized according to their accessibility into three categories and this are roofs not accessible except for normal maintenance and repair (category H), roofs accessible with occupancy according to categories A to D (category I) and roofs accessible for special services, such as helicopter landing areas (category K). in our case our building is an apartment building, so it is fair to say that our roof is inaccessible except for normal maintenance and repair which is roof the type of category H. Imposed loads for roofs of category H is within the range 0.00 KN/m2 to 1.0 KN/m2 for qk and Qk may be selected within the range 0.9 KN to 1.5 KN (ES EN 1991-1.1:2015 Table 6.10). The recommended values are: qk= 0.4 KN/m^2 and Qk= 1.0 KN.

4.5.2 Load transfer to purlin

Dead load = weight*thickness*purling spacing

Dead load = $77.08 KN/m^3 * 0.00035 m*1.375 m = 0.037$ KN/m

Imposed load (dis) = $0.4 KN/m^2 * 1.375 m = 0.55$ KN/m

Imposed load (conc) = 1KN

Wind load (compression) = $+0.377 KN/m^2 * 1.375 m = +0.518$ KN/m

Wind load (suction) = -1.107 KN/m^2 * 1.375m =-1.522 KN/m

4.5.3 Load combination for purlin

According to ES EN 1990:2015 section 6.4.3.1(1) P for each critical load case, the design values of the effects of actions (Ed) shall be determined by combining the value of actions that are considered to occur simultaneously. Dead load and imposed load or dead load and wind load that act on the purling have high probability of occurring simultaneously. But imposed loads and wind loads have low probability occurring simultaneously thus the combination of effects of this actions should be based on the design value of the leading variable action and the design combination values of the accompanying variable actions. Therefor various combination of ultimate limit state for persistent and transient design situations is used for design.

Values of ψ factor

Recommended value of ψ factored for the most common actions is obtained from ES EN 1990:2015 table A1.1 as follows: -

For actions of imposed loads category H: roofs $\psi_0 = 0$

For actions of wind loads on building $\psi_0 = 0.6$

Design value of actions in persistent and transient design situation

The load combination for transient and transient design situations is given in ES EN 1990:2015 A1.2 (B) expression 6.10 as follows: -

$Ed = \gamma_{Gj,sup} G_{kjsup} + \gamma_{Q1} Q_{k1} + \gamma_{Qi} \psi_{0,i} Q_{k,i}$

Where

$\gamma_{Gj,sup}$ Is the factor of safety for dead load (recommended 1.35)

G_{kjsup} Is dead load

γ_{Q1} Is the factor of safety of the leading variable (recommended 1.5)

Q_{k1} Is leading variable imposed load

$Q_{k,i}$ Is accompanying variable imposed load

$\psi_{0,i}$ Is multiplier for accompanying variable imposed load

Case 1: only dead load and imposed load

Combination 1, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(distributed)$

Combination 2, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(concentrated)$

Case 2: imposed load leading variable and wind load accompanying variable

Combination 3, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(distributed) + 1.5 * 0.6 Q_{k,i}(compression)$

Combination 4, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(concentrated) + 1.5 * 0.6 Q_{k,i}(compression)$

Combination 5, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(distributed) + 1.5 * 0.6 Q_{k,i}(suction)$

Combination 6, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(concentrated) + 1.5 * 0.6 Q_{k,i}(suction)$

Case 3: wind load leading variable and imposed load accompanying variable

Commination 7, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(compression) + 1.5 * 0 Q_{k,i}(distributed)$

Combination 8, $Ed = 1.35 G_{kjsup} + 1.5 Q_{k1}(compression) + 1.5 * 0 Q_{k,i}(concentrated)$

Combination 9, $Ed = 1G_{kjsup} + 1.5Q_{k1}(suction) + 1.5 * 0Q_{k,i}(distributed)$

Combination 10, $Ed = 1G_{kjsup} + 1.5Q_{k1}(suction) + 1.5 * 0Q_{k,i}(concentrated)$

4.6 Maximum bending moment and shear on the members result from SAP2000v19.2.0

Maximum bending moment

Table 4-12 Maximum bending moment (at the mid span)

Member	Design RHS section	Design combination	M_{major} (KNm)	V_{major} (KN)
Top	RHS 30X30X3	Comb4	0.043	0.37
Bottom	RHS 30X30X3	Comb8	0.019	0.05

Maximum shear

Table 4-13 Maximum shear force (support)

Member	Design RHS section	Design combination	M_{major}(KNm)	V_{major}(KN)
Top	RHS 30X30X3	Comb4	0.0241	0.36
Bottom	RHS 30X30X3	Comb8	0.015	0.092

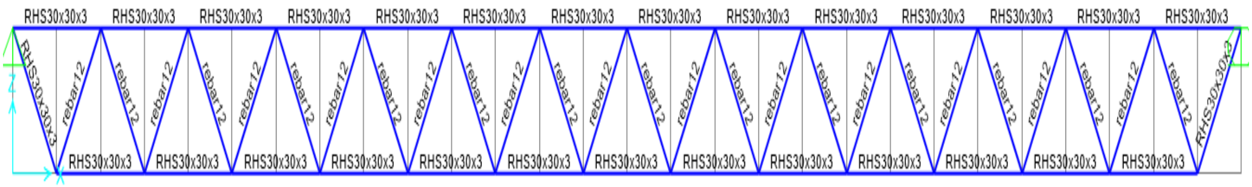


Figure 4-6 purlin sizing

From the analysis of the purlin using SAP2000V19.2.0 lattice purlin having cross-section of 30x30x3 is adequate. If we use the standard RHS purlin for the same loading a section having 100x60x4.5 RHS section is adequate also.

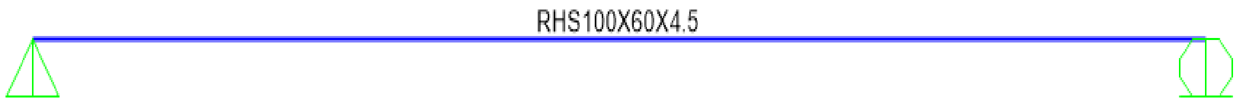


Figure 4-7 Standard RHS section

4.6.1 Comparison between lattice purlin and standard RHS purlin

In order to decide which type of the purlin to use we should to determine their mass and take the one which have the lesser mass.

Mass calculation

Mass per meter of standard RHS 100x60x4.5 from the kality manual is 10.26kg/m.

Length of the standard RHS100x60x4.5=4.8m

Mass of the standard RHS 100x60x4.5=10.26kg/m*4.8m=49.25kg

Total length of RHS30X30X3=0.29*14+0.29*15=8.41m

Total length of bar diameter=26*0.29=7.54m

$weight\ per\ meter\ of\ 12mm\ diameter = unitweight * Area\ of\ reinforcement\ bar$

$\frac{weight}{meter} = 7850\frac{KN}{m^3} * \frac{3.14*(12mm)^2}{4}$

$\mathrm{Weight/meter = 0.888KN/m}$

Table 4-14 Mass calculation of lattice purlin

Section	Length(m)	Kg/m	Kg
RHS30x30x3	8.41	2.36	19.85
ϕ12mm bar diameter	7.54	0.888	6.69
		Total	**26.54**

The total dead load of standard RHS purlin is 49.25kg and the total dead load of the lattice purlin is 26.54kg. The total dead weight of the standard RHS is almost twice of that lattice purlin. Due to this reason the total weight transferred to the truss will be increased. This makes the section to be used for the truss large which will make us uneconomical. Taking this into consideration we have used lattice purlin.

4.6.2 Serviceability limit state (Deflection requirement)

From the SAP2000V19.2.0 the maximum deflection is 3.6mm. And the allowable deflection according to ES EN 1993-1-1 Table 4.1the recommended value for vertical deflection.

Table 4-15 Vertical deflection calculation

Condition	Limiting value	
	δmax(mm)	δ2(mm)
Roof generally	L/200=4080/200=20.4mm	L/250=4080/250=16.32mm

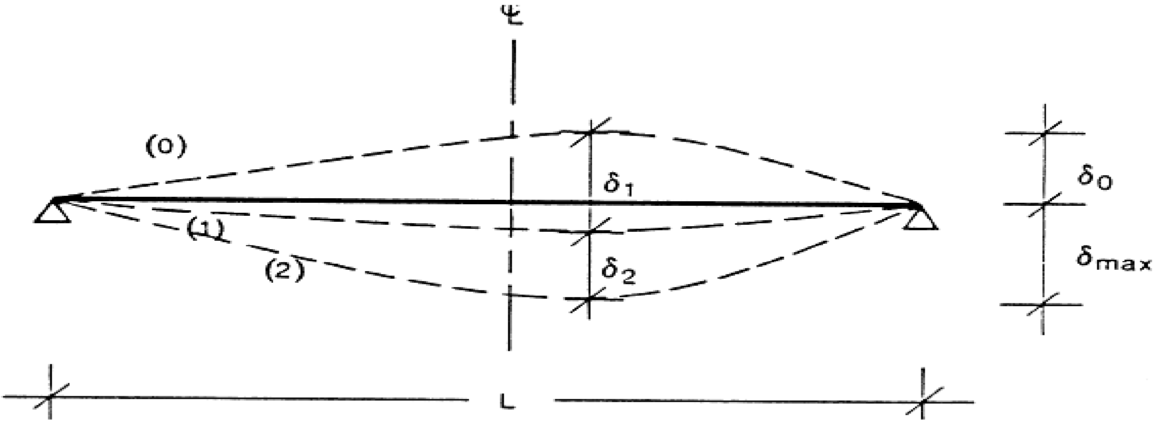


Figure 4-8 Vertical deflections to be considered

Since the maximum deflection is less than the limiting value of deflection the deflection requirements are satisfied.

4.7 Truss analysis and design

For covering large industrial or residential areas, to protect them against rain sun, dust or other natural vagaries, we require roofing. The material used for roofing are called covering, which may be range from tiles, corrugated steel sheets to light FRP covers, EGA sheets and tarpaulins. However, these materials are not structurally strong enough to support themselves and need to be supported by steel or concrete structures. Beams are some of the more common structural element to support roofs. But when the area and span also to be covered become too large, beams becomes too heavy and uneconomical as structural members. The next most common type of roof are roof supporting structures are truss elements, called roof truss.

Roof trusses are composed of tension and compression members joined together by welding or riveting. The loads supported on the roofing elements are transferred from purlin. The shape of the roof trusses are largely determined largely by the area and space to be covered, the used under which the covered premises is put and the type of roof cover used.

The span of the truss is15m having maximum spacing of 6m.

Table 4-16 Loads supported by the truss

Load type	Load
Load from EGA sheet	0.0269KN/m^2
Concentrated live load	1KN
Distributed live load	0.4KN/m^2
Wind load compression	0.377KN/m^2
Wind load suction	-1.107KN/m^2

Load transferred to the joints of the truss members can be computed by

$$P = w * \frac{(l_{ts,1} + l_{ts,2})}{2} * \frac{(l1 + l2)}{2}$$

Where

P is load at the node,

w is the areal load on the roof,

$l_{ts,1}$ Spacing between the truss under consideration and the truss on the left side it,

$l_{ts,2}$ Spacing between the truss under consideration and the truss on the right side it,

l_1 Spacing between the nodes under consideration and the node on the left side of it, and

L_2 Spacing between the nodes under consideration and the nodes on the right side it.

Sample calculation

Loads at node 1

$\text{Superimposed load(Gk, super)} = 0.0269 * 3.92 * \frac{(1.525+1.5250)}{2} = 0.16\text{KN}$

$\text{Cocentrated live load(Qk, con)} = 0.4 * 3.92 * \frac{(1.525+1.5250)}{2} = 2.39\text{KN}$

$\text{Distributed live load} = 1 * 0.5 * 2 = 1\text{KN}$

$\text{wind load compression(Qwind, comp)} = 0.377 * 3.92 * \frac{(1.525+1.5250)}{2} = 2.25\text{KN}$

$\text{wind load suction(Qwind, suc)} = 1.107 * 3.92 * \frac{(1.525 + 1.5250)}{2} = -6.61\text{KN}$

The dead weight of the purlin is computed using SAP2000V19.2.0 which is 0.14KN**.**

The loads transferred from the purlin to the truss can be computed by following the same procedure .and the values at each node of the truss are tabulated as shown in the table below.

Since the loads transferred to the truss are inclined we can decompose the load by the pitch angle.

N.B The load combination used for the design of the roofing truss is the same as with the combinations used for the purlin design.

Table 4-17 Loads at each joints of the load

	Loads on each nods		Decomposed load	
Node	Load Type	Load at each node	X-direction	Z-direction
1	wind suction	-6.61	0.00	-6.61
	wind compression	2.25	0.00	2.25
	live concentrated	1.00	0.00	1.00
	live distributed	2.39	0.00	2.39
	Dead load	0.14	0.00	0.14
	super imposed	0.16	0.00	0.16
2	wind suction	-6.29	-1.63	-6.29
	wind compression	2.14	0.55	2.07
	live concentrated	1.00	0.26	0.97
	live distributed	2.27	0.59	2.19
	Dead load	0.14	0.03	0.13
	super imposed	0.15	0.04	0.14
3	wind suction	-1.96	-0.51	-1.89
	wind compression	2.03	0.53	1.96
	live concentrated	1.00	0.26	0.97
	live distributed	2.15	0.56	2.08
	Dead load	0.14	0.03	0.13
	super imposed	0.14	0.04	0.14
4	wind suction	-5.96	-1.54	-5.76
	wind compression	2.03	0.53	1.96
	live concentrated	1.00	0.26	0.97
	live distributed	2.15	0.56	2.08
	Dead load	0.14	0.03	0.13
	super imposed	0.14	0.04	0.14
5	wind suction	-5.96	-1.54	-5.76

	wind compression	2.03	0.53	1.96
	live concentrated	1.00	0.26	0.97
	live distributed	2.15	0.56	2.08
	Dead load	0.14	0.03	0.13
	super imposed	0.14	0.04	0.14
6	wind suction	-4.58	-1.19	-4.42
	wind compression	1.56	0.40	1.51
	live concentrated	1.00	0.26	0.97
	live distributed	1.65	0.43	1.59
	Dead load	0.14	0.03	0.13
	super imposed	0.11	0.03	0.11
7	wind suction	-1.61	-0.42	-1.56
	wind compression	0.54	0.14	0.52
	live concentrated	1.00	0.26	0.97
	live distributed	0.58	0.15	0.56
	Dead load	0.14	0.03	0.13

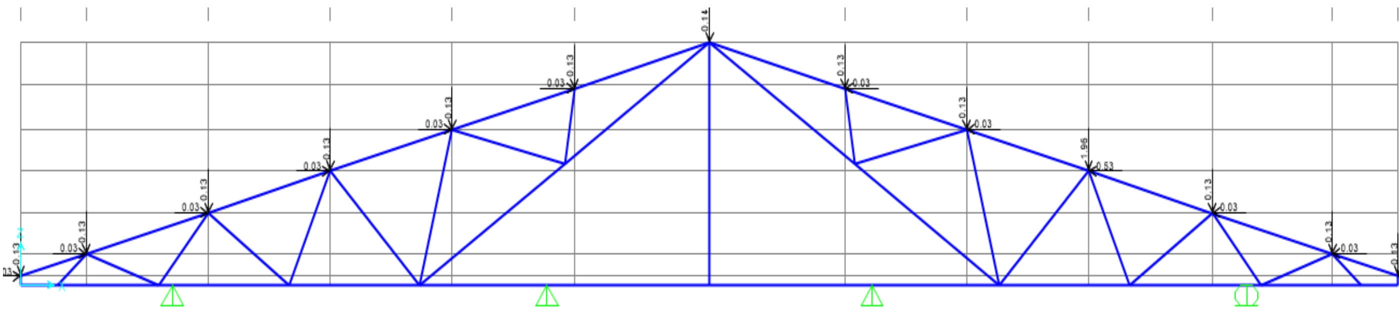


Figure 4-9 General loading of roofing truss

The truss elements are designed using SAP200V19.2.0 by taking a trial section for the governing design combination,

Table 4-18 Sizing of truss members

Cord member	Member	Governing combo	Size of the section
Top cord	Top	Combination 5	RHS 80X80X3.6
Bottom cord	Bottom	Combination 5	RHS 80X80X3.6
Vertical cord	Vertical	Combination 5	RHS 80X80X3.2
Diagonal member	Member1	Combination 5	RHS 30X30X3.2
	Member2	Combination 5	RHS 50X30X3.2

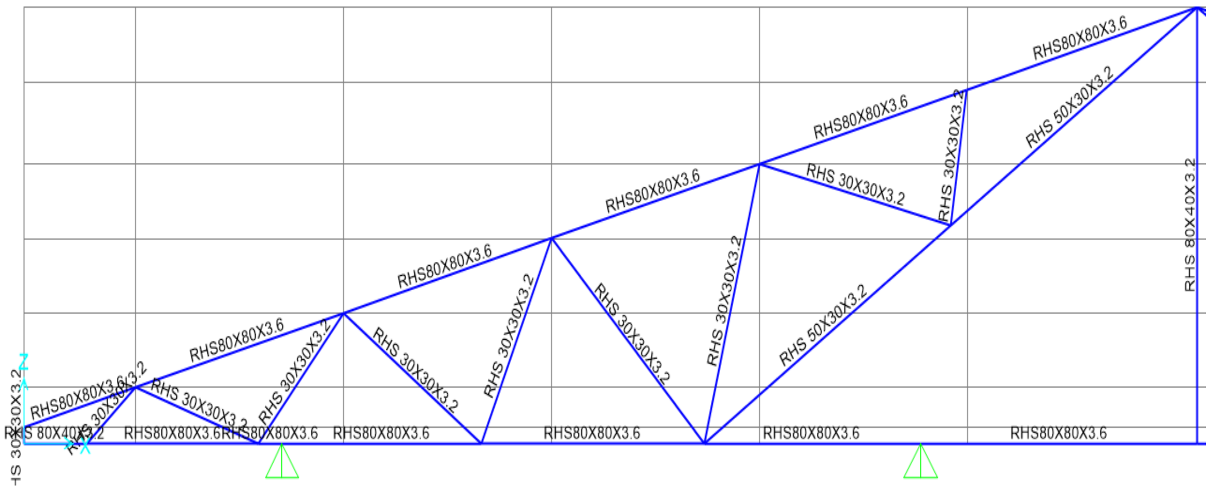


Figure 4-10 Sizing of truss section

5 Design for Earthquake Resistance

5.1 Introduction

Earthquake-resistant design can be considered as the art of balancing the seismic capacity of structures with the expected seismic demand to which they may be subjected. In this sense, earthquake-resistant design is the mitigation of seismic risk, which may be defined as the possibility of losses (human, social or economic) due to the effects of future earthquakes. Seismic risk is often considered as the convolution of seismic hazard, exposure and vulnerability.

Exposure refers to the people, buildings, infrastructure, commercial and industrial facilities located in an area where earthquake effects may be felt; exposure is usually determined by planners and investors, although in some cases avoidance of major geo-hazards may lead to relocation of new infrastructure.

Vulnerability is the susceptibility of structures to earthquake effects and is generally defined by the expected degree of damage that would result under different levels of seismic demand; this is the component of the risk equation that can be controlled by engineering design. Seismic hazards are the potentially damaging effects of earthquakes at a particular location, which may include surface rupture, tsunami run-up, liquefaction and landslides, although the most important cause of damages on a global scale is earthquake induced ground shaking. The focus is exclusively on this particular hazard and the definition of seismic actions in terms of strong ground motions. In the context of probabilistic seismic hazard analysis (PSHA), seismic hazard actually refers to the probability of exceeding a specific level of ground shaking within a given time.

If resources were unlimited, seismic protection would be achieved by simply providing as much earthquake resistance as possible to structures. In practice, it is not feasible to reduce seismic vulnerability to an absolute minimum because the costs would be prohibitive and certainly not justified since they would be for protection against a loading case that may not even occur during the useful life of the structure. Seismic design therefore seeks to balance the investment in provision of seismic resistance against the level of damage, loss or disruption that earthquake loading could impose. For this reason, quantitative assessment and characterization of the expected levels of ground shaking constitute an indispensable first step of seismic design, and it is this process of seismic hazard analysis.

5.2 Earth quake analysis

According to ES EN 1998-1:2015 section 4.3.3 there are three methods of earth quake analysis

5.2.1 Lateral force method of analysis

The lateral force method of analysis is a simplified approach widely used for simple structures in seismic standards and codes. It is based on the assumption that the influence of higher vibration modes is negligible. According to ES EN 1998-1:2015, 'this type of analysis may be applied to buildings whose response is not significantly affected by contributions from

modes of vibration higher than the fundamental mode in each principal direction' ES EN 1998-1:2015 considers that this requirement is deemed to be satisfied in buildings that are regular in elevation and do have a fundamental period less than 2 s and four times the corner period T_c of the applicable design spectrum.

ES EN 1998-1:2015 section 4.3.3.2.1 states that,

P This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction.

(1) The requirement in (1) P of this sub clause is deemed to be satisfied in buildings which fulfill both of the two following conditions.

(2) They have fundamental periods of vibration T_1 in the two main directions which is smaller than the following values

$$T_1 \le \begin{cases} 4T_c \\ 2.0s \end{cases}$$

Where Tc Is the upper limit of the period of the constant spectral acceleration branch which given in ES EN 1998-1:2003 (E) Table 3.2 or 3.3

They meet the criteria for regularity in elevation given in ES EN 1998-1:2015 section 4.2.3.3

5.2.2 Modal response spectrum analysis

ES EN 1998-1:2015 section 4.3.3.3.1 states that,

(1) P This type of analysis shall be applied to buildings which do not satisfy the conditions given ES EN 1998-1:2015 in 4.3.3.2.1(2) for applying the lateral force method of analysis.

(2) P The response of all modes of vibration contributing significantly to the global response shall be taken into account.

5.2.3 Non-linear methods

ES EN 1998-1:2015 section 4.3.3.4.1 states that,

(1) P the mathematical model used for elastic analysis shall be extended to include the strength of structural elements and their post-elastic behavior.

Selecting an appropriate type of analysis for our building

Check to use lateral force method of analysis

Requirements:

a) $T_1 \leq \begin{cases} 4T_c \\ 2.0s \end{cases}$ Where T_1 is fundamental period of vibration

Then according to ES EN 1998-1:2015 section 4.3.3.2.2 expression 4.6 for buildings with heights of up to 40 m the value of T1 (s) may be approximated by the following expression

$T_1 = C_t H^{3/4}$ *b*ut our building height is greater than 40 m so use

$$T_1 = 2 \times \sqrt{d}$$

Where

d is the lateral elastic displacement of the top of the building, in m, due to the Gravity loads applied in the horizontal direction and obtained from ETABS 2016 analysis data.

$T_1 = 2\sqrt{0.121}$, where d=120.92mm

T_1=0.7sec

As we have given our site is ground type D and we assumed type 2 spectrum

According to ES EN 1998-1:2015 section 3.2.2.2 table 3.3

$$T_c = 0.3$$

$T_1 \leq \begin{cases} 4T_c \\ 2.0s \end{cases}$

$0.7\ \text{sec} \leq \begin{cases} 4 * 0.3 \\ 2\ \text{sec} \end{cases}$

$0.7\ \text{sec} \leq \begin{cases} 1.2\ \text{sec} \\ 2\ \text{sec} \end{cases}$………………. ok Check the criteria for regularity in elevation

b) According to ES EN 1998-1:2015 section 4.2.3.3 the criteria for regularity in elevation are: All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building. Both the lateral stiffness and the mass of the individual stories shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.

In framed buildings, the ratio of the actual story resistance to the resistance required by the analysis should not vary disproportionately between adjacent stories. In our building, no set backs are present as our building satisfy all above criteria. It is also regular in elevation.

5.3 Base shear force

According to ES EN 1998-1:2015 section 4.3.3.2.2

(1) P The seismic base shear forceF_b, for each horizontal direction in which the building is analyzed, shall be determined using the following expression:

$$F_b = S_d(T_1).m.\lambda$$

Where

$\mathrm{S_d(T_1)}$	Is the ordinate of the design spectrum at period T_1,
$\mathrm{T_1}$	Is the fundamental period of vibration of the building for lateral motion in the direction considered
M	Is the total mass of the building, above the foundation or above the top of a rigid basement, and
λ	Is the correction factor, the value of which is equal to: λ = 0,85 if $T_1 < 2\ T_C$ and the building has more than two story, or λ = 1,0 otherwise

Within the scope of ES EN 1998 the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an “elastic response spectrum”.

5.4 Design spectrum for elastic analysis

According to ES EN 1998-1:2015 section 3.2.2.5, the design spectrum for elastic analysis of the seismic action, $S_d(T)$, is defined by the following expressions

$$0 \leq T \leq T_B: S_d(T) = a_g.S.\left[\frac{2}{3} + \frac{T}{T_B}.\left(\frac{2.5}{\eta} - \frac{2}{3}\right)\right]$$

$$T_B \leq T \leq T_c: S_d(T) = a_g.S.\frac{2.5}{q}$$

$$T_C \leq T \leq T_D: S_d(T)\begin{cases} = a_g.S.\frac{2.5}{q}.\left[\frac{T_C}{T}\right] \\ \geq \beta * a_g \end{cases}$$

$$T_D \leq T : S_d(T)\begin{cases} = a_g.S.\frac{2.5}{q}.\left[\frac{T_C.T_D}{T^2}\right] \\ \geq \beta * a_g \end{cases}$$

Where

T	is the vibration period of a linear single-degree-of-freedom system
a_g	is the design ground acceleration on type A ground ($a_g = \gamma_I.a_{gR}$),
T_B	is the lower limit of the period of the constant spectral acceleration branch
T_C	is the upper limit of the period of the constant spectral acceleration branch
T_D	is the value defining the beginning of the constant displacement response range of the spectrum
S	is the soil factor, and
β	is the lower bound factor for the horizontal design spectrum

Values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

According to ES EN 1998-1:2015 section 2.2.5 for ground type D and for type 2 elastic response spectra the values of S, T_B, T_C and T_D taken from ES EN 1998-1:2015 table 3.3

Table 5-1 Type 2 elastic response spectra for ground type D

S	T_B	T_C	T_D
1.8	0.1	0.3	1.2

For $T_C \leq T_1 \leq T_D$

$$S_d(T_1) = max\begin{cases} a_g * S\frac{2.5}{q}\left(\frac{T_c}{T_!}\right) \\ \beta * a_g \end{cases}$$

The design ground acceleration, $a_g = \gamma_1 a_{gR}$, but $a_{gR} = a_0$ from national annex of Ethiopia from zonation map Ethiopia for Addis Ababa

$\frac{a_0}{g} = 0.1$ So $a_0 = 0.1 * g = 0.1*9.81\frac{M}{S^2} = 0.981\frac{M}{S^2}$

γ_1 Is importance factor and According to ES EN 1998-1:2015 section 4.2.5 table 4.3 for importance class II and Ordinary buildings, not belonging in the other categories shall be by definition equal to 1.0

Ag =1*0.981 $\frac{M}{S^2}$=0.981 $\frac{M}{S^2}$

q The behavior factor, according to EN 1998-1:2015 section 5.3.3 a q of 1.5 may be used in deriving the seismic action for DCM regardless of the structural system and the regularity in elevation.

$$S_d(T_1) = \max\begin{cases} 0.981 \ \frac{M}{S^2} * 1.8 * \frac{2.5}{1.5} * \left[\frac{0.3sec}{0.7sec}\right] \\ 0.2 * 0.981 \ \frac{M}{S^2} \end{cases}$$

$$S_d(T_1) = \max\begin{cases} 1.26 \\ 0.196 \end{cases} = 1.26\frac{M}{S^2}$$

The correction factor λ

$\lambda = 0.85$ If $T_1 \leq 2T_c$ or otherwise $\lambda = 1$

$T_1 = 0.7$ sec and $2T_c = 0.6$

$0.7 \leq 0.6$sec…………………not ok

Then $\lambda = 1$

5.5 The total mass of the building

According to ES EN 1998-1:2015 section 3.2.4

The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\sum G_{k,j} + \sum \psi_{E,i} * Q_{k,i}$$

Where

$\psi_{E,i}$ is the combination coefficient for variable action

$G_{k,j}$ Characteristic value of permanent action, j

$Q_{k,i}$ Characteristic value of loading variable action, I

The combination coefficient, $\psi_{E,i}$

$$\psi_{E,i} = \varphi * \psi_{2,i}$$

Where

φ Is 0.8 for story with correlated occupancies and for category A building according to ES EN 1998-1:2003 section 4.2.4 table 4.2, and

$\psi_{2,i}$ According to ES EN 1998-1:2003 section A1.2.2 table A1.1 for category A Building The recommended value is 0.3.

Then $\psi_{E,i} = 0.8 * 0.3 = 0.24$

Therefore $\sum G_{k,j} + \sum 0.24 * Q_{k,i}$

Table 5-2 Weight summery of typical floor and its mass center

Item	Total weight(kN)		mass center X(m)	mass center Y(m)	weight*x	weight*y
Column		153.6	7.55	6.933	1159.68	1074.17
Beam	-	352.605	7.53	6.834	2655.11565	2409.703
floor finish	-	38.432	7.547	6.432	290.046304	247.1946
Plastering	-	56.813	7.547	6.432	428.767711	365.4212
Slab	-	650.531	7.516	6.403	4889.390996	4165.35
stair case	-	57.927	7.053	11.67	408.559131	676.0081
shear wall	-	89.6	7.33	11.164	656.768	1000.294
wall and partition wall	-	569.78	6.588	7.28	3934.48536	4347.762
live load	-	80.21	7.547	6.432	605.34487	515.9107
cement screed	-	100.3	7.547	6.432	870.146459	741.5903

Total	-	2653.035	-	-	19377.34448	18728.87
		Mass center of floor (m)		G-1st	7.30	7.05
				2nd-7th	7.28	7.07
				8th-10th	7.25	7.08

Table 5-3 Weight summery of roof and its mass center

item	Total mass(KN	mass center X(m)	mass center Y(m)	weight*x	weight*y
top tie beam	175	7.55	6.983	1321.25	1222.02
roof load	258.23	7.55	6.56	1949.63	1693.98
Total	433.23			3270.88	2916.01
			mass center of roof(m)	7.55	6.73

Table 5-4 Summary of storey weight with their mass center

storey	weight(kN)	mass center X(m)	mass center Y(m)
GROUND	967.05	7.30	7.05
1ST	2610.598	7.30	7.05
2nd	2341.798	7.28	7.07
3rd	2341.798	7.28	7.07
4th	2341.798	7.28	7.07
5th	2341.798	7.28	7.07
6th	2341.798	7.28	7.07
7th	2341.798	7.28	7.07
8th	2149.789	7.25	7.08
9th	2149.789	7.25	7.08
10th	2149.789	7.25	7.08

ROOF	433.23	7.55	6.73
Total	24511.033		

From the above Table 5-4 the total weight of the building is 24,511.033 kN.

$$\text{mass of the building}, m = \frac{24511.033 * 10^3 N}{9.81\, {}^{m}/_{s^2}} = 2{,}498{,}573\ kg$$

$$\text{The base shear force}, F_b = S_d(T_1). m. \lambda$$

Where

$S_d(T_1)$.	$0.358 {}^{m}/_{s^2}$
M	$2{,}498{,}573\ kg$
λ	1

$$F_b = 0.358\ {}^{m}/_{s^2} * 2{,}498{,}573\ \ kg * 1 = 894.49\ KN$$

5.6 Distribution of horizontal seismic force

According to ES EN 1998-1:2015 section 4.3.3.2.3 when the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i should be taken as being given by:

$$F_i = F_b * \frac{Z_i m_i}{\sum Z_j m_j}$$

Where

m_i, m_j Are the story mass computed in accordance to ES EN 1998-1:2015 section 3.2.4

Z_i, Z_j Are the heights of the masses m_i , m_j above the level of application of the seismic action (foundation or top of a rigid basement).

Table 5-5 Distribution of seismic force

storey	weight (kN)	height (m)	weight*height	(weight*height)/∑(weight*height)	F_b (kN)	F_i(kN)
Ground	967.05	2.5	2417.25	0.005111752	894.49	4.572410792
1st	2610.598	5.7	14880.4086	0.031462677	894.49	28.14304984
2nd	2341.798	8.9	20842.0022	0.044067686	894.49	39.41810487
3rd	2341.798	12.1	28335.7558	0.059912248	894.49	53.59090662
4th	2341.798	15.3	35829.5094	0.075756809	894.49	67.76370838
5th	2341.798	18.5	43323.263	0.091601371	894.49	81.93651013
6th	2341.798	21.7	50817.0166	0.107445932	894.49	96.10931188
7th	2341.798	24.9	58310.7702	0.123290494	894.49	110.2821136
8th	2149.789	28.1	60409.0709	0.127727076	894.49	114.2505921
9th	2149.789	31.3	67288.3957	0.142272508	894.49	127.2613357
10th	2149.789	34.5	74167.7205	0.15681794	894.49	140.2720793
Roof	433.23	37.7	16332.771	0.034533507	894.49	30.8898768
	∑(weight*height)		472954.3089			

6 Frame analysis

A frame system is a structural system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65 % of the total shear resistance of the whole structural system. A minimum torsional rigidity should also be provided. This chapter entails the 3D modeling of the structure using ETABS V16.1.0. The modeling is based on the loads obtained from chapters 2, 3, 4 and 5 also including the frame system of the building.

6.1 Accidental Torsional Effects

According to ES EN 1998-1:2015 section 4.3.2 in order to account for un certainties in the location of masses and in the spatial variation of the seismic motion ,the calculated center of mass at each floor i shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity:

$$e_{ai} = \pm 0.05 * L_i$$

Where

e_{ai} is the accidental eccentricity of storey mass i from its nominal location, applied in the direction at all floors; and

L_i is the floor dimension perpendicular to the direction of the seismic action

$L_{ix} = 15m \; and L_{iy} = 13.5m$ Then, $e_{aix} = \pm 0.75m \; and \; e_{aiy} = \pm 0.675m$

Table 6-1 Accidental eccentricity

Storey	Center of mass		Displaced center of mass			
	X(m)	Y(m)	X^- (m)	X^+ (m)	Y- (m)	Y^+ (m)
Ground – 10th floor	7.304	7.059	6.554	8.054	6.384	7.734
Roof	7.55	6.78	6.8	8.3	6.105	7.455

6.2 Stiffness modifier

Stiffness modifiers are used to account for the cracking of reinforced concrete sections. In the absence of stiffness modifiers, the structure would be stiffer and thus attract higher lateral forces due to earthquake. Henceforth, the outcome may be a heavily reinforced shear walls, moment frames etc. At the same time, under estimation of the drift may occur.

According to ES EN 1998-1:2015 section 4.3.1 Unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to one-half of the corresponding stiffness of the un cracked elements.

Shear area in X-direction = 0.5

Shear area in Y-direction = 0.5

Moment of inertia about X-direction = 0.5

Moment of inertia about Y-direction = 0.5

Torsion constant = 0.1

6.3 Load combination

The design load combinations are used for determining the various combinations of load cases for which the structure needs to be designed. The load combinations factors are applied to the forces obtained from the associated load cases and are summed to obtain the factored load design forces. Therefore, based on ES EN 1990:2015 section 6.4.3.2 and 6.4.3.4 and ES EN 1992-1-1:2015 section 5.2 (geometric imperfection) the following combinations are obtained:

Serviceability limit state

1. $E_d = \sum G_{k,j} + Q_{k,1} = G_{k,i} + Q_{k,1}$

Ultimate limit state

2. $E_d = 1.35G_{k,j} + 1.5Q_{k,1}$

Seismic load combination for lateral load method of analysis with geometric imperfection Seismic actin $E_d = \sum G_{k,j} + \gamma_1 A_{Ek} + \sum \Psi_{2,i} Q_{k,i}$

3. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ + 0.3EQY^+ + IMPX$

4. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ + 0.3EQY^+ - IMPX$

5. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ + 0.3EQY^+ - IMPY$

6. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ + 0.3EQY^+ + IMPY$

7. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^+ + IMPX$

8. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^+ - IMPX$

9. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^+ - IMPY$

10. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^+ + IMPY$

11. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ + 0.3EQY^- + IMPX$

12. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ + 0.3EQY^- - IMPX$

13. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ + 0.3EQY^- - IMPY$

14. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^- + IMPY$

15. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^- + IMPX$

16. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^- - IMPX$

17. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^- - IMPY$

18. $E_d = G_{k,j} + 0.3Q_{k,i} + EQX^+ - 0.3EQY^- + IMPY$

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Where

$EQX^{\pm}$ is the seismic action in the X-direction;

$EQY^{\pm}$ is the seismic action in the Y-direction;

IMPX is the imperfection factor in the X-direction; and

IMPY Is the imperfection factor in the Y-direction

6.4 Geometric imperfection

Geometric imperfection is the buckling or deviation of the structure laterally due to the loads applied on it. This occurs on structures that have thin wall structures, that are steel structures, structures that are not laterally braced, and structures that are highly susceptible to buckling. So, these loads must be taken into account in designing a certain building because they affect the stability of the structure. Buildings such as this type are subjected to loads that result the lateral buckling of the building since they are subjected to cases such as geometric imperfection in their construction as well as the effect resulting from the frame actions that make the building to have a sway system and result the buckling.

According to ES EN 1992-1-1:2015 section 5.2, the unfavorable effects of possible deviations in the geometry of the structure and the position of loads shall be taken into account in the analysis of members and structures Imperfections shall be taken into account in ultimate limit states in persistent and accidental design situations and need not be considered for serviceability limit states.

The imperfection may be represented by an inclination,θ_i, given by

$$\theta_i = \theta_0 * \alpha_n * \alpha_m$$

Where

θ_0 is the basic value, recommended value is $1/200$;

α_n is the reduction factor for height; $\alpha_n = {}^{2}/_{\sqrt{l}}; 2/3 \leq \alpha_n \leq 1$

α_m Is the reduction factor for number of member;

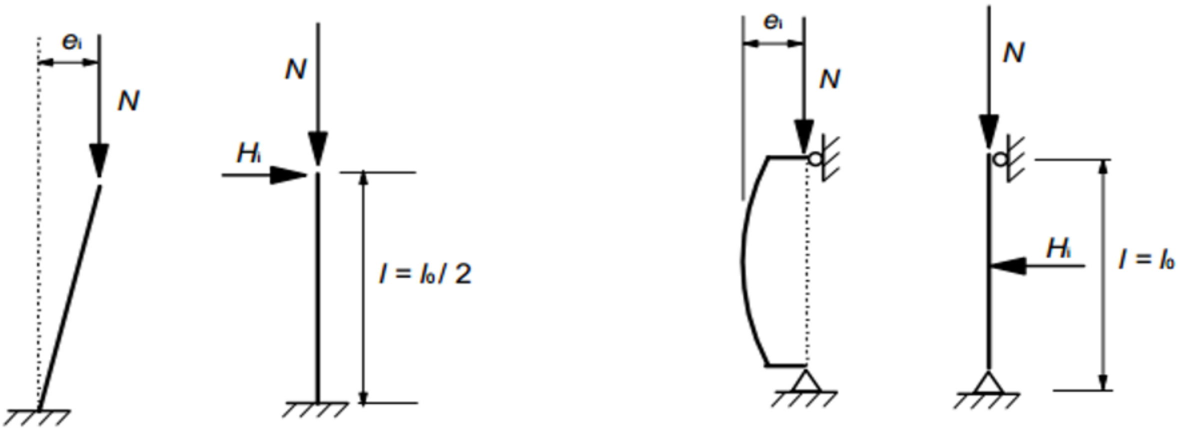
$$\alpha_m = \sqrt{0.5 * (1 + 1/m)}$$

l Is the length or height(m), and

m Is the number of vertical member’s contribution to the total effect

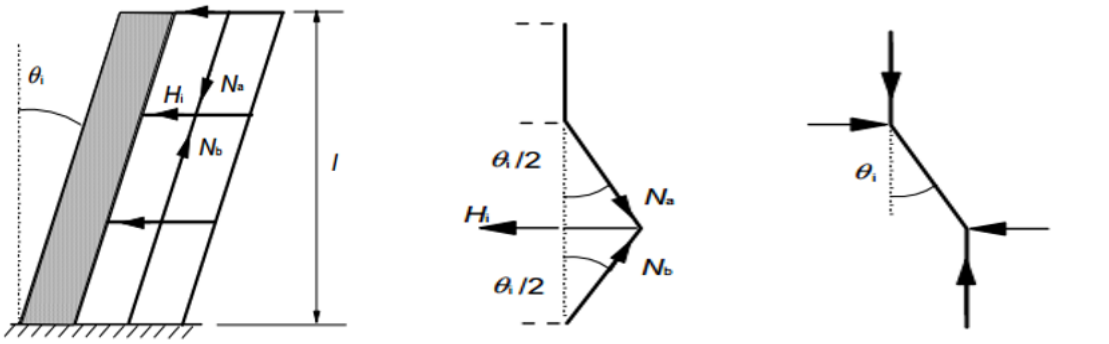
The definition of l and m depends on the effect considered, for which three main cases can be distinguished (see also Figure 6-1):

- Effect on isolated member: l = actual length of member, m =1.
- Effect on bracing system: l = height of building, m = number of vertical members contributing to the horizontal force on the bracing system.
- Effect on floor or roof diaphragms distributing the horizontal loads: l = storey height, m = number of vertical elements in the storey(s) contributing to the total horizontal force on the floor.



(a1) unbraced (a2) braced

a, Isolated members with eccentric axial force or lateral force



(b) bracing system (c) floor diaphragm (d) roof diaphragm

Figure 6-1Examples of geometric imperfection

For Effect on bracing system, $\mathrm{l} = 35.2\mathrm{m}$ and $\mathrm{m} = 12$(number of columns)

$$\alpha_n = {}^{2}/_{\sqrt{l}} = {}^{2}/_{\sqrt{35.2m}} = 0.337 \; but \, \tfrac{2}{3} \le \alpha_n \le 1 \; and \; \alpha_n = \tfrac{2}{3} = 0.667$$

$$\alpha_m = \sqrt{0.5 * (1 + 1/m)} = \sqrt{0.5 * \left(1 + \tfrac{1}{12}\right)} = 0.735$$

$$\theta_i = \tfrac{1}{200} * 0.667 * 0.735 = 0.00245$$

For structures, the effect of the inclination $\theta_i$may be represented by transverse forces, to be included in the analysis together with other actions.

Effect on bracing system, (see Figure 6-1b)

$$H_i = \theta_i * (N_b - N_a)$$

Effect on floor diaphragm, (see Figure 6-1c)

$$H_i = \theta_i * {}^{(N_b + N_a)}/_{2}$$

Effect on roof diaphragm, (see Figure 6-1d)

$$H_i = \theta_i * N_a$$

Where

N_b, N_a are vertical forces contributing to H_i

Table 6-2 Transverse forces due to geometric imperfection

Store's	load combination	Location	P(kN)	θ_i	$N_b - N_a$	H_i(kN)
Roof	Comb2:SLS	Bottom	467.6919	0.00245	467.692	1.14585
10th floor	Comb2:SLS	Bottom	3379.915	0.00245	2912.22	7.13495
9th floor	Comb2:SLS	Bottom	6294.04	0.00245	2914.12	7.1396
8th floor	Comb2:SLS	Bottom	9382.787	0.00245	3088.75	7.56743
7th floor	Comb2:SLS	Bottom	12471.54	0.00245	3088.75	7.56743
6th floor	Comb2:SLS	Bottom	15560.28	0.00245	3088.75	7.56743
5th floor	Comb2:SLS	Bottom	18649.03	0.00245	3088.75	7.56743
4th floor	Comb2:SLS	Bottom	21756.92	0.00245	3107.89	7.61434
3rd floor	Comb2:SLS	Bottom	24874.39	0.00245	3117.46	7.63779
2nd floor	Comb2:SLS	Bottom	28214.54	0.00245	3340.15	8.18336
1st floor	Comb2:SLS	Bottom	31554.68	0.00245	3340.15	8.18336

Ground floor	Comb2:SLS	Bottom	34664.25	0.00245	3109.57	7.61844

6.5 Safety verification

6.5.1 Ultimate limit state

According to ES EN 1998-1:2015 section 4.4.2 (2) Second-order effects (P-Δ effects) need not be taken into account if the following condition is fulfilled in all stores:

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h} \leq 0.1$$

Where

θ is the inter storey drift sensitivity coefficient;

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation;

d_r is the design inter storey drift, evaluated as the difference of the average lateral displacement d_s at the top and bottom of the storey under consideration and calculated in accordance ES EN 1991-1:2015 section 4.3.4;

V_{tot} is the total seismic storey shear; and

h is the inter storey height.

- If $\theta \leq 0.1$ no need to consider second order effect (non-sway frame)
- If $0.1 \leq \theta \leq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$.
- If $\theta > 0.2$ unstable frame and θ shall not exceed 0.3.

According to ES EN 1991-1:2015 section 4.3.4

$$d_s = q_d * d_e$$

Where

d_s is the displacement of a point the structural system induced by the design seismic action;

q_d is the displacement behavior factor, assumed equal to q unless otherwise specified; and

d_e is the displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum.

Sample calculation of Inter-storey drift sensitivity coefficient

For Roof

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h} \quad \text{, Where} \frac{d_r}{h} = \frac{d_e}{h} * q_d \text{, then}$$

$\frac{d_e}{h} =$ 0.0010 From output of ETABS

$q_d = 1.5$ for low ductility class

Then $\frac{d_r}{h} = 0.0010 * 1.5 = 0.0015$

$P_{tot} = 467.69kN$ Output of ETABS

$V_{tot} = 30.89kN$ Output of ETABS

$\theta = 0.02 < 0.1\ non\ sway$

Table 6-3 Inter storey drift coefficient

Story	Load combo	P_{tot}(kn)	V_{tot}(kn)	Direction	Drift($\frac{d_e}{h}$)	Q	$\frac{d_r}{h}$	θ	Stability
ROOF	Comb16	467.69	30.89	X	0.0010	1.5	0.0015	0.02	Non-sway
10th floor	Comb16	3064.94	171.16	X	0.0011	1.5	0.0017	0.03	Non-sway
9th floor	Comb16	5664.09	298.42	X	0.0016	1.5	0.0024	0.04	Non-sway
8th floor	Comb16	8437.87	412.67	X	0.0014	1.5	0.0021	0.04	Non-sway

7th floor	Comb16	11211.64	522.95	X	0.0016	1.5	0.0024	0.05	Non-sway
6th floor	Comb16	13985.42	619.06	X	0.0018	1.5	0.0027	0.06	Non-sway
5th floor	Comb16	16759.19	701	X	0.0019	1.5	0.0029	0.07	Non-sway
4th floor	Comb16	19552.11	768.76	X	0.0020	1.5	0.0030	0.08	Non-sway
3rd floor	Comb16	22354.61	822.35	X	0.0020	1.5	0.0030	0.08	Non-sway
2nd floor	Comb16	25379.78	861.77	X	0.0016	1.5	0.0025	0.07	Non-sway
1st floor	Comb16	28404.9567	889.91	X	0.0013	1.5	0.0019	0.06	Non-sway
Ground floor	Comb16	31213.2938	894.48	X	0.0005	1.5	0.0008	0.028	Non-sway

6.5.2 Damage limitation requirement

6.5.2.1 Limitation of inter storey drift

According to ES EN 1998-1:2015 section 4.4.3.2

- for buildings having non-structural elements of brittle materials attached to the structure:

  $d_r * v \leq 0.005h;$

- for buildings having ductile non-structural elements:

  $d_r * v \leq 0.0075h;$

- for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r * v \leq 0.01h \ .$$

Where

v Is the reduction factor which takes into account the lower period of the seismic action associated with the damage limitation requirement.

The value of the reduction factor v may also depend on the importance class of the building.

The recommended values of V are:

- $v = 0.4$ for importance classes III and IV and

- $v = 0.5$ for importance classes I and II

Sample calculation for Damage limitation requirement

For Roof

Here it is assumed that the buildings having a non-structural elements of brittle materials attached to the structure, then

$$d_r * v \leq 0.005h \; , \frac{d_r * v}{h} \leq 0.005$$

Where $\frac{d_r}{h} = \; 0.0010 * 1.5 = 0.0016$

$v = 0.5$ (For building of importance class II)

$\frac{d_r * v}{h} = 0.0016 * 0.5 = 0.00078 < 0.005 \ldots\ldots\ldots\ldots\ldots\ldots\ldots . ok$

Table 6-4 Damage limitation requirement

Storey	Load Combo	Drift	Q	dr/h	V	drV/h	maximum value	remark
ROOF	Comb16	0.0010	1.5	0.0016	0.5	0.00078	0.005	ok
10th floor	Comb16	0.0011	1.5	0.0017	0.5	0.00085	0.005	ok
9th floor	Comb16	0.0016	1.5	0.0024	0.5	0.00118	0.005	ok
8th floor	Comb16	0.0014	1.5	0.0021	0.5	0.00103	0.005	ok
7th floor	Comb16	0.0016	1.5	0.0024	0.5	0.00119	0.005	ok
6th floor	Comb16	0.0018	1.5	0.0027	0.5	0.00134	0.005	ok
5th floor	Comb16	0.0019	1.5	0.0029	0.5	0.00145	0.005	ok
4th floor	Comb16	0.0020	1.5	0.0030	0.5	0.00149	0.005	ok
3rd floor	Comb16	0.0020	1.5	0.0030	0.5	0.00148	0.005	ok
2nd floor	Comb16	0.0016	1.5	0.0025	0.5	0.00123	0.005	ok
1st floor	Comb16	0.0013	1.5	0.0019	0.5	0.00096	0.005	ok
Ground floor	Comb16	0.0005	1.5	0.0008	0.5	0.00041	0.005	ok

7 Beam Analysis and Design

7.1 Introduction

A beam is a structural element that is capable of withstanding load primarily by resisting bending. The bending force induced into the material of the beam as result of the external loads, own weight, span and external reactions to these loads is called bending moment. Beams are structural members that transfer the load by developing moment and shear force. Is axial forces are developed in the beam, and then it is no longer a beam but designed as beam column. Beams are usually horizontal. When beams are curved in elevation, they are known as arch beams. Arch beams are not technically beams because they only carry compression force. Since arch beams do not carry bending moments. They have no deflection, hence having small beams.

Reinforced concrete beam analysis and design consists primarily of producing member details which will adequately resist the ultimate bending moments, shear forces and also torsional moments if necessary. At the same time serviceability requirements must be considered to ensure that the member will behave satisfactorily under working loads. It is difficult to separate these two criteria; hence the design procedure consists of a series of interrelated steps and checks. These steps may be condensed into three basic design stages which are preliminary analysis and member sizing, detailed analysis and design of reinforcement and serviceability calculations. The materials in the design of beams depends on the theory and design specification of concrete and reinforcement described in chapter one section 1.6 of this document And the design of beams is in accordance with ES EN 1992-1.1:2015.

7.2 Basic principles and assumptions

Although the method used in the analysis and design of reinforced concrete beams are different from those used in the design of homogenous beams such as structural steel, the fundamental principles are essentially the same. Accordingly, the basic equations for flexural design of beams are derived based on the following basic principles and assumptions at ultimate limit state described on ES EN 1992-1.1:2015 section 6.1(2) P.

1. Internal stress resultants such as bending moments, shear forces etc. at any section of the member are in equilibrium with the external action effects.
2. Plane sections before bending remains plane after bending.

3. The strain in bonded reinforcement or bonded pre stressing tendons, whether in tension or in compression, is the same as that in the surrounding concrete.
4. The tensile strength of concrete is ignored.
5. The stresses in the concrete in compression are derived from the design stress/strain relationship given in ES EN 1992-1.1:2015 section 3.1.7 stress-strain relations for the design of cross-sections.

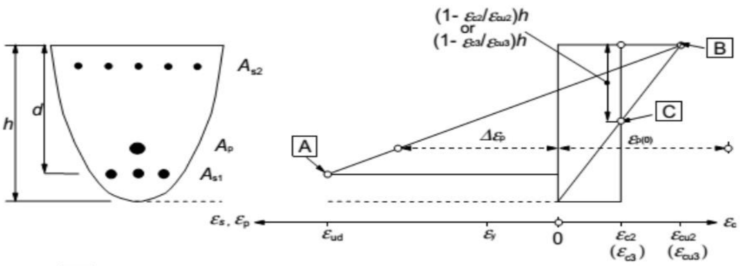


Figure 7-1 possible Strain diagram at Ultimate Limit State ES EN-1992-1.1:2015 Figure 6.1

7.3 Preliminary analysis and beam sizing

The layout and size of members are very often controlled by architectural details and clearances for machinery and equipment. Therefore, it is necessary to either check that beam sizes are adequate to carry the loading, or alternatively, decide on the sizes that are adequate. The preliminary analysis need only provide the maximum moments and shears in order to ascertain reasonable dimensions. Beam dimensions required are: -

i. Cover to the reinforcement
ii. Breadth (b)
iii. Effective depth (d)
iv. Overall depth (h)

7.3.1 Design for cover

The recommended procedure for the determination of the nominal concrete cover is the same as the procedure used for slab in chapter 2 sections 2.2.2 of this document. The only difference is that, nominal concrete cover for beam sections is designed for both the longitudinal reinforcement and the shear reinforcement (stirrup). Then the governing nominal concrete cover is the minimum nominal concrete cover found by comparing the longitudinal and shear reinforcement (stirrup) nominal concrete cover.

At it has been described in chapter 2 section 2.2.2 of this document, the nominal concrete cover is designed to meet requirements of durability, bond and fire resistance according to ES EN 1992-1.1:2015 section 4.4.1 and ES EN 1992-1.2:2015 section 5.6, therefore the nominal concrete cover for the beam is designed for a design service life of 50 years, normal quality control, maximum aggregate size of 20mm, 1HR fire resistance and exposure class of XC1 (dry or permanently wet) as follows.

$$Cmin = max\begin{cases} Cmin, bond \\ Cmin, dur \\ 10mm \end{cases}$$

$\mathrm{Cmin, bond}$ For the longitudinal reinforcement and shear reinforcement is equal to the diameter of the longitudinal reinforcement and the shear reinforcement respectively. The longitudinal and shear reinforcement we used in this project are Ø16mm and Ø8mm respectively. Therefore $\mathrm{Cmin, bond}$ for the longitudinal reinforcement and shear reinforcement is 16mm and 8mm.

$\mathrm{Cmin, dur}$, according to ES EN 1992-1.1:2015 table 4.4N, for a structural class of four and exposure XC1 (dry or permanently wet). The minimum concrete cover is 15mm. This applied for both longitudinal and shears reinforcements.

$$\text{Longitudinal reinforcement, } Cmin = max\begin{cases} 16mm \\ 15mm \\ 10mm \end{cases}$$

$Cmin = 16mm$, allowing for in design deviation, Δc,dev=10mm

The nominal concrete cover, $Cnom = \ cmin + \ \Delta cdev \ = 16mm + 10mm = 26mm$

$$\text{Shear reinforcement, } Cmin = max\begin{cases} 8mm \\ 15mm \\ 10mm \end{cases}$$

$Cmin = 15mm$, allowing for in design deviation, Δc,dev=10mm

The nominal concrete cover, $Cnom = \ cmin + \ \Delta cdev \ = 15mm + 10mm = 25mm$ It can be seen from the above calculation that the nominal concrete cover for the shear reinforcement governs. Therefor the provided nominal cover for our beam is. $nom \ = 25mm$.

7.3.1.1 Check fire resistance

According to ES EN 1992-1.2:2015 table 5.6 for standard fire resistance of R60, the recommended $b_{min} = 200mm$ and a (nominal cover) $= 12mm$. therefore the nominal concrete cover, $C_{nom} = 25mm$ provided is also satisfactory for R60 fire resistance.

7.3.2 Depth and width

As it has been mentioned in section 2.2.3 of this document, the minimum depth of the slab should satisfy for the serviceability requirement in accordance with ES EN 1992-1-1:2015 section 7.4. The deformation of a beam member should be in way that does not affect its appearance and functionality. This can be checked by limiting the span/depth ratio, according to the ES EN 1992-1-1:2015 expression (7.16a) on the Table 7-1 below.

Table 7-1 Effective depth calculation

Beam span	l(mm)	ρ(slightly stressed)	ρ0 ($\sqrt{fck} * 10^{-3}$)	k	Check	l/d	d (mm)
AB	3760	0.5%	0.5%	1.3	Ok	30	125
BC	4080	0.5%	0.5%	1.5	Ok	35	116
CD	3760	0.5%	0.5%	1.3	Ok	20	125

Required depth (H) = 125mm+16mm/2+25mm+8mm = 166mm

Provided depth (H) = 550mm > 166mm……………… ok

Required minimum width for fire resistance (W) = 200mm

Provided width (W) = 300mm > 200mm……………………………. ok

7.3.3 Analysis of beam section (bending moment and shear force)

Based on the modeling and analysis of the building on ETAB'S 2016 V16.1.0 for a total of 130 load combinations, the maximum bending moment and shear force determined are shown below for a beam on axis 2 of 3rd floor beams.

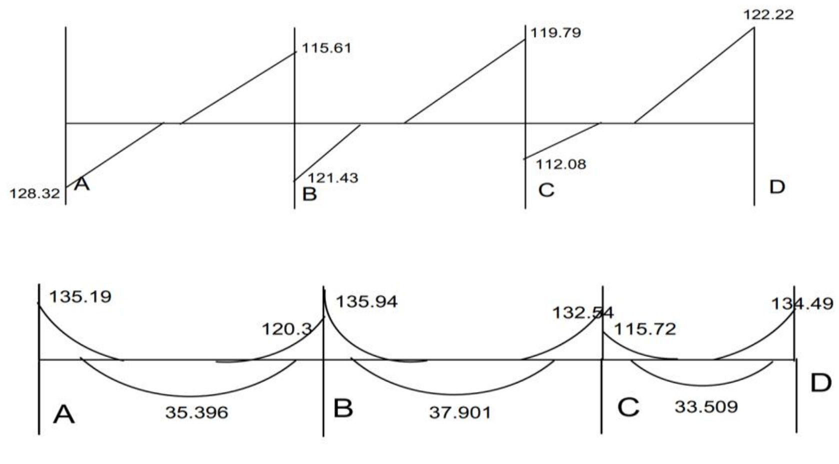


Figure 7-2 Shear force and bending moment diagram for beam on axis 2

7.4 Design of beam section for ultimate limit state

During construction operation of beams, concrete is placed in the beams and slabs in a monolithic pour. As a result, the slab serves as the top flange of the beams, as indicated by the shading in Figure 7-3. Such a beam is can be either a T beam or an inverted L beam depending whether the beam is an interior beam which has flange on both sides or an exterior beam which has flange on one side only.

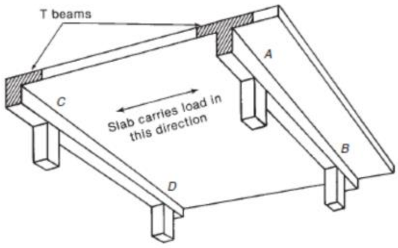


Figure 7-3 T beams and inverted L beams

7.4.1 Effective width of flange

The typical beam section on axis 2 of 3rd floor beams is an interior T beam which means that it will have flanges on both sides. In T beams the effective flange width, over which uniform conditions of stress can be assumed, depends on the web and flange dimensions, the type of loading, the span, the support conditions and traverse reinforcement (ES EN 1992-1.2:2015 section 5.3.2.1(1) P.

According to ES EN 1992-1.1:2015 section 5.3.2.1(1) P, the effective width of a flange should be based on the distance l_o between points of zero moments, which are obtained from ES EN 1992-1.1:2015 figure 5.2.

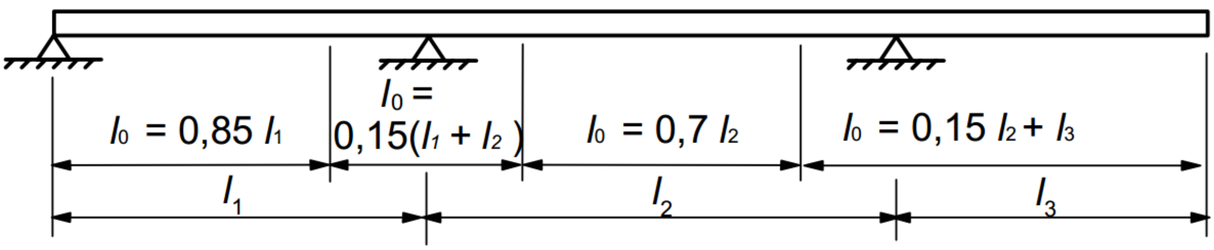


Figure 7-4 Definition of l, for calculation of flange width

The effective flange width with b_{eff} for a T beam is driven by ES EN 1992-1.1:2015 expression 5.7: -

$$b_{eff} = \sum b_{eff}, i + b_w \leq b$$

Where

$b_{eff}, i = 0.2b_i + 0.1l_o \leq 0.2l_o$ *and*

$b_{eff}, i \leq b_i$

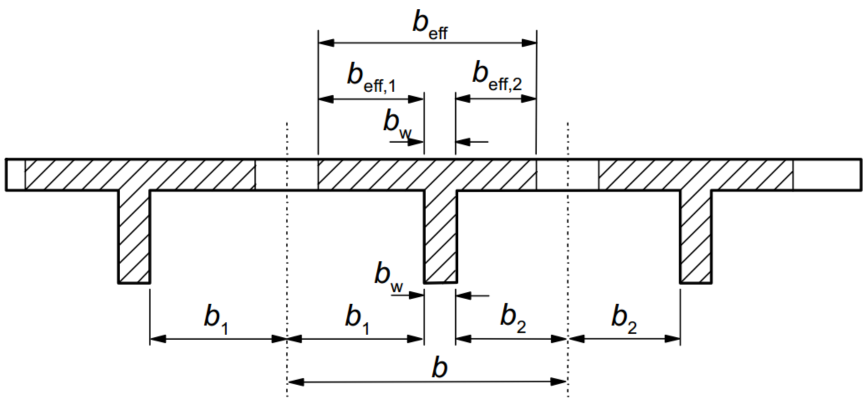


Figure 7-5 Effective flange width parameters

$l_o = 0.85l_1 = 0.85 * 3.76 = 3.196m$ for span AB and CD

$l_o = 0.15(l_1 + l_2) = 0.15 * (3.76 + 4.08) = 1.176m$ for support B and C

$l_o = 0.7l_2 = 0.7 * 4.08 = 2.856m$ for span BC

For span AB and CD

$$b_{eff}, 1 = 0.2\left(\frac{5.9 - 0.3}{2}\right) + 0.1 * 3.196 \leq 0.2 * 3.196m$$

$b_{eff}, 1 = 0.8796m \leq 0.6392m$ not ok, then

$b_{eff}, 1 = 0.6392m, but\ b_{eff}, 1 \leq b_1 = 2.8m$ Ok, then

$b_{eff}, 1 = 0.6392m = 638.2mm$

$$b_{eff}, 2 = 0.2\left(\frac{6.3 - 0.3}{2}\right) + 0.1 * 3.196 \leq 0.2 * 3.196m$$

$b_{eff}, 2 = 0.9196 \leq 0.6392m$ Not ok, then

$b_{eff}, 2 = 0.6392m, but\ b_{eff}, 2 \leq b_2 = 3m\ Ok, then$

$b_{eff}, 2 = 0.6392m = 638.2mm$

$b_{eff} = b_{eff}, 1 + b_{eff}, 2 + b_w \leq b$

$b_{eff} = 638.2mm + 638.2mm + 300mm \leq 2800mm + 3000mm + 300mm$

$b_{eff} = 1580mm \leq 6100mm$ Ok, then

$b_{eff} = 1580mm$

For span BC and supports B and C the same procedure is applied to determine the effective flange width. As result the effective flange width for span BC is, $\mathrm{b_{eff}} = 1440\mathrm{mm}$ and for the supports B and C, $\mathrm{b_{eff}} = 700\mathrm{mm}$.

7.4.2 Design for flexure

At the ultimate limit state, it is important that member sections in flexure should be ductile and that failure should occur with the gradual yielding of the tension steel and not by a sudden catastrophic compression failure of the concrete. Also, yielding of the reinforcement enables the formation of plastic hinges so that redistribution of maximum moments can occur, resulting in a safer and more economical structure. To ensure that a beam member is ductile enough, the ratio of the neutral axis to effective depth (x/d) should not be greater than 0.45m for concrete grades C50 or below. If the ratio of neutral axis to effective depth (x/d) greater than 0.45, then the member should be resized by increasing depth or width of the member and also be providing a compression reinforcement in the case of beams in addition to the tensile reinforcement.

For T beams and inverted L beams, when the beams are resisting sagging moments, the slab acts as a compression flange and the members may be designed as T or L beams or as rectangular beams depending on the position of the neutral axis. If the neutral axis is within the flange depth, then the T or L beam is designed as rectangular beam. But if the neutral axis is beyond the flange width then they are designed as T or L beams using rectangular stress-strain distribution curve.

Support moment A on axis 2

Design moment, $\mathrm{Msd} = 135.19\ \mathrm{KNm}$

Tensile reinforcement bar, $\emptyset = 16\mathrm{mm}$

Reinforcement bar for compression, $\emptyset = 14\mathrm{mm}$

Effective depth, $\mathrm{d} = 550\mathrm{mm} - 25\mathrm{mm} - 8\mathrm{mm} - 16/2\mathrm{mm} = 509\mathrm{mm}$

Effective width, $\mathrm{b_{eff}} = 1580\mathrm{mm}$

Effective web, b_w = 300mm

Rectangular beam section is considered because it is at the support. But according to ES EN 1992-1.1:2015 section 9.2.1.2(2) an intermediate supports of continuous beams, the total area of tension reinforcement As1 can be spread over the effective width of the flange and part of it may be placed on the web. But this may not be necessary if the width of the web is enough to place the tensile reinforcement with adequate spacing.

$$\mu sd = Msd/(fcd * b * d^2).$$

$\mu sd = (135.19 * 10\text{^}6Nmm)/(14.167N/mm2 * 300mm * (509mm)^2).$

$\mu sd = 0.054 \leq 0.295$

This indicates that the section is designed as singly reinforced beam.

$Kz = 0.932$

$z = 0.932d = 0.919$*509mm = 474.388mm

$As1 = \frac{Msd}{fyd*Z}$

$As1 = 135.19 * 10^6 * Nmm/(347.83 \frac{N}{mm^2} * 474.38mm) = 819.3\text{mm}^2$

Check for maximum and minimum reinforcement limits

$$\text{Minimum reinforcement, } As, min = max \begin{cases} \frac{0.26*fctm*bt*d}{fyk} \\ 0.0013bt * d \end{cases}$$

$$A_{s,min} = max \begin{cases} \dfrac{0.26 * 2.6 \frac{N}{mm^2} * 300mm * 509mm}{400 \frac{N}{mm^2}} = 258.063mm^2 \\ 0.0013 * 300mm * 509mm = 198.51mm^2 \end{cases}$$

$A_{s,min}$ = 258.063 mm^2 < As1 = 819.3mm^2.. ok

$A_{s,max} = 0.04A_C$ = 0.04*300mm*550mm

$A_{s,max} = 6600mm^2 > As1$ = 819.3mm^2... ok

Therefore $As1$ = 819.3mm^2

Number of bars in tension, $n = \frac{As1}{ast} = \frac{819.3}{\pi*16^2/4} = 4.1$

Provide 5Ø16 tensile reinforcement

Check for longitudinal reinforcement spacing

According to ES EN 1992-1.1:2015 section 8.2(1) P, the spacing of bars shall be in such that concrete can be placed and compacted satisfactory for the development of adequate bond The clear distance between individual parallel or horizontal layers of parallel bars should not be less than the maximum of k_1 *bar diameter, (dg +k_2) or 20mm where dg is the maximum size of aggregate (in our case 20mm). The recommended value of k_1 and k_2 are 1 and 5mm respectively.

Spacing required = max (1*16mm, 20mm+5mm, 20mm)

Spacing required = max (16mm, 25mm, 20mm)

Spacing required = 25mm

Spacing available = (300mm-2*25mm-2*8mm-5*16mm)/4

Spacing available =40mm > Spacing required = 25mm……………………………………ok

Span moment AB on axis 2

Design moment, Msd = 35.9011 KNm

Tensile reinforcement bar, Ø = 16mm

Reinforcement bar for compression, Ø = 14mm

Effective depth, d = 550mm – 25mm – 8mm – 16/2mm = 509mm

Effective width, b_{eff} = 1440mm

Effective web, b_w = 300mm

Since the beam at this span is resisting a sagging bending moment, the first thing to do is to check whether the beam should be designed as rectangular or T beam. To do this we have to determine the neutral axis using a simplified stress-strain distribution called rectangular stress-strain distribution curve as show in

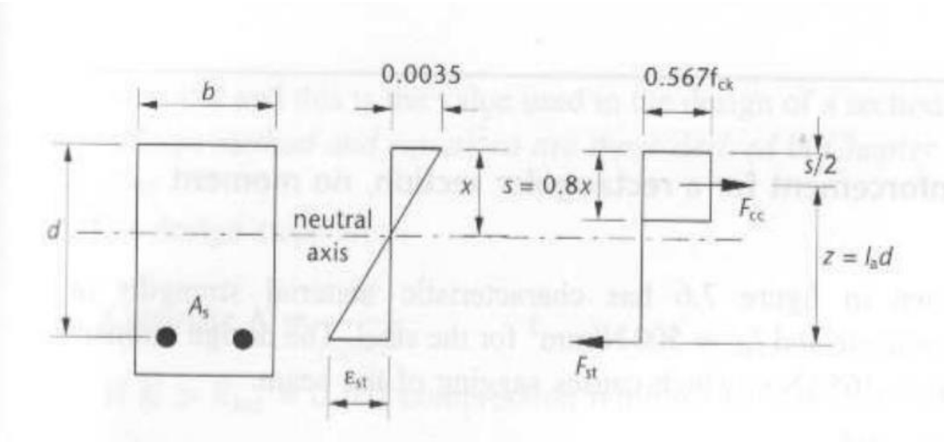


Figure 7-6 Rectangular stress-strain block

$Msd = 0.8x * f_{cd} * b_{eff} * (d - 0.4x)$

$37.9011 * 106$ Nmm $= 0.8x * 11.33$ N/〖mm〗^2 $* 1440mm * (509mm - 0.4x)$

$2.9 * 103 = 509x - 0.4x2$

$x = 5.72mm < tf = 170mm$, the beam is designed as a rectangular beam

$\mu sd = Msd/(fcd * b * d^2)$.

$\mu sd = (37.9011 * 10^6Nmm)/(14.167N/mm2 * 300mm * (509mm)^2)$.

$\mu sd = 0.034 \leq 0.295$

This indicates that the section is designed as singly reinforced beam.

$Kz = 0.978$

$z = 0.978d = 0.978 * 509mm = 498.01mm$

As1 = 37.9011 * 〖10〗^6 * Nmm/(347.83 N/(mm^2) * 498.01mm)
= 219.2mm2

Check for maximum and minimum reinforcement limits

A_(s, min) = 258.063 mm^2 > As1 = 219.2mm2……………………………….not ok, therefore As1 = As, min = 258.063 mm^2

$A_{s,max} = 0.04A_C = 0.04*300mm*550mm$

$A_{s,max} = 6600mm^2 > As1 = 258.063\ mm^2$ ……………………………………..ok

Therefore $As1 = 258.063\ mm^2$

Number of bars in tension, N $=\frac{\text{As1}}{\text{ast}}=\frac{258.063}{\pi*16^2/4}=1.28$

Provide 2Ø16 tensile reinforcement

Check for longitudinal reinforcement spacing

Spacing required = 25mm

Spacing available = (300mm-2*25mm-2*8mm-2*16mm)/1

Spacing available =202mm > Spacing required = 25mm……………………………. ok

Table 7-2 Summary design for flexure of beam on axis 2

Beam	position	Msd,KNm	d,mm	Design type	µsd	Reinforcement type	Kz	Z,mm	As1,mm^2	As,min, mm^2	check	As1,mm^2	N =As1/as	spacing required	spacing available	check	Tensile reinforcement
	span moment AB	35.396	509	Rectangular	0.032	Singly reinforced	0.979	498.41	204.17	258.063	Not ok	258.063	2	25	202	OK	2Ø16
	span moment BC	37.9011	509	Rectangular	0.034	Singly reinforced	0.978	498.0.1	219.2	258.063	Not ok	258.063	2	25	202	OK	2Ø16
on	span moment CD	33.509	509	Rectangular	0.03	Singly reinforced	0.98	498.82	193.13	258.063	Not ok	258.063	2	25	202	OK	2Ø16
Axis 2	support moment A	135.19	509	Rectangular	0.122	Singly reinforced	0.932	474.388	819.3	258.063	OK	819.3	5	25	40	OK	5Ø16
	support moment B	135.94	509	Rectangular	0.123	Singly reinforced	0.932	474.489	823.67	258.063	OK	823.67	5	25	40	OK	5Ø16
	support moment C	132.54	509	Rectangular	0.12	Singly reinforced	0.934	475.41	801.51	258.063	OK	801.51	5	25	57	OK	4Ø16
	support moment D	134.49	509	Rectangular	0.122	Singly reinforced	0.933	474.75	814.43	258.063	OK	814.43	5	25	40	OK	5Ø16

7.4.3 Design for shear

When a beam is uniformly loaded the distribution of principal stress across the span of a homogeneous concrete beam is as shown in Figure 7-7. The direction of the principal compressive stresses takes the form of an arch, while the tensile stresses have the curve of catenary or suspended chain. Towards the mid-span, where the shear is low and the bending stresses are dominant, the direction of the stressed tends to be parallel to the beam axis. Near the supports, where the shearing forces are greater, the principal stresses become inclined and the greater the shear force the greater angle of inclination. The tensile stresses due to shear are liable to cause diagonal cracking of the concrete near to the support so that shear reinforcement must be provided. This reinforcement is either in the form of stirrups, or inclined bars (used in conjunction with stirrups)

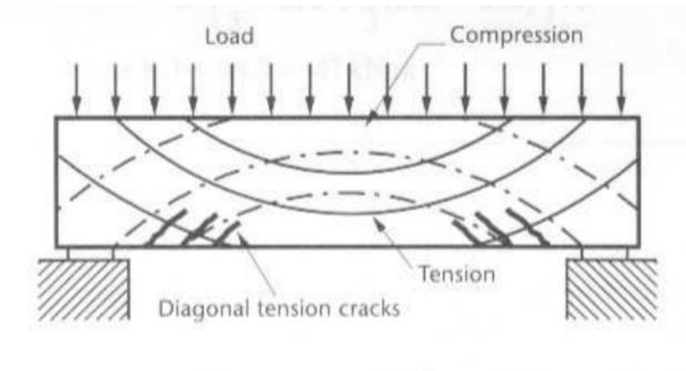


Figure 7-7 Principal stresses in beam

The concrete itself can resist shear by combination of the un-cracked concrete in the compression zone, the dowelling action of the bending reinforcement and aggregate interlock across tension cracks but, because concrete is weak in tension, the shear reinforcement is designed to resist all the tensile stresses caused by the shear forces. Even where the shear forces are small near the center of span of abeam a minimum amount of shear reinforcement in the form of links must be provided in order to form a cage supporting the longitudinal reinforcement and to resist any tensile stress due to factors such as thermal movement and shrinkage of concrete.

7.4.3.1 Members that don't require shear reinforcement

Beams are generally heavily loaded and have a smaller cross-section do that they nearly are always require shear reinforcement. Even lightly loaded beams are required to have a minimum amount of shear links. According to ES EN 1992-1.1:2015 section 6.2.1(3), in regions of the members where designs shear force $V_{ED} \leq V_{RD,}$ c (design shear resistance of a member without shear reinforcement) no calculated shear reinforcement is necessary Nevertheless, when no shear reinforcement is required, minimum shear reinforcement should nevertheless be provided according to ES EN 1992-1.1:2015 section 9.2.2.

7.4.3.2 Members that require shear reinforcement

According to ES EN 1992-1.1:2015 section 6.2.1(3), in regions of the members where design shear force. $V_{ED} > V_{RD,}$ c (design shear resistance of a member without shear reinforcement) sufficient calculated shear reinforcement is necessary so that $V_{ED} \leq V_{RD}$ (design shear resistance of a member with shear reinforcement). For beam members requiring design shear reinforcement, the design of the beam members with shear reinforcement is based on a truss model shown in (ES EN 1992-1.1:2015 section 6.2.3(1)) figure 6.5, which is shown in Figure 7-8 in this document. In Figure 6.5 the following notations are shown:

α is the angle between shear reinforcement and the main tension chord (measured positive as shown);

td is the design value of the tensile force in the longitudinal reinforcement;

θ is the angle between concrete compression struts and the main tension chord F;

F_{cd} is the design value of the concrete compression force in the direction of the longitudinal member axis;

b_w is the minimum width between tension and compression chords; and

z is the inner lever arm, for a member with constant depth, corresponding to the maximum bending moment in the element under consideration. In the shear analysis, the approximate value z = 0,9d may normally be used

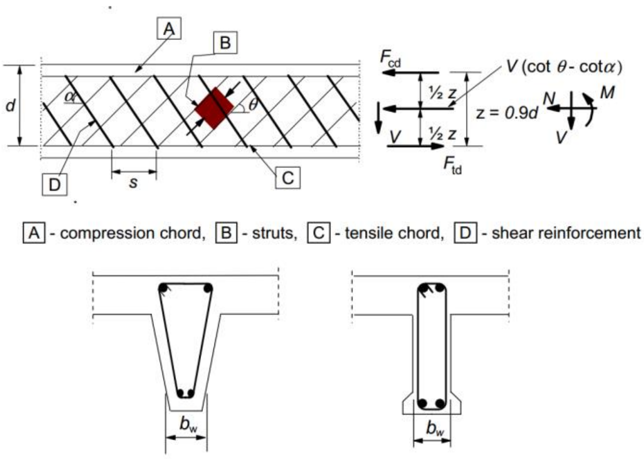


Figure 7-8 Truss model and notation for shear reinforced members

The angle θ increases with the magnitude of the maximum shear force on the beam and hence the compressive forces in the diagonal concrete members. But in ES EN 1992-1.1:2015 section, the angle θ is limited to have a value between 21.8 and 45 degrees. For most cases of predominantly uniformly distributed loading the angle θ will be 21.8 degrees but for heavy and concentrated loads it can be higher in order to resist crushing of concrete diagonal members.

Shear design of span AB axis 2

As mentioned in section 3.2.4 of this document, the design shear force for members predominantly subjected to distribute loading should be determined at a distance d from the face of the support. But for simplicity and due to shear force diagram in Figure 7-2 being as a result of envelop combination, the shear force at the center of the supports has been taken as the design shear force.

V_{ED} = 128.32KN

COMPRESSION CAPACITY OF COMPRESSION STRUT, V_{RD}, max, taking θ = 21.8 degrees

According to ES EN 1992-1.1:2015 section 6.2.1(6), the design shear force, V_{ED}, should not exceed the maximum permitted value, $V_{RD,}$ max (is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts).

$$v_1 = 0.6\left(1-\frac{fck}{250}\right) = 0.6\left(1-\frac{25}{250}\right) = 0.54$$

f_{cd} = 14.167Mpa

$\alpha_{cw} = 1 \text{ for non} - \text{prestressed structures}$

$z = 0.9d = 0.9 * 509mm = 458.1mm$

$b_w = 300mm$

$$V_{Rd,max} = \frac{1 * 300mm * 458.1mm * 0.54 * 14.167N/mm^2}{(2.5 + 0.4)}$$

$V_{Rd,max} = 362.52KN > V_{ED} = 128.32KN$ …………………………………… ok

SHEAR RESISTANCE OF CONCRETE, V_{RD}, c

$$c_{Rd} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12\ N/mm^2$$

$$k = 1 + \sqrt{\frac{200}{509}} = 1.39 \leq 2.0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. ok$$

$$\rho_1 = \frac{A_s}{b_w * d} \leq 0.02$$

$$\rho_1 = \frac{258.063}{300 * 509} = 0.00169 \leq 0.02 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots ok$$

$$\sigma_{cp} = \frac{N_{ed}}{A_c} < 0.2f_{cd} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. because\ N_{ed} = 0$$

$$v_{min} = 0.035 * k^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}}$$

$$v_{min} = 0.035 * 1.39^{\frac{3}{2}} * 25^{0.5}$$

$$v_{min} = 0.1282$$

$$V_{Rd,c} = max\begin{cases}\left[c_{Rd} * k * (100 * \rho_1 * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right] * b_w * d \\ \left(v_{min} + k_1 * \sigma_{cp}\right) * b_w * d\end{cases}$$

$$V_{Rd,c} = max\begin{cases}\left[0.12\frac{N}{mm^2}*1.39*(100*0.00169*25)^{\frac{1}{3}}+1*0\right]*300mm*509mm \\ (0.1282+1*0)*300mm*509mm\end{cases}$$

$$V_{Rd,c} = max\begin{cases}41.17KN \\ 19.576KN\end{cases}$$

$V_{Rd,c} = 41.17\ KN > V_{ED} = 128.32KN,$ therefore shear reinforcement is required

Diameter and spacing of links

Where V_{ED} > $V_{RD,}$ c, shear reinforcement is provided according to ES EN1992-1.1:2015 expression 6.8.

$$V_{RD,s} = V_{ED} = \frac{A_{sw}}{s} * zf_{ywd}cot\ \theta$$

Where

A_{sw} Is the cross-sectional area of the shear reinforcement;

S Is the spacing of stirrups; and

f_{ywd} Is the design yield strength of the shear reinforcement, $347.83\,\frac{N}{mm^2}$

Using a shear reinforcement bar of diameter Ø8mm, A_{sw} =2*π * 4^2 = 100.53mm²

$$V_{ED}\Big/ zf_{ywd}cot\ \theta = \frac{A_{sw}}{s}$$

$$128.32\Big/ 0.9*509mm*\frac{347.83N}{mm^2}*cot\ 21.8 = \frac{A_{sw}}{s}$$

$$0.322 = \frac{A_{sw}}{s}$$

$$s = A_{sw}/0.322$$

$$s = 100.53/0.322$$

$$s = 300mm$$

According to ES EN 1992-1.1:2015 section 9.2.2(6), the maximum spacing between vertical shear assemblies should not exceed, $S_{l,\ max}$. The recommended value is given by ES EN 1992-1.1:2015 expression 9.6N.

$$S_{max} = 0.75d$$

$S_{max} = 0.75*509mm$

$S_{max} = 380mm > s=300mm$…………………………………. ok

Therefore, provide Ø8 C/C 300mm

Table 7-3 summary shear design

Span	V_{ED} (KN)	$V_{RD,max}$ (KN)	Check	$V_{RD,C}$ (KN)	Check	S (mm)	S_{max} (mm)	check	Provide
AB	128.32	365.32	Ok	41.7	Requires shear reinforcement	300	380	ok	Ø8C/C300mm
BC	121.43	365.32	Ok	41.7	Requires shear reinforcement	330	380	ok	Ø8C/C330mm
CD	122.22	365.32	Ok	41.7	Requires shear reinforcement	330	380	ok	Ø8 C/C330mm

8 Column Design

A column is a vertical structural member supporting axial compressive loads, with or without moment. The cross-section dimensions of a column are generally considerably less than its height. A column supports vertical loads from the floor and roof and transmits these loads to the foundation.

8.1 Classification of columns

8.1.1 Based on lateral reinforcement

- 'Tied columns' in which the main longitudinal bars are confined within closely spaced lateral ties.
- 'Spiral columns' having main longitudinal reinforcements enclosed within closely spaced and continuously wound spiral reinforcement.

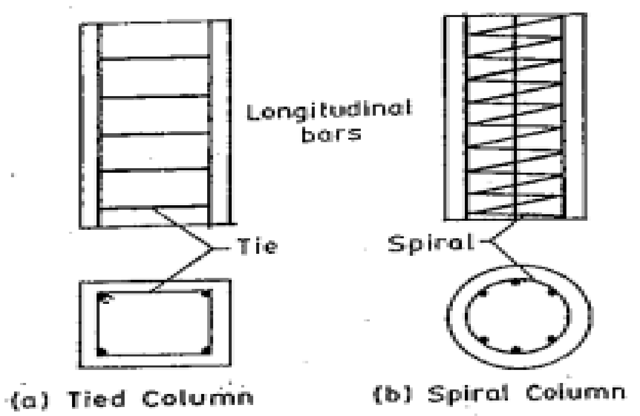


Figure 8-1 Tied columns (a) and spiral columns (b)

8.1.2 Based on type of loading

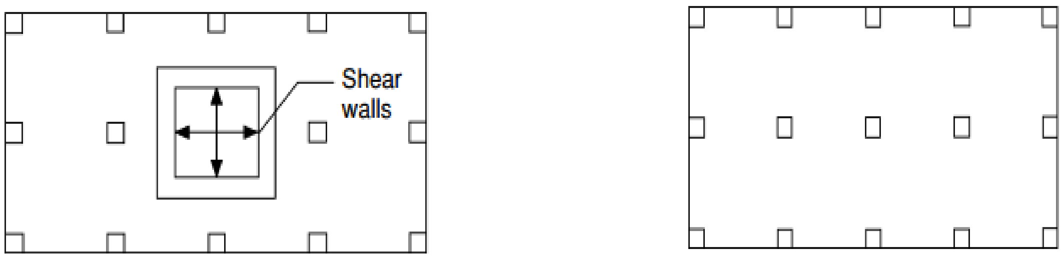
- Axially loaded columns: are columns which are not exposed to any moment in any direction that is the only action will be the axial load.
- Uniaxial columns: are columns which are highly exposed to bending moment in one of the direction rather than both directions in addition to the axial load.
- Biaxial columns: are columns which are subjected to moment in both of direction besides the axial load.

8.1.3 Based on degree of slenderness

- Short column: are columns for which the strength is governed by the strength the materials and the geometry of the cross section.in short columns, second-order effects are negligible.
- Slender column: when the unsupported length is long lateral deflection shall be so high that the moments shall increase and weaken the column. Such a column, whose axial load carrying capacity is significantly reduced by moment resulting from lateral deflection of the column, is referred to as slender column.

8.2 Braced and unbraced columns

A column may be considered braced if the lateral loads, due to wind for example, are resisted by shear walls or some other form of bracing rather than by the column. A column may be considered to be unbraced if the lateral loads are resisted by the sway action of the column.



(a) (b)

Figure 8-2 (a) Braced columns (b) Unbraced columns

8.3 Second order effects on columns

According to ES EN 1992-1-1:2015 section 5.8 second order effects are additional action caused by structural deformation Second order effects may be ignored if they are less than 10% of the corresponding first order or satisfice the following criteria.

8.3.1 Simplified criteria for second order effects

8.3.1.1 Slenderness criteria

According to EN ES 1992-1-1:2015 section 5.8.3.1 second order effects may be ignored if the slenderness λ is below a certain λ_{lim} value

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

Where

A	${}^{1}/_{(1 + 0.2\varphi_{ef})}$ (if φ_{ef} is not known ,A =0.7 may be used),
B	$\sqrt{1 + 2\omega}$ (if ω is not known, B =1.1 may be used),
C	$1.7 - r_m$ (if r_m is not known,C =0.7 may be used),
φ_{ef}	Effective creep ratio ,
ω	Mechanical reinforcement ratio,
n	${}^{N_{Ed}}/_{(A_c * f_{cd})}$; relative normal force ,
r_m	${}^{M_{01}}/_{M_{02}}$;moment ratio ,and

M_{01}, M_{02} Are the first order of moments,$|M_{02}| \geq |M_{01}|$

In cases with biaxial bending, the slenderness criterion may be checked separately for each direction. Depending on the outcome of this check, second order effects (a) may be ignored in both directions, (b) should be taken into account in one direction, or (c) should be taken into account in both directions.

8.3.1.2 Slenderness and effective length

The slenderness ratio is defined as follows:

$$\lambda = {}^{l_0}/_{i}$$

Where

l_0 is the effective length, and

i is the radius of gyration of the un cracked concrete section

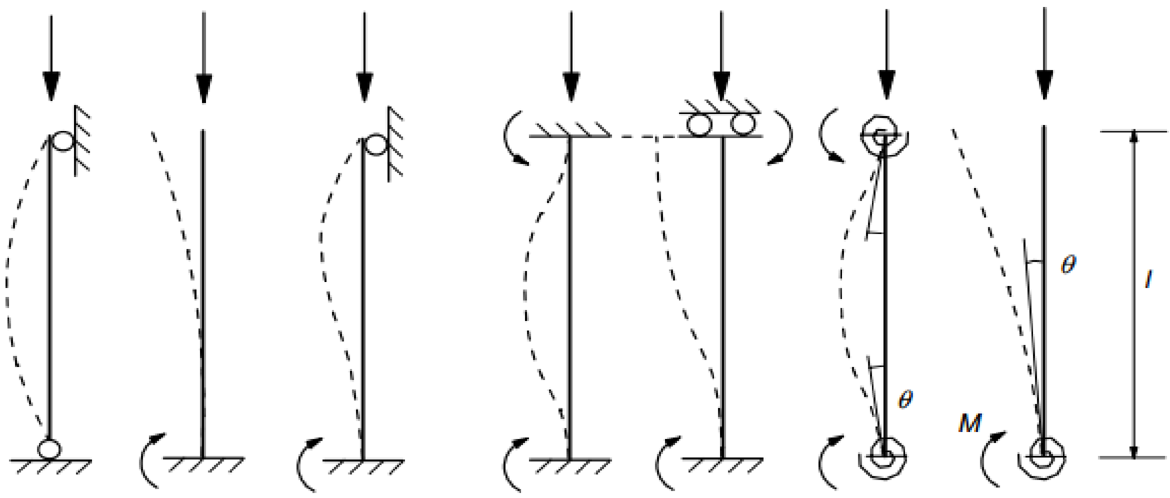


Figure 8-3 Examples of different buckling modes and corresponding effective

For compression members in regular frames, the slenderness criterion should be checked with an effective length l_0 determined in the following way.

Braced members

$$l_0 = 0.5l * \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) * \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

Un Braced members

$$l_0 = l * max \begin{cases} \sqrt{1 + 10 * \dfrac{k_1 * k_2}{k_1 + k_2}} \\ \left(1 + \dfrac{k_1}{1 + k_1}\right) * \left(1 + \dfrac{k_2}{1 + k_2}\right) \end{cases}$$

Where

k_1, k_2 are the relative flexibilities of rotational restraints at ends 1 and 2 respectively,

k $\left({}^{\theta}/_{M}\right) * \left({}^{EI}/_{l}\right)$,

θ is the rotation of restraining members for bending moment M,

EI is the bending stiffness of compression members, and

l is the clear height of compression member between end restraints.

Note: $k = 0$ is the theoretical limit for rigid rotational restraint, and $k = \infty$ represents the limit for no restraint at all. Since fully rigid restraint is rare in practice, a minimum value of 0.1 is recommended for k_1 and k_2.

Method of analysis

If $\lambda > \lambda_{lim}$ the second order effect must be analyzed using the following method.

a. Second order analysis based on nominal stiffness, and

b. Method based on estimation of curvature.

8.3.2 Design for cover

The recommended procedure for the determination of the nominal concrete cover is the same as the procedure used for slab in chapter 2 sections 2.2.2 of this document and for beam in chapter 7 sections 7.2.1.

$Cmin, dur$, according to ES EN 1992-1.1:2015 table 4.4N, for a structural class of four and exposure XC1 (dry or permanently wet). The minimum concrete cover is 15mm. this applied for both longitudinal and shears reinforcements.

$$\text{Longitudinal reinforcement, } Cmin = \max\begin{cases} 20mm \\ 15mm \\ 10mm \end{cases}$$

$Cmin = 20mm$, allowing for in design deviation, $\Delta c,dev=10mm$

The nominal concrete cover, $Cnom = cmin + \Delta cdev = 20mm + 10mm = 30mm$ Shear

$$\text{reinforcement, } Cmin = \max\begin{cases} 8mm \\ 15mm \\ 10mm \end{cases}$$

$Cmin = 15mm$, allowing for in design deviation, $\Delta c,dev=10mm$

The nominal concrete cover, $Cnom = cmin + \Delta cdev = 15mm + 10mm = 25mm$

It can be seen from the above calculation that the nominal concrete cover for the shear reinforcement governs. Therefor the provided nominal cover for our beam is , $Cnom = 25mm$.

8.3.2.1 Check for fire resistance

According to ES EN 1992-1.2:2015 table 5.2(a) for standard fire resistance of R60, the recommended Column width b_{min} /axis distance a of the main bars for Column exposed on more than one side and $\mu_{fi} = 0.7$ are $b_{min} = 350$mm and a $= 40$mm. therefore the nominal concrete cover, $C_{nom} = 25$mm provided is also satisfactory for R60 fire resistance. And also the minimum column section of our project is 400mm*400mm, this complies with the minimum width of column required for R60 fire resistance.

Where

μ_{fi} is a reduction factor for the design load level in the fire situation

8.4 Longitudinal reinforcement

The general procedure followed to calculate longitudinal reinforcement is:

i. calculate first order moment
ii. calculate the effective length and radius of gyration
iii. calculate the slenderness ratio and slenderness limit and check for second order effect
iv. calculate the accidental eccentricity
v. calculate equivalent first order moment
vi. calculate equivalent first order moment with addition of accidental eccentricity moment
vii. If slenderness limit is greater than slenderness ratio calculates the longitudinal reinforcement using the moment and in 6 if not use one of the two second order analysis to calculate the additional moment due to second order effect and add to the moment in 6 to calculate longitudinal reinforcement.
viii. Calculate $A_s$using moment in 7 and axial force.
ix. Check for $A_{s,min}$ and $A_{s,max}$.

8.4.1 First order moment

First order moment are moments coming from ETABS analysis of the building and those moments are due to the load effect of the structure

Table 8-1 shows the result of first order moment form ETABS for column 12.

Table 8-1 First order moment and axial force from ENVX for column 12

Column	Moment	$M_3 = M_y$(kNm)	$M_2 = M_x$(kNm)	N_{Ed} (kN)
Ground floor	M_{02}	-229.238	95.158	
	M_{01}	-103.78	48.5879	5678.59
1st floor	M_{02}	194.7	-14.63	
	M_{01}	2.02	-93.0167	5139.6
2nd floor	M_{02}	170.8838	-79.95	
	M_{01}	-55.9925	17.2021	4595.32
3rd floor	M_{02}	160.0693	-67.4438	
	M_{01}	-82.2931	31.4286	4057.32
4th floor	M_{02}	148.0836	-62.0338	
	M_{01}	-108.0431	30.7124	3530.38
5th floor	M_{02}	103.8225	-43.8578	
	M_{01}	-83.84	36.4148	3005.31
6th floor	M_{02}	114.5386	-49.25	
	M_{01}	-83.0651	36.62	2501.1962
7th floor	M_{02}	97.85	-42.77	
	M_{01}	-74.02	32.3149	2004.04
8th floor	M_{02}	93.56	-43.88	
	M_{01}	-82.616	41.19	1514.3903
9th floor	M_{02}	41.95	-18.78	
	M_{01}	-34.31	16.78	1020.6405
10th floor	M_{02}	43.37	10.7	
	M_{01}	-29.9169	-7.161	527.5974
Roof	M_{02}	32.19	-21.75	
	M_{01}	22.20	-5.40	35.09

8.4.2 Effective length and radius of gyration

8.4.2.1 Effective length

The effective length, l_0 of a member is defined as the length of a pin-ended strut with constant normal force having the same cross-section and buckling load.

Sample calculation of effective length for ground floor column

In the x-direction

For un Braced members

$$l_0 = 0.5l * \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) * \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

Where, $\mathrm{l} = 1.95\mathrm{m}$, $\mathrm{k}_1 = 0.1$ because it is rigid and k_2 is calculated as follow

$$k = \frac{\text{Column stiffness}}{\sum 2 * \text{beam stiffness}} = \frac{\left(\frac{EI}{l}\right)\text{column}}{\sum(2 * \frac{EI}{l})\text{beam}}$$

$$I_{column} = \frac{800*800^3}{12} = 3.4 * 10^{10}mm^4$$

$$I_{beam} = \frac{300*550^3}{12} = 0.42 * 10^{10}mm^4$$

$$k_2 = \frac{\left(\frac{E*3.4*10^{10}mm^4}{1950mm}\right)}{\left(2*\frac{E*0.42*10^{10}mm^4}{3760mm}\right)+\left(2*\frac{E*0.42*10^{10}mm^4}{4080mm}\right)} = 4.056$$

$$l_0 = 1711mm$$

In the y direction

$$l_0 = 0.5l * \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) * \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

Where, $l = 1.95m$, $k_1 = 0.1$ because it is rigid and k_2 is calculated as follow

$$k = \frac{\text{Column stiffness}}{\sum 2*\text{beam stiffness}} = \frac{\left(\frac{EI}{l}\right)\text{column}}{\sum(2*\frac{EI}{l})\text{beam}}$$

$$I_{column} = \frac{800*800^3}{12} = 3.4 * 10^{10}mm^4$$

$$I_{beam} = \frac{300*550^3}{12} = 0.42 * 10^{10}mm^4$$

$$k_2 = \frac{\left(\frac{E*3.4*10^{10}mm^4}{1950mm}\right)}{\left(2*\frac{E*0.42*10^{10}mm^4}{5900mm}\right)+\left(2*\frac{E*0.42*10^{10}mm^4}{6300mm}\right)} = 6.77$$

$$l_0 = 1721mm$$

8.4.2.2 Radius of gyration

$$i = \sqrt{I_{column}/A_{c,column}} = \sqrt{3.4 * 10^{10}mm^4/800 * 800mm^2} = 230.94mm$$

Table 8-2 effective length and radius of gyration of c-12

column	k_1		k_2		l_0(mm)		i(mm)
	X	Y	x	Y	X	Y	
Ground	0.1	0.1	4.117352	6.410896	1712.032	1721.152	2.31E+02
1st floor	3.029749	4.717452	3.029749	4.717452	2562.894	2591.665	2.31E+02
2nd floor	3.029749	4.717452	3.029749	4.717452	2562.894	2591.665	2.31E+02
3rd floor	1.775983	4.717452	1.775983	2.765284	2512.503	2573.694	2.02E+02
4th floor	1.775983	2.765284	1.775983	2.765284	2512.503	2555.597	2.02E+02
5th floor	0.958632	2.765284	0.958632	1.492631	2429.156	2523.907	1.73E+02
6th floor	0.958632	1.492631	0.958632	1.492631	2429.156	2491.814	1.73E+02
7th floor	0.958632	1.492631	0.958632	1.492631	2429.156	2491.814	1.73E+02
8th floor	0.958632	1.492631	0.958632	1.492631	2429.156	2491.814	1.73E+02
9th floor	0.189359	1.492631	0.189359	0.294841	2133.349	2356.936	1.15E+02
10th floor	0.189359	0.294841	0.189359	0.294841	2133.349	2213.856	1.15E+02
Roof	0.189359	0.294841	0.590712	0.919765	2242.256	2320.574	1.15E+02

8.4.3 Slenderness ratio, slenderness limit and check for second order effect

8.4.3.1 Slenderness limit

Sample calculation for ground floor column

In the x direction

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

Where $A = 0.7$, $B = 1.1$ and $C$and n are calculated as followed

$$C = 1.7 - r_m$$

$$r_m = M_{01}/M_{02} = -103.78kNm/-229.2385kNm = 0.453$$

$$C = 1.7 - r_m = 1.7 - 0.453 = 1.247$$

$$n = N_{Ed}/(A_c * f_{cd}) = 5678.59kN/640000mm^2 * 14.167\frac{N}{mm^2} = 0.626$$

$$\lambda_{lim} = 20 * 0.7 * 1.1 * 1.247/\sqrt{0.626} = 24.27$$

In the y direction

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

Where= 0.7 , $B = 1.1$ and $C$and n are calculated as followed

$$C = 1.7 - r_m$$

$$r_m = {M_{01}}/{M_{02}} = {48.5879kNm}/{95.1568kNm} = 0.51$$

$$C = 1.7 - r_m = 1.7 - 0.51 = 1.19$$

$$n = {N_{Ed}}/{(A_c * f_{cd})} = {5678.59kN}/{640000mm^2 * 14.167 \frac{N}{mm^2}} = 0.626$$

$$\lambda_{lim} = {20 * 0.7 * 1.1 * 1.19}/{\sqrt{0.626}} = 23.162$$

8.4.3.2 Slenderness ratio

Sample calculation for ground floor column

In the x direction

$$\lambda = {l_0}/{i} = {1711\text{mm}}/{230.94\text{mm}} = 7.409$$

In the y direction

$$\lambda = {l_0}/{i} = {1721\text{mm}}/{230.94\text{mm}} = 7.452$$

Check for second order effect

For both directions

$$\lambda < \lambda_{lim} \ldots\ldots\ldots no\ need\ of\ second\ order\ effect$$

Table 8-3 slenderness ratio, slenderness limit and check for second order effect

column	l_0(mm)		i(mm)	λ		λ_{lim}		remark
	X	Y		X	Y	X	y	
Ground	1712.032	1721.152	2.31E+02	7.41E+00	7.45E+00	24.27168	23.14496	ok
1st	2562.894	2591.665	2.31E+02	1.11E+01	1.12E+01	34.56013	37.98946	ok
2nd	2562.894	2591.665	2.31E+02	1.11E+01	1.12E+01	43.86194	41.42829	ok
3rd	2512.503	2573.694	2.02E+02	1.24E+01	1.27E+01	50.97169	49.86408	ok
4th	2512.503	2555.597	2.02E+02	1.24E+01	1.26E+01	59.96189	54.17408	ok
5th	2429.156	2523.907	1.73E+02	1.40E+01	1.46E+01	67.07358	67.6824	ok
6th	2429.156	2491.814	1.73E+02	1.40E+01	1.44E+01	71.64693	71.64693	ok
7th	2429.156	2491.814	1.73E+02	1.40E+01	1.44E+01	80.46497	80.43505	ok

8th	2429.156	2491.814	1.73E+02	1.40E+01	1.44E+01	97.3329	99.43063	ok
9th	2133.349	2356.936	1.15E+02	1.85E+01	2.04E+01	115.5708	119.042	ok
10th	2133.349	2213.856	1.15E+02	1.85E+01	1.92E+01	152.5672	151.255	ok
Roof	2242.256	2320.574	1.15E+02	1.94E+01	2.01E+01	250.108	359.3702	ok

NB: As we can see in the above Table 8-3 no need of second order effect for our columns.

8.4.4 Accidental eccentricity

Is the effects of cracking, creep, non-linear material properties and geometric imperfections, which normally means considering the structure being constructed 'out of plumb' (not vertical), which in isolated members is allowed for by introducing an additional eccentricity,e_i of the axial load.

$$e_i = \frac{l_0}{400} \text{ And the moment introduced is } M_i = e_i * N_{Ed}$$

Sample calculation for accidental eccentricity moment for ground floor column

In the x direction

$$e_i = \frac{l_0}{400} = \frac{1.711m}{400} = 0.004275m$$

$$M_i = e_i * N_{Ed} = 0.004275m * 5678.59kN = 24.276kNm$$

In the y direction

$$e_i = \frac{l_0}{400} = \frac{1.721m}{400} = 0.0043m$$

$$M_i = e_i * N_{Ed} = 0.0043m * 5678.59kN = 24.43kNm$$

8.4.5 Equivalent first order moment

Classic analyses of buckling commonly consider the deformation of a pinned ended strut, but this is not the normal configuration of a column in a building. A normal column built monolithically into a structure at its top and bottom will deform, and be subjected to moments like those shown It will be seen that the section of the column between the points of contra-flexure in the final state of the column may be considered to be a pinned ended strut equivalent to that for which the analysis was carried out. The distance between the points of contra-flexure is the effective length of the column. The maximum moment due to deflection will be seen to occur at mid-height of the effective column. This will normally be somewhere close to mid-height of the real column. Clearly, the total moment to which the critical section is subjected is made up of the maximum moment due to the deflection plus the first-order moment at this height plus any allowance for accidental effects.

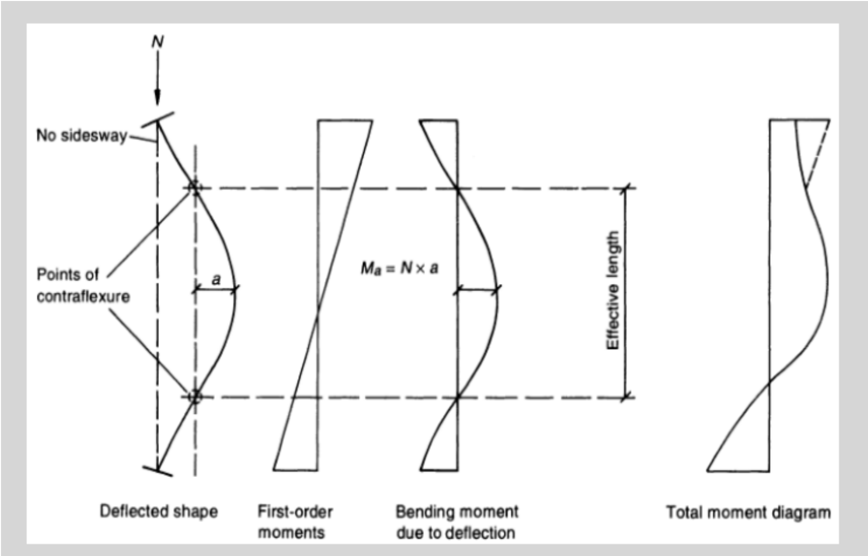


Figure 8-4 Moment and deformation of a braced isolated column

According to ES EN 1992-1-1:2015 section 5.8.8.2 a reasonable estimate of the first-order moment near mid-height of the column is given by

$$M_{oe} = max\begin{cases}0.6 * M_{02} + 0.4 * M_{01} \\ 0.4 * M_{02}\end{cases}$$

But, their no need of calculation of equivalent moment in our building because our columns are short columns. And the design moment is the moment due to first order effects,M_{ED}, being numerically equal to the sum of the larger elastic end moment, M_{02} , plus any moment due to geometric imperfection, M_i .(Chanakya Arya, Concrete, steelwork, masonry and timber designs to British Standards and Euro codes)

According to ES EN 1992-1-1:2015 section 6.1(4) For reinforced concrete cross-sections subjected to a combination of bending moment and compression, the design value of the bending moment should be at least $M_{ED} = e_0 * N_{ED}$ where e_0= h/30 but not less than 20 mm where h is the depth of the section.

$$M_{ED} = max\begin{cases}M_{02} + M_i \\ e_0 * N_{ED}\end{cases}$$

Sample calculation of design moment for ground floor

In the x direction

$$M_{ED} = max \begin{cases} 229.23kNm + 24.276kNm \\ \frac{800mm}{30} * 5678.59kN \end{cases} = max \begin{cases} 253.506kNm \\ 151.43kNm \end{cases} = 253.506kNM$$

In the y direction

$$M_{ED} = max \begin{cases} 95.157kNm + 24.43kNm \\ \frac{800mm}{30} * 5678.59kN \end{cases} = max \begin{cases} 119.587kNm \\ 151.43kNm \end{cases} = 151.43kNM$$

Table 8-4 Design moment and moment accidental eccentricity

column	N_{ED}(kN)	M_{02}(kNm)		M_i(kNm)		$e_0 * N_{ED}$(kNm)	M_{ED}(kNm)	
		X	Y	X	Y		y	x
Ground	5678.59	229.238	95.158	24.30	24.43	151.6184	253.54	151.62
1st	5139.6	194.7	93.017	32.93	33.30	137.2273	227.63	137.23
2nd	4595.32	170.8838	79.95	29.44	29.77	122.695	200.33	122.70
3rd	4057.32	160.0693	67.4438	25.49	26.11	93.31836	185.55	93.55
4th	3530.38	148.0836	62.0338	22.18	22.56	81.19874	170.26	84.59
5th	3005.31	103.8225	43.8578	18.25	18.96	60.1062	122.07	62.82
6th	2501.2	114.5386	49.25	15.19	15.58	50.02392	129.73	64.83
7th	2004.04	97.85	42.77	12.17	12.48	40.0808	110.02	55.25
8th	1514.39	93.56	43.88	9.20	9.43	30.28781	102.76	53.31
9th	1020.64	41.95	18.78	5.44	6.01	20.41281	47.39	24.79
10th	527.597	43.37	10.7	2.81	2.92	10.55195	46.18	13.62
Roof	35.09	32.19	21.75	0.20	0.20	0.7018	32.39	21.95

8.4.6 Calculate $A_s$using M_{ED}

Using interaction charts prepared for biaxial bending. The procedure involves:

1. Using, d_2 and b_2 evaluate $\frac{d_2}{h}, \frac{b_2}{h}$ to choose appropriate chart.
2. Compute Normal force ratio: $v = \frac{N_{Ed}}{f_{cd}*b*h}$, Moment ratios: $\mu_{sd} = \frac{M_{ED}}{f_{cd}*b*h^2}$
3. Enter the chart and pick ω (the mechanical steel ratio).
4. ComputeA_s , $A_s = \frac{ω*A_c*f_{cd}}{f_{yd}}$

Sample calculation for ground floor column

$$d_2 = 43mm, b_2 = 43 \; and \; \frac{d_2}{h}, \frac{b_2}{h} = 0.053 \cong 0.1$$

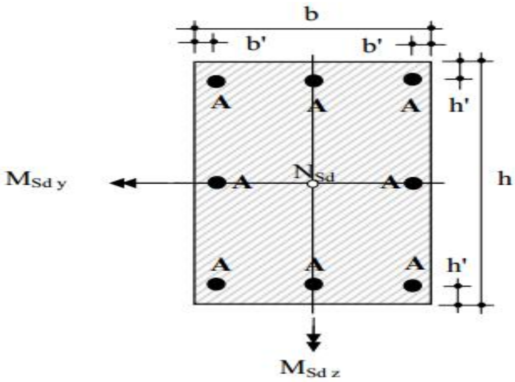


Figure 8-5 Reinforced column section

$N_{Ed} = 5678.59kN$, $v = \frac{N_{Ed}}{f_{cd}*b*h} = \frac{5678.59kN}{14.167*800*800} = 0.626 \cong 0.7$

$M_{ed,y} = 253.506kNm$, $\mu_{sd,y} = \frac{253.506}{14.167*800*800^2} = 0.0349 \cong 0.035$

$M_{ed,x} = 151.43kNm$, $\mu_{sd,x} = \frac{151.43}{14.167*800*800^2} = 0.0208 \cong 0.021$

Using the above data ω = 0 which means use minimum longitudinal reinforcement.

According to ES EN 1992-1-1:2015 section 9.5.2(2) minimum longitudinal reinforcement is

$$A_{s,min} = \max \begin{cases} \frac{0.1 * N_{Ed}}{f_{yd}} \\ 0.002 * A_c \end{cases}$$

$$A_{s,min} = \max \begin{cases} \frac{0.1*5678.59kN}{347.826Mpa} \\ 0.002 * 800^2mm^2 \end{cases} = \max \begin{cases} 1632.59mm^2 \\ 1280mm^2 \end{cases} = 1632.59mm^2$$

Table 8-5 Area of longitudinal reinforcement

column	N_{ED}(kN)	M_{ED}(kNm)		v	$\mu_{sd,y}$	$\mu_{sd,x}$	ω	$A_{s,min}$ (mm^2)
		Y	x					
Ground	5678.59	253.54	151.62	0.6263	0.034955	0.020903	0	1632.595
1st	5139.6	227.63	137.23	0.566854	0.031382	0.018919	0	1477.635
2nd	4595.32	200.33	122.70	0.506825	0.027618	0.016915	0	1321.155
3rd	4057.32	185.55	93.55	0.584474	0.038186	0.012897	0	1166.48
4th	3530.38	170.26	84.59	0.508566	0.035038	0.011662	0	1014.985
5th	3005.31	122.07	62.82	0.589263	0.039892	0.008661	0	864.0268
6th	2501.196	129.73	64.83	0.490419	0.042394	0.008938	0	720
7th	2004.04	110.02	55.25	0.39294	0.035954	0.007618	0	720
8th	1514.39	102.76	53.31	0.296932	0.03358	0.00735	0	720
9th	1020.641	47.39	24.79	0.450272	0.052271	0.003418	0	320
10th	527.5974	46.18	13.62	0.232758	0.050937	0.001878	0	320
Roof	35.09	32.39	21.95	0.015481	0.03572	0.003027	0	320

8.4.7 Number of bar

$$N = \frac{A_{s,min}}{a_s} = \frac{1632.595\text{mm}^2}{10^2 * \pi} = 5.199 \cong 6\emptyset 20$$

Table 8-6 Number of bars for longitudinal reinforcement

Column	$A_{s,min}$ (mm^2)	N
Ground	1632.595	$6\emptyset 20$
1st	1477.635	$5\emptyset 20$
2nd	1321.155	$5\emptyset 20$
3rd	1166.48	$6\emptyset 16$
4th	1014.985	$6\emptyset 16$
5th	864.0268	$6\emptyset 14$
6th	720	$5\emptyset 14$
7th	720	$5\emptyset 14$
8th	720	$5\emptyset 14$
9th	320	$3\emptyset 14$
10th	320	$3\emptyset 14$
Roof	320	$3\emptyset 14$

8.4.8 Transverse reinforcement

According to ES EN 1992-1-1:2015 section 9.5.3 The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars, whichever is the greater. The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5 mm. The transverse reinforcement should be anchored adequately

Check for diameter of transverse reinforcement

$$\emptyset_t = max\begin{cases} 6mm \\ \frac{20}{4}mm \end{cases} < \emptyset_{t,provided} = 8mm \ldots\ldots\ldots\ldots..ok$$

The spacing of the transverse reinforcement along the column should not exceed$S_{cl,tmax}$.

Check for spacing

The value of $S_{cl,tmax}$ for use in a Country may be found in its National Annex. The recommended value is the least of the following three distances:

- 20 times the minimum diameter of the longitudinal bars
- he lesser dimension of the column
- 400 mm

$$S_{cl,tmax} = max\begin{cases} 20 * 14mm \\ 400mm \\ 400mm \end{cases} > S_{cl,provided} = 200mm \ldots\ldots\ldots\ldots..ok$$

8.5 Detailing

8.5.1 Lap length

According to ES EN 1992-1-1:2015 section 8.7.1 Forces are transmitted from one bar to another by: lapping of bars, with or without bends or hooks, welding and mechanical devices assuring load transfer in tension-compression or in compression only. The detailing of laps between bars shall be such that:

- the transmission of the forces from one bar to the next is assured;
- spilling of the concrete in the neighborhood of the joints does not occur;
- Large cracks which affect the performance of the structure do not occur.

The design lap length is:

$$l_0 = \alpha_1 * \alpha_2 * \alpha_3 * \alpha_5 * \alpha_6 * l_{b,req} \ \geq \ l_{0,min}$$

$$\alpha_1, \alpha_2, \alpha_3, \alpha_5, \alpha_6 = 1.5$$

$$l_{b,req} = \frac{\emptyset * \sigma_{sd}}{4 * f_{bd}} \text{ Where, } f_{bd} = 2.7Mpa, \sigma_{sd} = 347.826Mpa$$

Table 8-7 Summary lap length for column

$\emptyset$ (mm)	$l_{b,req}$ (mm)	l_0 (mm) provided lap length
14	450.89	680
16	515.3	775
20	644.13	970

Check for minimum

$$l_{0,min} > max\begin{cases} 0.3 * l_{b,req} \\ 15 * \emptyset \\ 200mm \end{cases} < l_0 \text{ ………… ok}$$

8.5.2 Anchorage of links and shear reinforcement for columns

Anchorage of links and shear reinforcement for columns is the same as that of for beams in section 7.6.4.

9 Foundation

A building is generally composed of a superstructure above the ground and a substructure which forms the foundation below ground. The foundation transfers and spread the loads from a structure's columns and walls in to the ground. The safe bearing capacity of the soil must not have exceeded otherwise excessive settlement may occur, resulting in damage to the building and its service facilities. Foundation failure can also affect the overall stability of a structure so that it is liable to slide, to lift vertically or even overturn.

The earth under the foundation is the most variable of all the materials that are considered in the design and construction of an engineering structure. Under one small building the soil may vary from soft clay to dense rock. Also the nature and properties of the soil will change with the seasons and the weather. So that it is important to have engineering survey made of the soil under a proposed structure so that variations in the strata and the soil properties can be determined.

9.1 Types of foundations commonly used are

- Isolated bases for individual columns
- Combined bases for several columns
- Rafts for whole buildings which may incorporate basements

The type of foundation to be used depends on a number of factors such as,

- Soil properties and conditions
- Type of structure and loading
- Permissible amount of differential settlement

The design of any foundation consists of two parts

- Geotechnical design to determine the safe bearing strength of the soil
- Structural design of the foundation using reinforced concrete

9.2 Pad footings

The footing for a single column may be made square in plan, but where there is a large moment acting about one axis it may be un economical to have a rectangular base.

Assuming there is a linear distribution the bearing pressures across the base will take one of the three forms shown in Figure 9-1, according to the magnitudes of the axial load N and M acting on the base.

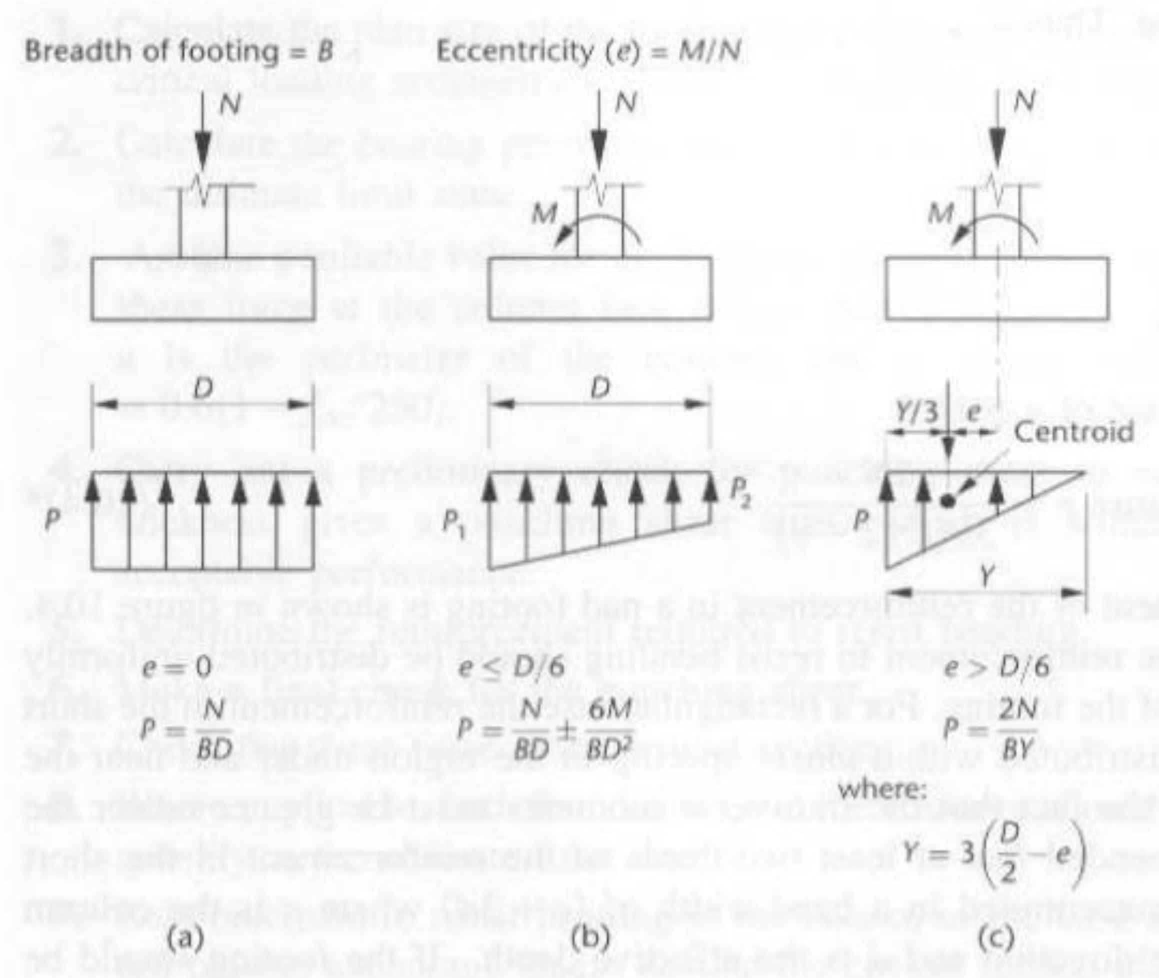


Figure 9-1 Pad footing pressure distributions

1, In the Figure 9-1(a) there is no moment and the pressure is uniform

$$p = \frac{N}{BD}$$

2, with a moment M acting as shown, the pressures are given by the equation for axial load plus bending. This is provided there is positive contact between the base and the ground along the complete length D of the footing, as in Figure 9-1(b) so that

$$P = \frac{N}{BD} \pm \frac{My}{I}$$

Where

I Is the second moment area of the base about the axis of bending; and

Y Is the distance from the axis to where the pressure is being calculated

Substituting for $I = {BD^3}/{12}$ and= ${D}/{2}$, the maximum pressure is

$p_1 = \frac{N}{BD} + \frac{6M}{BD^2}$, and the minimum pressure is, $p_2 = \frac{N}{BD} - \frac{6M}{BD^2}$

There is positive contact along the base if p_2 is positive.

When p_2 is just equal to zero, $\frac{M}{N} = \frac{D}{6}$ so that for p_2 always to be positive ${M}/{N}$ or the effective eccentricity, e must never be greater than ${D}/{6}$. In this case the eccentricity of loading is said to lie with in the middle third of base.

When the eccentricity, e is greater than ${D}/{6}$ there is no longer a positive pressure along the length D and the pressure diagram is triangular as shown in figure 2(c).

Balancing the downward upward pressures

$$\frac{1}{2} pBY = N$$

Therefore

Maximum pressure $p = \frac{2N}{BY}$

Where

y Is the length of positive contact

The centroid of the pressure diagram must coincide with the eccentricity of loading in order for the load and reaction to be equal and opposite. Thus

$$\frac{Y}{3} = \frac{D}{2} - e \text{ Or } y = 3\left(\frac{D}{2} - e\right)$$

There for in this case of $e > \frac{D}{6}$ the maximum pressure $p = \frac{2N}{3D(D/_2 - e)}$

A typical arrangement of the reinforcement in a pad footing is shown in figure 3. With a square base the reinforcement to resist bending should be distributed uniformly across the full width of footing. For a rectangular base the reinforcement in the short direction should be distributed with a closer spacing in the region under and near the column, to allow for the fact that the transverse moments must be grater nearer the column. It is recommended that at least two-thirds of reinforcement in the sort direction should be concentrated in a band width of

$$(c + 3d)$$

Where

C Is the column dimension in the longer direction; and

D Is effective depth.

If the footing should be subjected to a large overturning moment so there is only partial bearing, or if there is a resultant uplift force, the reinforcement may also be required in top face.

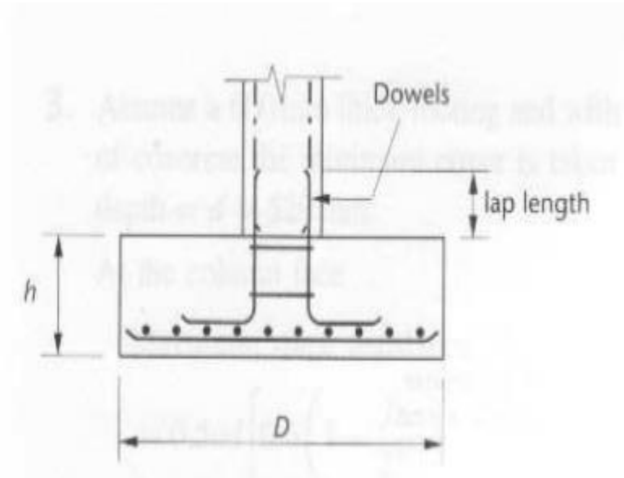


Figure 9-2 Pad footing reinforcement details

Dowels or starter bars should extend from the footing in to the column in order to provide continuity to the reinforcement. These dowels should be embedded in to the footing and extend in to the columns a full lap length. Sometimes a 75mm length of the column is constructed in to the same concrete pour as the footing so as to form a ‘kicker’ or support for column’s shutters. In this case the dowel’s lap length should be measured from the top of the kicker.

The critical sections through the base for checking shear, punching shear and bending are shown on Figure 9-3. The shearing force and bending moments are caused by the ultimate loads from the column and the weight of the base should not be included in these calculations.

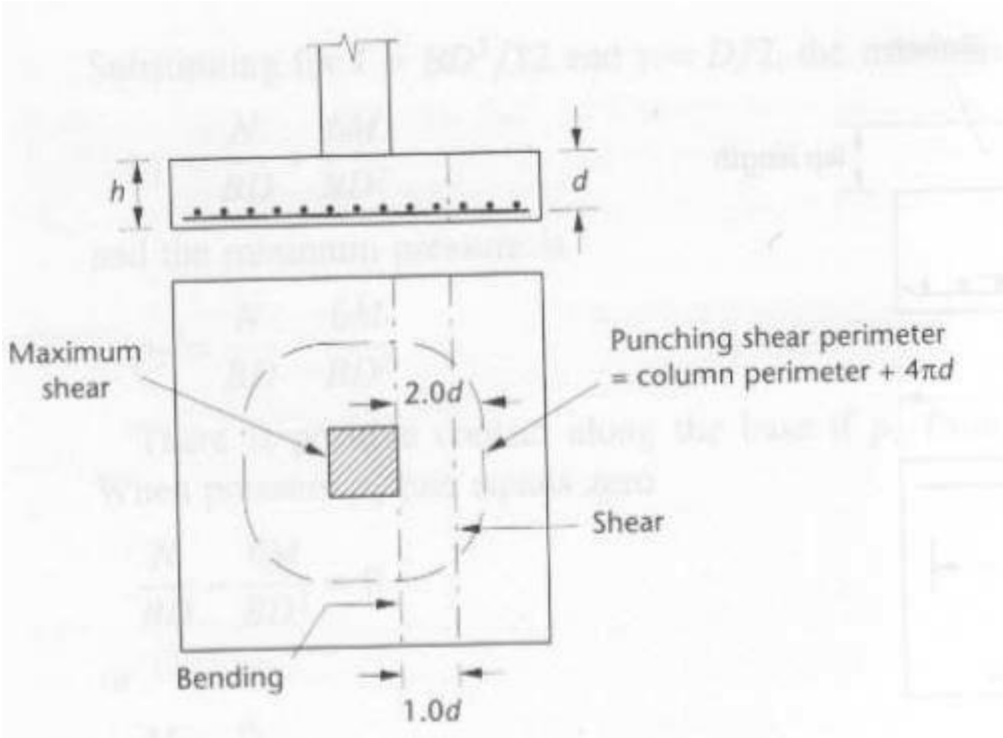


Figure 9-3 Critical section for design

The thickness of the base is often governed by the requirements for shear resistance. Following the 'prescriptive method' the principal steps in the design calculation are as follows:

- Calculate the plan size of footing using the permissible bearing pressure and the critical loading arrangement for the serviceability limit state.
- Calculate the bearing pressure associated with the critical loading arrangement at the ultimate limit state.
- Assume a suitable value for the thickness (h) and effective depth (d). Check that the shear force at the column face is less than

$$0.5v_1 f_{cd} ud = 0.5v_1 ({f_{ck}}/{1.5}) ud$$

Where

U Is the perimeter of the column; and

v_1 Is the strength reduction factor ($u = 0.6(1 - {f_{ck}}/{250})$).

- Carry out a preliminary check for punching shear to ensure that the footing thickness gives a punching shear stress which is within the likely range of acceptable performance.
- Determine the reinforcement required to resist bending.
- Make a final check for the punching shear
- Check the shear force at the critical sections.
- Where applicable, both foundations and the structure should be checked for overall stability at the ultimate limit state.
- Reinforcement to resist bending in the bottom of the base should extend at last a full tension anchorage length beyond the critical section of bending.

9.3 Design of pad footing

Allowable bearing capacity of soil is assumed to be 500kpa

Table 9-1 Superstructure Load transferred to footing from ETABS analysis for column C12

Footing for column C13	N_{ed} (kN)	M_x(kNm)	M_y(kNm)
SLS	4127.28	-3.4467	-1.2679
Max. loads	5678.59	194.71	230

Area proportioning

Using SlS load from above table

$$\sigma_{all} = \frac{N_{ED}}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y}$$

Assuming $L = 1.2B$

$$500 \geq \frac{4127.28}{B^2} \pm \frac{3.447 * {B}/{2}}{\frac{1.2B * B^3}{12}} \pm \frac{1.268 * {B}/{2}}{\frac{B * (1.2B)^3}{12}}$$

Then this calculation gives $B = 2.6$ m and $L = 3.1$ m

Provide rectangular footing $2.6 \text{ m} * 3.1 \text{ m}$

9.4 Structural design

Since the load is eccentric loading, the pressure distribution under the footing should considers the loading cases in Figure 9-1(b) and (c) above

Case (b) occurs when the eccentricity lies within the middle third of the base,$e \leq \frac{B}{6}$

The maximum pressure is $P_1 = \frac{N}{BD} + \frac{6M}{BD^2}$ and the minimum pressure is $P_2 = \frac{N}{BD} - \frac{6M}{BD^2}$

Case (c) occurs when the eccentricity lies out of the middle third of the base,$e > \frac{B}{6}$

In our case $e = \frac{M}{N_{ED}} = \frac{230}{5678.59} = 0.04m \ and \ \frac{B}{6} = o.43m$ the case (b) occurs

Thus, using pressure distribution in under case(b)

$p_{1x} = 760.29$ kpa and $p_{2x} = 648.79$ kpa and $p_{1y} = 759.77$ kpa and $p_{2y} = 649.31$ kpa

For ULS load combination 1($N_{ED} = 1.35G_K + 1.5Q_K$) will give the largest set of actions

($N_{ED} = 5678.79\text{kN}$)

Earth pressure $= \frac{5678.79\text{kN}}{3.1\text{m}*2.6\text{m}} = 704.56\text{kN/m}^2$

Assumptions

- $D = 800\text{mm}$
- $\emptyset 24\text{mm reinforcement}$
- $\text{Cover} = 35\text{mm}$
- $d_x = 729\text{mm}$
- $d_y = 753\text{mm}$
- $d_{mean} = 741\text{mm}$

Dimensions of column

- $C13\ 800mm * 800mm$

Concrete grade

- $C30/35$

Steel grade

- S-500

9.5 Design for shear

9.5.1 Maximum shear resistance capacity, $V_{RD,max}$

$$V_{RD,max} = 0.5udV_1 \frac{f_{ck}}{1.5}$$

Where

V_1 is strength reduction factor $=0.6(1 - \frac{f_{ck}}{250})$ =0.528

u is perimeter of the column

d is mean effective depth

f_{ck} is characteristic cylindrical strength of concrete, MPa

$V_{RD,max} = 12{,}519.94\text{kN} (> N_{ed}$ =5678.59kN) …….. ok

9.5.2 Punching shear

The critical section of checking punching shear is at a distance 2d as shown in figure 4.

Critical perimeter = column perimeter+$4\pi d = 13{,}026\text{mm}$

Area within perimeter = $C^2 + 4 * (4d^2) + 4\pi d^2 = 8.06 * 10^6\ \text{mm}^2$

Therefore

Punching shear force, $v_{ED} = 704.56(2.6 * 3.1 - 8.06) = 0$ kN

Punching shear stress $v_{ED} = \frac{V_{ED}}{\text{perimeter}*d} = \frac{0}{10{,}348\text{mm}*576\text{mm}} = 0\text{N/mm}^2 < 0.36\ ^{\text{N}}/_{\text{mm}^2}$, which is shear resistance of slabs without reinforcement, $V_{RD,c}\ ^{\text{N}}/_{\text{mm}^2}$ for concrete class C30/35 for reinforcement ratio 0.25%

This ultimate shear stress is not excessive, therefore $D = 800\text{mm}$ is suitable estimate.

9.5.3 Wide beam shear

For wide beam shear critical section is at d distance from face of column.

a) Capacity

$$V_{RD,C} = \frac{0.18 * k * (100\rho_1 f_{ck})^{\frac{1}{3}} * b_w d}{\gamma_c}$$

Where

ρ_1 is reinforcement ratio But at this time we do not know area of reinforcement , thus we instead used $\rho_{min} = 0.0013$ for $f_{ck} = 30\ Mpa$

$k = 1 + \sqrt{\frac{200}{d}} \leq 2$

$K = 1.52$

$V_{RD,C} = 553.14\ kN$

b) Action

$$V_{ED,R} = \Delta V_{ED} = \sigma_{m*} A_c \text{ But } \sigma_m = \frac{p_1 + p_d}{2} = \frac{760.29+648.79}{2} = 704.54 \text{ kpa}$$

Where

A_c is surface area ,on which ΔV_{ED} is acting,

$A_c = b_w * (\frac{B}{2} - d - \frac{C}{2})$ = 1,063,400 mm^2

$V_{ED,R} = A_c * \sigma_m = 749.21$ kN $> V_{RD,C}$……... not ok

9.6 Design for flexure

9.6.1 Capacity

$$M_{bal} = 0.167 * f_{ck} * b_w d^2$$

Where

M_{bal} is the design bending resistance;

f_{ck} is characteristic cylindrical strength of concrete, Mpa; and

d is effective depth, mm

$M_{bal} = 7152.33$ kN

9.6.2 Action

The action moment (M_{ED}) can be calculated from stress distribution figure 2 (b) at face of column.

$$M_{ED} = P * A_c * r$$

Where

P Is stress distribution;

A_c Is Concrete area; and

r Is Moment arm.

$M_{EDy} = 1329.72 \text{ kN} < M_{bal}$ ………… ok

$M_{EDx} = 970.69 \text{ kN} < M_{bal}$ …………ok

9.6.3 Reinforcement calculation

SHORT SPAN

$k = \frac{M_{ED}}{f_{ck} b_w d^2} = 0.023$

$z = d\left[0.5 + \sqrt{0.25 - \frac{k}{1.134}}\right] \leq 0.95d$

$z = 692 \text{ mm}$

$A_s = \frac{M_{ED}}{0.87 f_{yk} z} = 3224.67 \text{ mm}^2$

$A_{s,min} = \rho_{min} b_w d = 2{,}464 \text{ mm}^2 < A_s$

$A_{s,max} = 0.04 b_w D = 83{,}200 \text{ mm}^2 > A_s$

Number of bars using Ø24mm bars

$$N = \frac{A_s}{\pi(^{\emptyset}/_{2})^2} = 8$$

$A_{s,provided} = N a_s = 3619 \text{ mm}^2$

Spacing, $s = \frac{b_w}{N} = 387.5 \text{ mm}$

Check maximum spacing

$S_{max} = \min\begin{cases} 3h \\ 400mm \end{cases} = 400mm > 387.5\text{ mm}$ is accepeteble but to be more conservative and minimize cost it is better to provide Ø20 mm reinforcement bars instead of Ø24mm reinforcement bars

Where

H Is depth of footing

Number of bars using Ø20mm bars

$$N = \frac{A_s}{\pi(^{\emptyset}/_{2})^2} = 11$$

$$A_{s,provided} = Na_s = 3455\text{ mm}^2$$

Spacing, $s = \frac{b_w}{N} = 280\text{ mm} < 400mm \ldots\ldots..\text{ok}$

Provide 11 Ø 20 C/C 280 in short span direction

LONG SPAN

$$k = \frac{M_{ED}}{f_{ck}b_w d^2} = 0.025$$

$$z = d\left[0.5 + \sqrt{0.25 - \frac{k}{1.134}}\right] \leq 0.95d$$

$$z = 715\text{ mm}$$

$$A_s = \frac{M_{ED}}{0.87f_{yk}z} = 4273.19\text{ mm}^2$$

$$A_{s,min} = \rho_{min}b_w d = 3034\text{ mm}^2 < A_s$$

$$A_{s,max} = 0.04b_w D = 99{,}200\text{ mm}^2 > A_s$$

Number of bars

$$N = \frac{A_s}{\pi(^{\emptyset}/_{2})^2} = 10$$

$$A_{s,provided} = Na_s = 4524\text{ mm}^2$$

Spacing, $s = \frac{b_w}{N} = 260\text{ mm}$

9.6.4 Check maximum spacing

$S_{max} = \min\begin{cases} 3h \\ 400mm \end{cases} = 400mm > 260\ mm$ Is acceptable but to be more conservative and minimize cost it is better to provide Ø20 mm reinforcement bars instead of Ø24mm reinforcement bars

Number of bars using Ø20mm bars

$N = \frac{A_s}{\pi(^{\emptyset}/_{2})^2} = 14$

$A_{s,provided} = Na_s = 4398.22\ mm^2$

Spacing, $s = \frac{b_w}{N} = 180\ mm < 400mm \ldots\ldots..ok$

Provide 14 Ø 20 C/C 180 in long span direction

9.7 Final Check for Wide Beam Shear

9.7.1 Capacity

$$V_{RD,C} = \frac{0.18 * k * (100\rho_m f_{ck})^{\frac{1}{3}} * b_w d}{\gamma_c}$$

$k = 1 + \sqrt{\frac{200}{d}} \leq 2$

$K = 1.52$

$\rho_m = \frac{A_{st}}{b_w d} = 0.0019$

$V_{RD,C} = 760.56kN$

a. Action

$V_{ED,R} = 749.21\ kN < V_{RD,C}$……... ok

9.8 Anchorage of column starter bars

Anchorage of column starter bar is Apart from the reinforcement in the base, column bars extend at least an anchorage length to which column reinforcement is attached.

$$l_b = \mathrm{Max}\begin{cases} 15\emptyset_{bar} \\ 200mm \end{cases} = 360mm$$

Conclusion

In this final year project, this required a lot struggle to fruit out the final design of this G + 10 residential use building. And each important part of the building was carefully analyzed and designed for the appropriate loading conditions as much as possible to get the final design result.

In design of this building, we have tried to satisfy the most basic requirements of design of a structure in accordance with revised Ethiopian building code standard derived from the European building code standard. The design of reinforced concrete members was made economical, safe and serviceable as much as possible.

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Appendix

Appendix A: summery of flexural design of panels

Msd(KNm)	Fcd(Mpa)	Fyd(Mpa)	b(mm)	d(mm)	µsd	Kz	z(mm)	As(mm2)	Asmin(mm2)	Ast(mm2)	S(mm)	Smax(mm)	Sprovide(mm)	Sprovide(mm)
15.2	14.167	347.83	1000	141	0.0540	0.969	136.63	319.84	238.29	319.84	157.16	400.00	157.16	Ø8C/C150
5.83	14.167	347.83	1000	141	0.0207	0.984	138.74	120.81	238.29	238.29	210.94	400.00	210.94	Ø8C/C 210
13.49	14.167	347.83	1000	141	0.0479	0.971	136.91	283.27	238.29	283.27	177.44	400.00	177.44	Ø8C/C 170
12.9	14.167	347.83	1000	141	0.0458	0.972	137.05	270.61	238.29	270.61	185.75	400.00	185.75	Ø8C/C 180
9.4	14.167	347.83	1000	141	0.0334	0.978	137.90	195.98	238.29	238.29	210.94	400.00	210.94	Ø8C/C 210
10.66	14.167	347.83	1000	141	0.0378	0.976	137.62	222.70	238.29	238.29	210.94	400.00	210.94	Ø8C/C 210
7.79	14.167	347.83	1000	133	0.0311	0.979	130.21	172.00	224.77	224.77	223.63	400.00	223.63	Ø8C/C220
9.38	14.167	347.83	1000	133	0.0374	0.976	129.81	207.75	224.77	224.77	223.63	400.00	223.63	Ø8C/C220
7.37	14.167	347.83	1000	133	0.0294	0.98	130.34	162.56	224.77	224.77	223.63	400.00	223.63	Ø8C/C220
4.89	14.167	347.83	1000	133	0.0195	0.977	129.94	108.19	224.77	224.77	223.63	400.00	223.63	Ø8C/C220
2.31	14.167	347.83	1000	133	0.0092	0.99	131.67	50.44	224.77	224.77	223.63	400.00	223.63	Ø8C/C220
3.88	14.167	347.83	1000	133	0.0155	0.989	131.54	84.80	224.77	224.77	223.63	400.00	223.63	Ø8C/C220
20.14	14.167	347.83	1000	181	0.0434	0.977	176.84	327.43	305.89	327.43	153.51	400.00	153.51	Ø8C/C150
20.53	14.167	347.83	1000	181	0.0442	0.976	176.66	334.11	305.89	334.11	150.44	400.00	150.44	Ø8C/C150
20.53	14.167	347.83	1000	181	0.0442	0.977	176.84	333.77	305.89	333.77	150.60	400.00	150.60	Ø8C/C150
20.14	14.167	347.83	1000	181	0.0434	0.977	176.84	327.43	305.89	327.43	153.51	400.00	153.51	Ø8C/C150
6.08	14.167	347.83	1000	181	0.0131	0.99	179.19	97.55	305.89	305.89	164.32	400.00	164.32	Ø8C/C160
14.92	14.167	347.83	1000	141	0.0530	0.969	136.63	313.95	238.29	313.95	160.11	400.00	160.11	Ø8C/C160
14.28	14.167	347.83	1000	141	0.0507	0.97	136.77	300.17	238.29	300.17	167.45	400.00	167.45	Ø8C/C160
8.59	14.167	347.83	1000	141	0.0305	0.98	138.18	178.72	238.29	238.29	210.94	400.00	210.94	Ø8C/C210
11.32	14.167	347.83	1000	141	0.0402	0.976	137.62	236.49	238.29	238.29	210.94	400.00	210.94	Ø8C/C210
7.55	14.167	347.83	1000	141	0.0268	0.981	138.32	156.92	238.29	238.29	210.94	400.00	210.94	Ø8C/C210
11.61	14.167	347.83	1000	141	0.0412	0.972	137.052	243.545	238.29	243.55	206.39	400.00	206.39	Ø8C/C200
8.77	14.167	347.83	1000	141	0.0311	0.979	138.039	182.655	238.29	238.29	210.94	400.00	210.94	Ø8C/C210