ADDIS ABABA UNIVERSITY INSTITUTE OF TECHNOLOGY SCHOOL OF
CIVEL AND ENVIRONMENTAL ENGINEERING

POST-TENSIONING CONCRETE FLAT SLAB DESIGN FOR BUILDINGS, CONSTRUCTION PRACTICE AND PROSPECTS IN ETHIOPIA

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By

Wondmagegne Dagne Awol

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ADDIS ABABA UNIVERSITY
INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING
POST GRADUATE PROGRAM UNDER STRUCTURAL ENGINEERING

“POST-TENSIONING CONCRETE FLAT SLAB DESIGN FOR BUILDINGS, CONSTRUCTION PRACTICE AND PROSPECTS IN ETHIOPIA”

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APPROVED BY BOARD OF EXAMINERS

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DECLARATION

I declare that this project entitled "Post-Tensioning Concrete Flat Slab Design for Buildings, Construction Practice and Prospects in Ethiopia" is my original work. This project has not been presented for any other university and is not concurrently submitted in candidature of any other degree, and that all sources of material used for the project have been duly acknowledged.

Candidate Name:- Wondmagegne Dagne Awol

Signature:- ________________________
Place: - Faculty of Technology, Addis Ababa University.
Date: - JULY 4, 2019

DEDICATION

IN LOVING MEMORY OF MY FATHER

ACKNOWLEDGEMENTS

I am extremely grateful thanks from the depth of my heart is to Almighty GOD for endowing me with the courage, strength as well as health throughout my school time and the full help provided by him for the successful accomplishment of this final project.

Next, it is my deepest gratitude and respect to my advisor Dr.Abreham Gebra, for his guidance, valuable advice and continuous encouragement given at the initial stage in the preparation of project proposal to the ultimate completion of the project.

I convey my heartiest gratitude and love to my loving mother Angoch Haile for her continuous moral support and encouragement. I also humbly appreciate my siblings and my friends for help, moral support and encouragement throughout the study period.

Thank you all.

GOD BLESSES ETHIOPIA.

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LIST OF SYMBOLS AND ABBREVATIONS

Symbols	Description
A:-	Gross area of concrete section
f_{ck}:-	Characteristic cylindrical strength of concrete.
f_{cm}:-	Mean strength of concrete.
$f_{ck}(t)$:-	Characteristic strength of concrete at time t
f_{ctm}:-	Mean tensile strength of concrete
f_{pk}:-	Characteristic strength of prestressing steel.
$f_{p0.1k}$:-	Stress at 0.1% strain.
P_{max}:-	Allowable maximum tendon force at transfer.
$P_{mt}(x)$:-	Mean prestress force after time t.
$P_{mo}(x)$:-	Prestress force at transfer
$\sigma_{p,max}$:-	Maximum tendon stress capacity at transfer.
$\sigma_{pmo}(x)$:-	Allowable tendon stress during tensioning or anchoring
$\sigma_{ck,all}(t)$:-	Allowable concrete stress during tensioning or anchoring at time t
E_{cm}:-	Mean secant modulus of concrete.
E_p:-	Modulus of tendon steel.
E_s:-	Modulus of reinforcing steel
Z_t:-	Centroid of section from top fiber.
Z_b:-	Centroid of section from bottom fiber.
f_{mid}:-	Tendon drape in mid span.
f_{end}:-	Tendon drape in end span.
$\Delta\sigma_c(t)$:-	Concrete stress due to prestressing at time t.
$d_{pl,sup}$:-	Depth of tendon centroid from top at support
$d_{pl,mid}$:-	Depth of tendon centroid from bottom.
e_1:-	Eccentricity of lower point of the profile.
e_2:-	Eccentricity of upper point of the profile.
S:-	Coefficient depends on the type of cement.
V_{min}:-	Minimum shear resistance.
$V_{RD,C}$:-	Allowable shear resistance.

V_{ED}:-	Shear stress on critical area due to factored load
σ_{cp}:-	Concrete stress due prestressing.
R:-	Radius of curvature.
θ:-	The angular displacement sum over the distance x.
μ:-	Coefficient of friction.
K:-	Unintentional angular displacement.
$\Delta\sigma_{p,\mu}$:-	Frictional stress losses
$\Delta\sigma_{p,el}$:-	Concrete deformation losses.
$\Delta\sigma_{p,set}$:-	Loss at anchorage.
L_{set}:-	Length affected by anchorage set.
$\Delta\sigma_{p,c+s+r}$:-	Creep, shrinkage and relaxation stress.
ε_{cs}:-	Total shrinkage strain.
ε_{cd}:-	Drying shrinkage strain.
ε_{ca}:-	Autogenous shrinkage strain.
$\Delta\sigma_{pr}$:-	Variation of stress due to relaxation.
$\Delta\sigma_{1000}$:-	The relaxation loss at 1000 hours after tensioning.
$\Delta\sigma_{C,QP}$:-	The concrete stress due to quasi-permanent actions.
$\Delta P_{c,c+s+r}$:-	The total long term losses.
K_{ec}:-	Column equivalent stiffness
K_{col}:-	Column stiffness.
K_s:-	Slab stiffness.
K_t:-	Torsional stiffness.
I_c:-	Column moment of inertia
I_s:-	Slab moment of inertia.
DF:-	Distribution factor
COF:-	Carryover factor.
M_A:-	Fixed end moment at A.
M_B:-	Fixed end moment at B.
M_C:-	Fixed end moment at C.

$M_{R,A}$:-	Fixed end moment due to restraint at end support.
$M_{R,B}$:-	Fixed end moment due to restraint at interior support.
K_c:-	Coefficient account for the nature of the stress distribution.
K_1:-	Coefficient account for the effect of axial forces.
$A_{s,min}$:-	Minimum reinforcement area.
ϕM_n:-	Design flexural strength.
M_u:-	Factored moment
$U_{out,ef}$:-	Control perimeter
Δ_{creep}:-	Creep deflection expected.
Δ_{LL}:-	Live load deflection
$\Delta_{Non,SLL}$:-	Deflection due to non-sustained live load.
$\Delta_{creep,NSLL}$:-	Sum of creep and non-sustained live load deflections
U_1:-	Critical perimeter
d_p:-	Tendon depth
A_{ps}:-	Tendon cross section area.
d_{duct}:-	Duct diameter
ϕ_p:-	Percentage of self-weight balanced by prestressing.
$D_{min,fire}$:-	Minimum depth required for given fire rating.

ABSTRACT

It’s been a decade since post-tension system began to be applied in earnest to buildings worldwide. In the meantime, post-tension system has been used in various buildings as main structural system including tall buildings and it plays a role to overcome architectural limit of regular reinforced concrete buildings particularly in the realization of long span with shallower depth than other structural system.

In Ethiopia the benefits of post stressing and particularly of post-tensioning flat slab in building construction yet to be fully recognized. In this project an attempt is made to assess the level of practice, the problems and achievements on the design and construction of post-tensioned flat slab in Ethiopia through distributing Questionnaires to be filled by professionals currently working on design and construction of buildings.

The study suggests that measures shall be taken for the top identified factors hindering the wide application of post-tensioned flat slab in our construction industry widely like; Universities has not been covered the topics deeply in under-graduate sections, Shortage of research literatures, seminars and related design examples, Lack of experienced consultant, contractor and other engineering professionals and Lack of professionals and expertise.

The basic concepts, principles and advantages of post-tensioned flat slab design are discussed and also this project attempt to compares the cost effectiveness of post-tensioned flat slab system with respect to reinforced concrete flat slab system. Both the systems are analyzed using ETABS which is based on the design methodology. The results indicate that post-tensioned flat slabs are cheaper than the reinforced flat slab systems.

The analysis of post-tensioned flat slab is basically a trial and error process in an effort to reach the beast proportions. Manual computations of the analysis may take a time for engineers to arrive in best design. The study of the spreadsheet analysis of post-tensioned flat slab using Excel spreadsheet will be developing in this project. The design example is shown based on simple procedures by using design excel spreadsheet including ETABS demonstrations.

Key words: - Prestressed concrete, Post-tensioned(PT), flat slab, equivalent frame method, load balancing method.

CHAPTER ONE

INTRODUCTION

1.1 General

Common practice of design and construction of buildings in Ethiopia is to support the slabs by beams and support the beams by columns. This may be called as beam-slab construction. The beams reduce the available net ceiling height. Hence in warehouses, offices and public halls sometimes beams are avoided and slabs are directly supported by columns are called Flat slabs.

Therefore, the main objective of a structural designer is to design the structures as light as possible without compromising the strength and stability of structures. These objectives can be safely achieved by varying the sections that is providing thin slabs and avoiding beams.

Bending moments and shear forces increases with increase in span, this leads to increase in the cross- section of members and amount of structural steel. Therefore, prismatic beams may become uneconomical. Moreover, with increase in depth there is a considerable reduction in head-room. Under such circumstances flat slabs may provide appealing solution.

Absence of beam gives a plain ceiling, thus giving better architectural and less vulnerability in case of fire than in usual cases where beams are used. Plain ceiling diffuses light better, easier to construct and requires cheaper form work.

The development of prestressing technology has certainly constituted one of the more important improvements in the fields of structural engineering and construction. Referring particularly to post-tensioning applications, it is generally recognized how it opens the possibility to improve economy, structural behavior and aesthetic aspects in concrete solutions.

A survey of practical situations in Ethiopia would show that post-tensioning solutions are frequently not adopted because some of those involved are not familiar with prestressing technology and its advantages.

It should be pointed out that post-tensioning allows more architectural freedom and can provide important functional advantages; longer spans providing more “flexible” solutions, transition structures solving the conflicts of vertical discontinuities in the building use, and slender column-free spaces for public areas, are good examples, where post-tensioning is the right solution.

Concerning economic aspects, the more significant cost in building structural corresponds to the floor structural system. The use of post-tensioning allows slender and lighter floor systems with gains on the total building height. Due to the reduction of permanent loads and seismic action effects, the floor weight influences the size of the vertical elements and foundations. It should be as well emphasized the construction time can in general be substantially reduced.

The objective of this project is to provide optimal design method, comparing ordinary reinforced concrete flat slab and post-tensioned flat slab coastwise, and assess construction practice and provide suggestion and prospects in Ethiopia.

1.2 Statement of the problem

As it is mentioned in the above section, the post-tensioned flat slab construction is widely used in most part of the world. However, the design and construction of post-tensioned flat slab in Ethiopia is still not well developed and also we have limited source of design tools .so it is useful to assess the practice of design and construction in Ethiopia and preparing excel design tool by using new Ethiopian design code.

The questionnaire is prepared to investigate the design and construction practice of post-tensioned concrete flat slab in our buildings. The response of questionnaire will be summarized and included in the thesis on chapter three. The main objectives of this thesis are to assess the practice and promote the widely application of pre-stressed concrete construction in the country.

1.3 General objective

The main objective of this chapter is to show the benefits of using post-tensioning flat slab floor system for the more common practical applications in concrete buildings.it is also drafted with the goal of motivating building designers to use post-tensioning flat slab: basic design aspects related to pre-stressing effects and design criteria are summarized and conceptual design aspects are emphasized.

The primary objective of this thesis is provide optimal excel design tool by using new Ethiopian code and assess the practice and prospects in Ethiopia.

1.4 Specific objectives

It is hoped that this project will be of interest to practicing engineers and aspiring students who want to “get it right the first time”!

The following specific objectives will be attained at the end:-

- To find optimal design method of post-tensioned flat slab.
- To understand the advantages of using post tensioned flat slab especially for long span slab panels.
- To understand the extent of practice in Ethiopia.
- To assist in the most effective use of construction materials and labors.

1.5 Methodology of the project

This study undertakes in three phases to identify the importance, the construction practice of post-tensioned flat slab construction of buildings in Ethiopia and preparations of design spreadsheet.

Phase 1:- preparing general overview and literature reviews by referring related references.

Phase 2:- prepare questionnaire survey which dealt with the construction practice in Ethiopia including slab construction. Asses the advantages, and cost comparison carry out for the reinforced concrete and post-tensioned flat slab.

Phase 3:- preparing excel design tool by using Ethiopian code and demonstrate by example and software analysis (ETABS 2016).

1.6 Study variables

- ➢ Independent variables
    - Materials (concrete ,high-strength steel tendon and reinforcement bars)
    - Dimensions.
    - Loads (live and dead)
- ➢ Dependent variable
    - Slab capacity, deflection and serviceability

1.7 Data collection process

This research adopts a combination of both quantitative and qualitative methods to assess the current practice of post-tensioned flat slab in buildings. The primary data were obtained with the use of structured questionnaires (quantitative) and selected qualitative subjective questionnaires for possible mitigation measures. The combination of quantitative with qualitative methods of data collection in research has become a common practice in recent years for greater understanding and validation of results (Brayman, 2006)

1.8 Research Questions

This research may answer the following questions:

- What is the purpose of using post-tensioned flat slab?
- How could we identify optimal design methods?
- What are the merits of using post- tensioned flat slab?
- In what extent Ethiopia uses post-tensioned flat slab?

1.9 Scope and Delimitations

This study shall identify in the first place the problem or reasons why post-tensioned concrete flat slab construction system in high-rise buildings is not widely used, in spite of the fact that many customary reinforced concrete buildings were built throughout the country in the past years. This study shall also summarize the basic concept and principles in the design and construction of post-tensioned flat slab by developing design excel spreadsheet and show the advantage of the system over the other are also demonstrate using examples by ETABS.

It is believed that, once the application of this system is started in the country, it will open the gate for further research in the adaption of new construction system of the developed world to the local condition.

The outline of the project mainly includes assess of practice of post-tensioned flat slab in Ethiopia, a theory, an excel design template preparation and demonstration parts. The theory part gives a description of the phenomenon post-tensioned flat slab. After the theory part the procedure and assessment for the practice in Ethiopia. The project finally tied together in a preparation of design template and followed by conclusions; however some delimitation existed is given here.

- The project only treated flat slabs without drop panels.
- The study only focused limited number of participants in Ethiopia construction sector.
- The design template did not include flat slab with column drop.
- The load combination did not include point loads, only distributed loads.
- Openings and inserts were not treated in this project.

1.10 Importance of the study

On its completion, this research will assist our country’s practice and prospect of using post-tensioned flat slab. And will provide knowledge of advantages and methods of designing over other like solid concrete flat slab. Therefore, all this merits will make this research very useful and practicable in future in our country, especially for long span high rise buildings.

1.11 Organization of the project

This project has been written according to the recommended format of the Addis Ababa University. The following is a brief outline of each chapter.

In chapter two, a brief review of post-tensioned flat slab, the history of its development, basic concept of prestressing and systems for prestressing are described.

The level of design and construction practice in Ethiopia, questionnaire evaluation, respondents’ discussion and current practice are described in chapter three.

In chapter four, general design consideration, literature revive, design methodology, design spreadsheet preparation, cost comparison of post-tensioned flat slab with respect to reinforced concrete flat slab based on Euro code, discussion and result are described. And in the last chapter, conclusion is described in short and precise manner.

CHAPTER TWO

BRIEF REVIEW OF POST-TENSIONED CONCRETE FLAT SLAB

"Today's flat-slab post-tensioned buildings, for example, with column spaced 12m on center and span-depth ratios of 40, are more complex and required more engineering attention than typical flat-slab buildings of 40 years ago, with column spaced at 6m on center and span-depth ratio of 20."

-Randall poston,

Chair of ACI Committee 318

(Poston and Dolan, 2008)

2.1 Historical overview

Prestressed concrete is not a new concept, dating back to 1872, when P.H. Jackson, an engineer from California, patented a prestressing system that used a tie rod to construct beams or arches from individual blocks. In 1888,C.W. Doehring of Germany obtained a patent for prestressing slabs with metal wires. But these early attempts at prestressing were not really successful because of the loss of the prestress with time.

After a long lapse of time during which little progress was made because of the unavailability of high-strength steel to overcome prestress losses, R.E. Dill of Alexandria, Nebraska, recognized the effect of the shrinkage and creep (transverse material flow) of concrete on the loss of prestress. He subsequently developed the idea that successive post-tensioning of unbonded rods would compensate for the time-dependent loss of stress in the rods due to the decrease in the length of the member because of creep and shrinkage.

Linear prestressing continued to develop in Europe and France, in particular through the ingenuity of Eugene Freyssinet, who proposed in 1926 through 1928 methods to overcome prestress losses through the use of high-strength and high-ductility steels. In 1940, he introduces the now well-known and well-accepted Freyssinet system comprising the conical wedge anchor for 12-wire tendons.

(Casson ,1971) and (Torr ,1964) discuss and illustrate Egyptian boats, built approximately 3500 years ago, in which the hull, posts and ropes formed structures to prevent "hogging," or negative curvature in hulls. These boat structures were prestressed by twisting ropes, as shown in Fig.2.1.

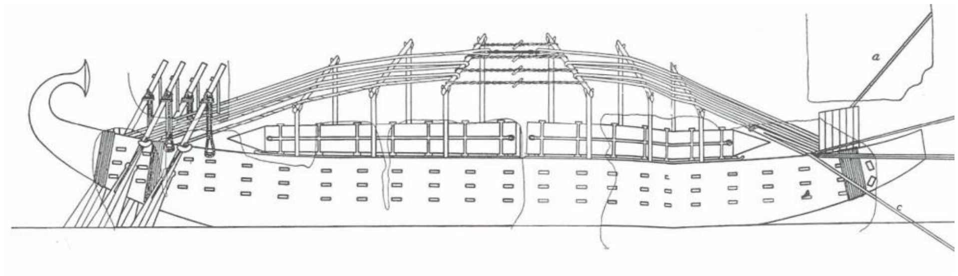


Fig.2.1:- Egyptian barge prestressed by twisting ropes.

Today, prestressed concrete is used in buildings, underground structures, TV towers, floating storage and offshore structures, power stations, nuclear reactor vessels, and numerous types of bridge systems including segmental and cable-stayed bridges. Note the variety of prestressed structures in the figure throughout this paper; they demonstrate the versatility of the prestressing concept and its all-encompassing applications.

The success in the developing and construction of all landmark structures worldwide has been due in no small measure to the advances in the technology of materials, particularly prestressing steel, and the accumulated knowledge in estimating the short-and long-term losses in the prestressing forces.

Fig.2.2:-Landmark 72 (source: Heerim Architecture)

2.2 Overview of Post-tensioned concrete

2.2.1. Introduction

Prestressed concrete has been used as alternative of reinforced concrete all over the world for decades. Now, prestressed technology provides efficient solutions for various structural members and situations: floor structure, vertical element, lateral load resisting system, etc.

The development of prestressing technology has certainly constituted one of the more important improvements in the field of structural engineering and construction. Referring particularly to post-tensioning applications, it is generally recognized how it opens the possibility to improve economy, structural behavior and aesthetic aspects in concrete solutions.

In spite of the simplicity of its basic concepts and well known advantages, the application extent of post-tensioning solutions cannot be considered harmonized in the different areas and structural applications. In fact, for various reasons, it appears that the potential offered by presterssing is far from being exploited, especially in building structures field.

It should be pointed out that post-tensioning allows more architectural freedom and can provide important functional advantages some of them are as follows:-

- Longer spans providing more flexible solutions, transition structures solving the conflicts of vertical discontinuities in the building use, and slender column-free spaces for public areas.
- For a given span post-tensioned floors requires less concrete.
- If a significant part of the load is resisted by post-tensioning the non-pre-stressed reinforcement can be simplified and standardized to a larger degree.
- It allows earlier stripping of formwork.
- Usually the permanent floor load is largely balanced by draped post-tensioning tendons so that only the weight of the wet concrete of the floor above induces flexural stress. These are often the same order as the design live load stress. Hence back-propping of one floor below is usually sufficient.
- Assembling of precast elements by post-tensioning avoids complicated reinforcement bar connections, and thus can significantly reduce erection time.
- Concerning economic aspect, the more significant cost in building structure corresponds to the floor structure system. The use of post-tensioning allows slender and lighter floor systems with gains on the total building height.

2.2.2 Basic concepts of Prestressing

2.2.2.1 General

Concrete is strong in compression, but weak in tension: its tensile strength varies from 8 to 14 percent of its compressive strength. Due to such a low tensile capacity, flexural cracks develop at early stages of loading, in order to reduce or prevent such cracks from development, a concentric or eccentric force is imposed in the longitudinal direction of the structural elements. This force prevents the cracks from developing by eliminating or considerably reducing the tensile stresses at the critical mid-span and support sections at service load, thereby raising the bending, shear, and torsional capacity of the sections.

Such an imposed longitudinal force is called a “prestressing force”, i.e., a compressive force that prestresses the section along the span of the structure element prior to the application of the transverse gravity dead and live loads. The type of prestressing force involved, together with its magnitude, is determined mainly on the basis of the type of system to be constructed and the span length and slenderness desired. Since the prestressing force is applied longitudinally along or parallel to the axis of the member, the prestressing principle involved is commonly known as *linear prestressing*.

Circular prestressing, used in liquid containment tanks, pipes, and pressure reactor vessels, essentially follows the same basic principles as does linear prestressing. The circumferential hoop, or "hugging" stress on the cylindrical or spherical structure, neutralizes the tensile stresses at the outer fibers of the curvilinear surface caused by the internal contained pressure.

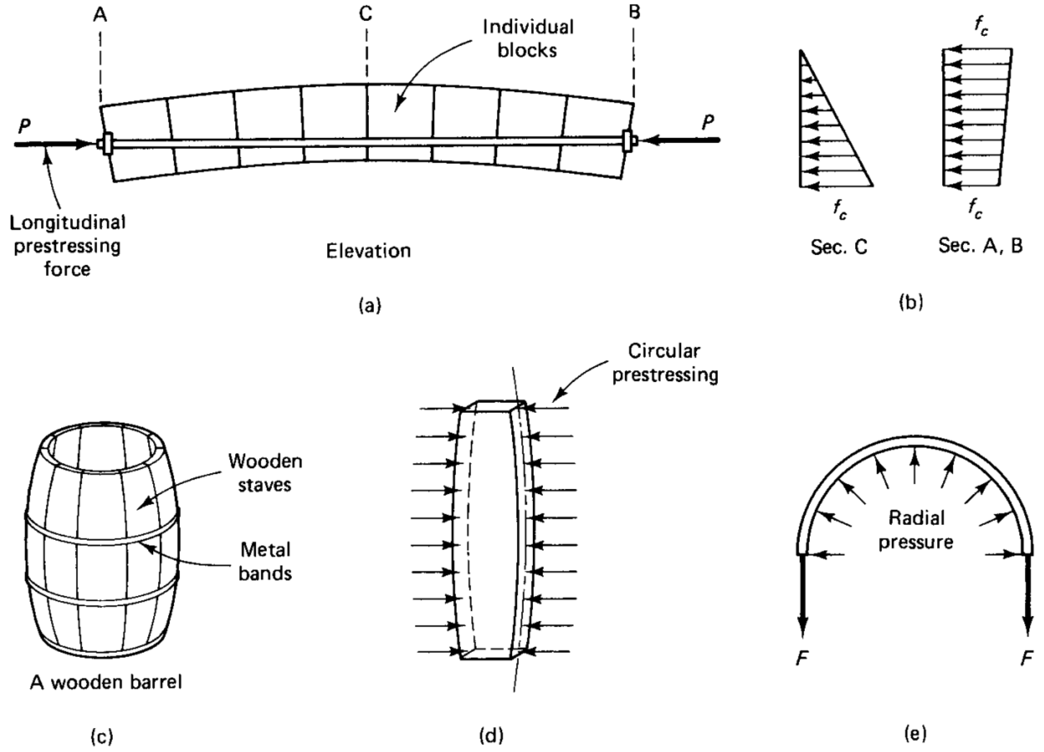


Fig.2.3:-Prestressing principle in linear and circular prestressing

Figure 1.3 illustrates, in a basic fashion, the prestressing action in both types of structural systems and the resulting stress response. In (a), the individual concrete blocks act together as a beam due to large compressive prestressing force P. Although it might appear that the blocks will slip and vertically simulate shear slip failure. In fact they will not because of the longitudinal force p. similarly, the wooden staves in (c) might appear to be capable of separating as a result of the high internal radial pressure exerted on them. But again, because of the compressive prestress imposed by the metal bands as a form of circular prestressing, they will remain in place.

If the material has a low tensile strength, which is the case for masonry and unreinforced concrete, the load bearing capacity will be correspondingly low. One way of compensating for this is to apply a compressive force to the element. This will increase the stresses on the compressive side and reduce, or even eliminate, the tensile stresses.

In modern concrete structure, most tensile forces are taken by reinforcement. However, no significant stress can develop in the reinforcement until the concrete has cracked. This cracking can often be accepted, but for various reasons it is sometimes desirable to prevent it, or at least reduce it. Then, again, prestressing can be used. The prestressing is then normally achieved by means of steel tendons in the form of bars, strands or cables, stressed in tension and thereby producing compression in the concrete.

2.2.2.2 Prestressing in concrete structures

Prestressing is a way of counteracting the effect of external loads on a structure by imposing a state of stresses contrary to the load effects. The most common way to achieve this is by means of tendons, which are stressed prior to final loading of the structure.

Prestressing with tendons has two main effects, axial and transverse, see Fig. 2.4. The axial effect gives compression in the concrete, caused by anchorage forces at the tendon ends. In case (b), the eccentricity of the straight tendon cause bending in addition to the axial effect. Finally, the use curved tendons (case c) introduces a transverse effect that can be design to more or less counteract the external loads with both axial, bending and shear effects.

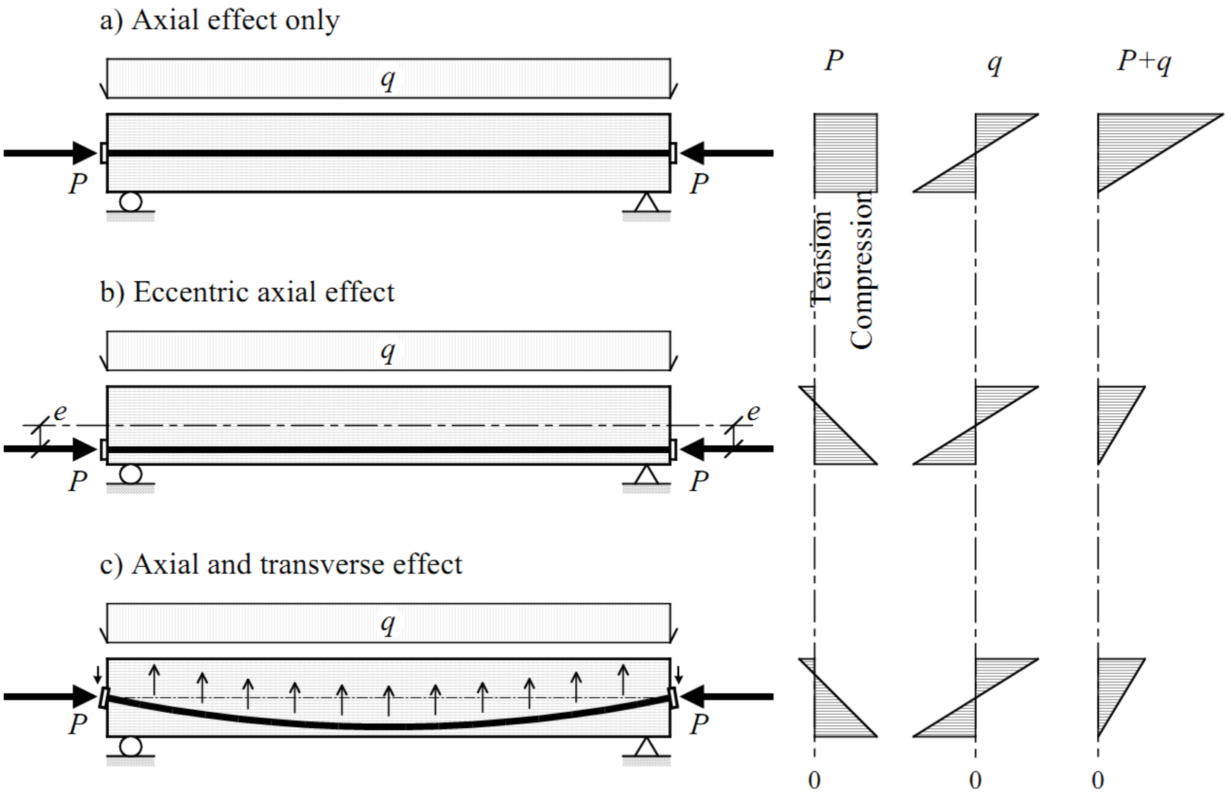


Fig. 2.4:- Illustration of prestress. a) The pure axial effect. b) Pre- tensioned member with eccentric straight tendons. c) Represents a post-tensioned member with curved tendons

The transverse effect of prestress will carry a certain part of the external load direction to the supports. For the remaining load, the structure will have an enhanced resistance to shear, punching and torsion due to compressive stresses from the axial effect. Prestress will also reduce deflections under service conditions, due to both the reduced effects of external load and the increase stiffness caused by delayed or eliminated cracking.

The fundamental and well- known advantages of prestressing can be summarized as the possibility to limit cracking and deformations in structure members with large spans, to reduce cross section dimensions for a given span and load and, finally, to increase the load capacity for given span and dimensions.

2.2.3 Main systems for prestressing

Prestressed members are classified in two main groups; pre-tensioned and post-tensioned. The member is called pre-tensioned, if the steel is stressed before casting the concrete. The member is called post-tensioned, if the steel is stressed after hardening of the concrete.

2.2.3.1 Pre-tensioned concrete

The Fig.1.5 below illustrates the procedure for pre-tensioning a concrete member. Such a procedure can be summarized as follows:

1. The prestressing tendons are initially tensioned between fixed rigid walls and anchored.
2. With the formwork in place, the concrete is cast around the stressed steel tendons and cured.
3. When the concrete has reached its required strength, the wires are cut (or released from the rigid walls).
4. As the tendons attempt to contract, the concrete is compressed. Prestress is transmitted through bond between the steel and the concrete.

Pre-tensioned concrete members are often precast in pre-tensioning yards that are usually long enough to accommodate many identical units simultaneously.

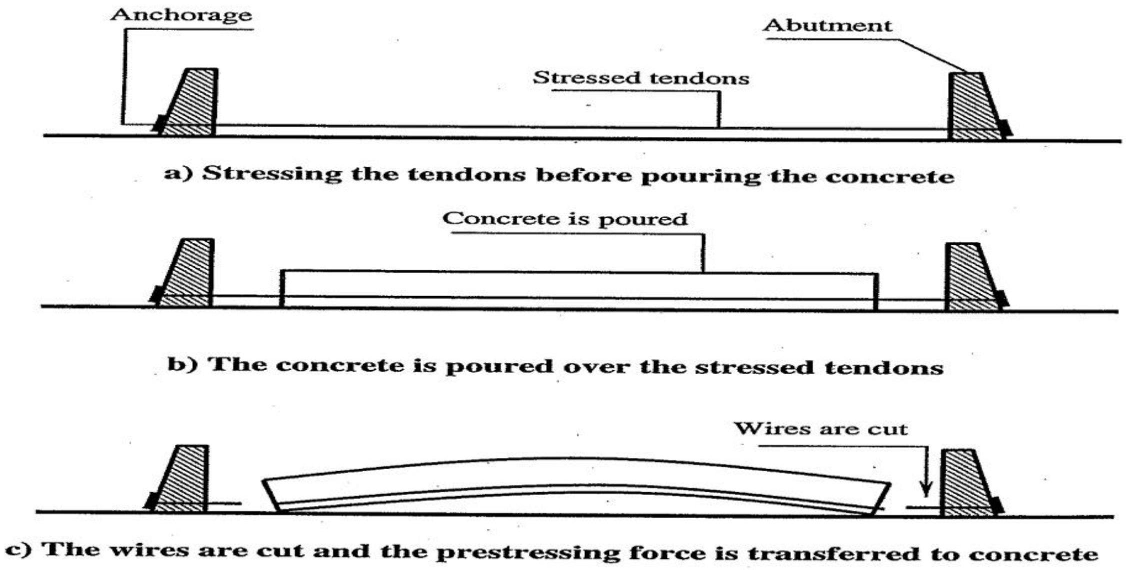


Fig.2.5:- A pre-tensioned beam during manufacturing

2.2.3.2 Post-tensioned concrete

The procedure for post-tensioning a concrete member is shown in the Fig. 1.6 below.

1. With the formwork in position, the concrete is cast around the hollow ducts, which is fixed to any desired profile.
2. The steel tendons are usually in place, unstressed in the ducts during the concrete pour, or alternatively may be threaded through the ducts later.
3. When the concrete has reached its required strength, the tendons are tensioned. Tendons may be stressed from one end with the other end anchored or may be stressed from both ends.
4. The tendons are anchored at each stressing end. In post-tensioning, the prestressing is maintained after the tendons are anchored by bearing of the end plates on to concrete. The ducts containing the tendons may be filled with grout under pressure. In this way, tendons are bonded to the concrete and are more efficient in controlling the cracks and providing ultimate strength. Bonded tendons are less likely to corrode.

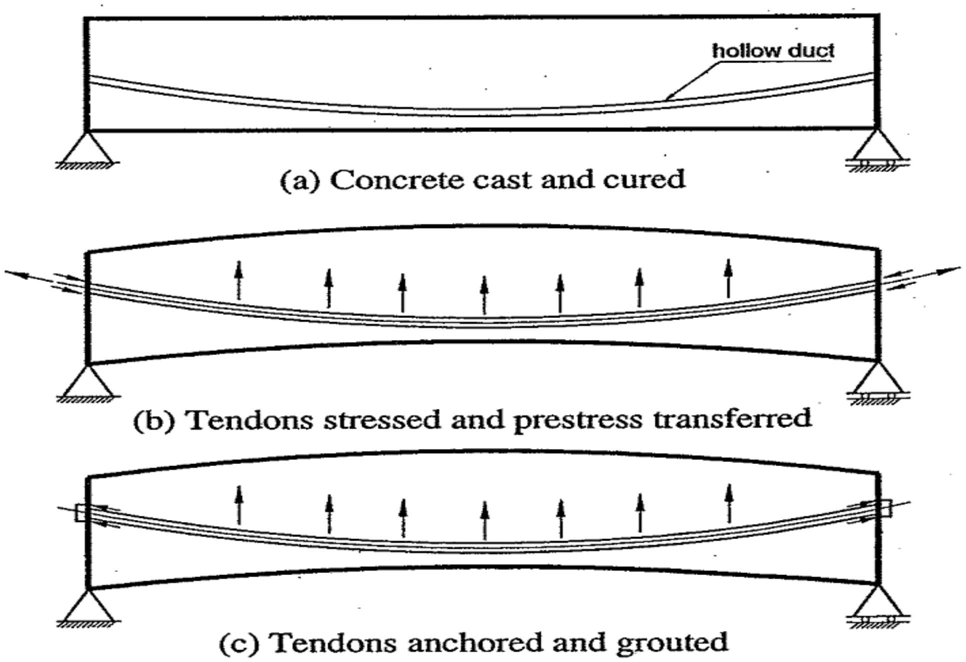


Fig.2.6:- A post-tensioned section

2.2.3.3 Bonded and unbounded systems

In thin slabs, for obvious reasons, tendons and anchorages must have small dimensions Post-tensioning with unbounded tendons offers some special advantages.

- Single strands tightly enclosed in plastic sheaths need less space than multi-strand tendons in ducts with room for grouting. They can therefore be placed closer to the surface.

- Large concrete covers are not needed for corrosion protection, since tendons have a built-in protection, and in principle no cover at all is needed for bond. However, conditions for fire protection are the same as for other reinforcement, and may sometimes govern the cover.
- Lighter stressing equipment and the absence of grouting simplifies execution.

In beams, the disadvantage of reduced eccentricity is normally less pronounced, due to the greater depth of cross-section. Bonded tendons can then be more economical than unbounded ones, since one tendon then consists of many strands, giving a higher prestressing force per anchorage.

In the comparison between bonded and unbounded systems, one should also consider the structural importance of bond, particularly in ultimate limit states. The advantages of unbounded system from this point of view must be taken into account in design.

CHAPTER THREE

POST-TENSIONED FLAT SLAB DESIGN AND CONSTRUCTION PRACTICE IN ETHIOPIA

3.1 General overview

Ethiopia is the fastest-growing, non-oil driven economy among African countries, the country has showed a remarkable growth over the past ten years. Real GDP growth was 10.9% in 2016/17 (IMF,2017).recently, the contribution of the industry sector which is 21.2% (UNDP,2017) and particularly that of the construction sector to the national economy is given high prominence and is mainly driven by the energetic performance of the construction sub-sector. Despite the construction sector has high importance, it is not improved and adopt new technology as compared with worldwide practice.

Every major metropolitan area like Addis Ababa is getting a facelift old buildings are being renovated and converted, while new construction continues to add to the skyline. Downtown living is making its resurgence, and with this booming office, shopping malls, and residential construction, post-tensioned flat plates are the structural system of choice.

Post-tensioned (PT) concrete floors are now widely used in worldwide, particularly for high-rise buildings. PT flat slabs provide the thinnest readily available structural option for spans of 7m or more, and can economically be used for spans up to 13mx13m. For longer spans, a one-way spanning slab onto band beams is frequently used.

In spite of the simplicity of its basic concepts and well known advantages, the application extent of post-tensioning solutions cannot be considered harmonized in the different areas and structural applications. In fact, for various reasons, it appears that the potential offered by presterssing is far from being exploited, especially in building structures field in Ethiopia.

The current study was conducted to access the level of design and construction practice of post tensioned flat slab in Ethiopia; to identify factors affecting the practice and to recommended possible mitigation measures. The study adopts quantitative and qualitative method with the help of primary and secondary data. Primary data was collected using self-administered questionnaires on 63 respondents from contractors, consultants and clients. Secondary data was collected through reviewing of related materials. Analysis of the quantitative data was made using equation 3.1 below.

3.2 Assessment Methodology

This research adopts a combination of both qualitative and quantitative methods. The primary data were obtained with the use of structured questionnaires (quantitative) and selected interviews (qualitative) for possible mitigation measures. The combination of quantitative with qualitative methods of data collection in research has become a common practice in recent years for greater understanding and validation of results (Brayman, 2016).

3.2.1 Sampling

The study was conducted in three stages: first to identify wither post-tensioned flat slab is used or not in Ethiopian and assess the knowledge and experience level of our professionals (Table 3.1); second to rank factors hindering the wide application of post-tensioned flat slab in buildings in Ethiopia from various stakeholders (consultant, contractor, client, and other engineering professionals) perspective (Table 3.2); and lastly to identify possible mitigation measures in order to use this technology widely in the country.

Professionals, who are currently working at the organizations namely East Africa Specialized Engineering (EASE), RAMA construction p.l.c, Asmelash ena Lechocu G.C, Afro-European Engineering P.L.C, GERETTA Consulting Architects and Engineering P.L.C and other responsible for the design, consulting, construction of buildings in the country, were selected to complete the questioner.

Table 3.1:- List of subjective questionnaires

Questions
1. How much do you know about post-tensioned slab design and construction? 2. Do you know the level of practice in Ethiopia? 3. Have you ever participated either design or construction of post-tension slab practice in you carrier? 4. Who do you think know better about it? 5. Who do you think the global practice of post tensioning concrete technology now a day? 6. Do you think we Ethiopians are not good in adopting new technology? 7. What do you propose for the wide application of prestressed concrete building design and construction in the country in the near future?

The factors in the questionnaire for respondents to rate qualitative questionnaires table 3.2 below shows different factors to serve as a questioner to rank the factors hindering the wide application of post-tensioned flat slab in buildings in Ethiopian construction sectors.

Table 3.2:- List of factors hindering the wide application of post-tensioned flat slab in buildings in Ethiopia

Factors hindering the wide application of post-tensioned flat slab in buildings.	**1**	**2**	**3**	**4**	**5**
1. Shortage of supply like high strength steel, cement, etc					
2. Unavailability of construction equipment					
3. Lack of professionals and expertise					
4. Lack of experienced consultant, contractor and other engineering professionals.					
5. Unavailability of design tools (like software, Design templates and guidelines)					
6. Economic condition					
7. Social influence (resistance against new technology)					
8. Universities has not been covered the topics deeply in under-graduate sections,					
9. Shortage of research literatures, seminars and related design examples.					
10. Not bring to a focus by Government as well as universities.					

1= very little; 2= little; 3=moderate; 4=high and 5= very high

3.2.2 Data analysis and Interpretations

The data collected through pre-tested structured questionnaire were categorized and analyzed. The data were analyzed and interpreted using equation 3.1 below. The five scales were converted to a Relative Importance Index (RII) for each individual factor using the following formula:

$$RII = \sum W \div (H + N) \tag{3.1}$$

Where, W is the total weigh given to each factor by the respondents, which ranges from 1 to 5 and is calculated by an addition of the various weightings given to a factor by the entire respondent, H is the highest ranking available (i.e. 5 in this case) and N is the total number of respondents that have answered the question.

3.3 Result and Discussion

This section presents the result and discussion regarding the level of post-tensioned flat slab design and construction practice in Ethiopian construction sector and the measures has been taken to mitigate the practice in to higher level.

3.3.1 Characteristics of Respondents

The respondents were categorized mainly into three groups, namely contractors, consultants and clients.

Table 3.3:- Response rate of the structured questionnaire

	Distributed	Returned	Valid	Valid among distributed in %	Remark
Contractors	31	27	27	87.09	OK
Consultants	21	15	15	71.42	OK
Clients	11	7	7	63.63	OK

Table 3.3 above indicate that the response rate for the questionnaire survey for contractors, consultants and clients are 87.09%, 71.42% and 63.63%, respectively. According to (Sekaran ,2001), a response rate of 50% is acceptable for most studies. Therefore, as the response rate of this study is more than what is referred as adequate by (Sekaran, 2001), the response rate was considered adequate for the study.

3.3.2 Respondents' response on factors hindering the wide application of post-tensioned flat slab in buildings in Ethiopia

The respondents were asked to rate with likert scale for factors hindering the wide application of post-tensioned flat slab in Ethiopia in construction sectors. The responses were collected with the help of structured questioner of 10 factors.

Table 3.4:- Response for factors hindering the wide application of post-tension flat slab in buildings

No	Response for hindering the wide application of post-tensioned flat slab in buildings.	RII
1	Shortage of supply like high strength steel, cement, etc.	1.80
2	Unavailability of construction equipment.	2.00
3	Lack of professionals and expertise.	2.90
4	Lack of experienced consultant, contractor and other engineering professionals.	3.97
5	Unavailability of design tools (like software, Design spreadsheets and guidelines).	3.50
6	Economic condition.	2.80
7	Social influence (resistance against new technology).	
8	Universities has not been covered the topics deeply in under-graduate sections.	3.00
9	Shortage of research literatures, seminars and related design examples.	3.50
10	Not bring to a focus by Government as well as universities.	2.00

As per the respondents, the top five factors identified hindering the wide application of post-tensioned flat slab in buildings in Ethiopia are Lack of experienced consultant, contractor and other engineering professionals (3.97), Unavailability of design tools (like software, Design templates and guidelines) (3.5), Shortage of research literatures, seminars and related design examples(3.5), Universities has not been covered the topics deeply in under-graduate sections(3.0) and Economic condition(2.8) (See Figure 3.1 and Table 3.4).

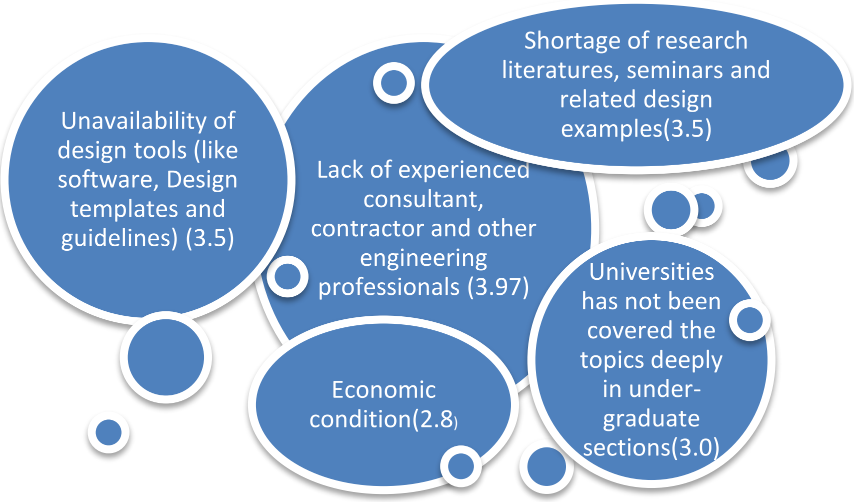


Fig.3.1:- Top five factors contributing not using post-tension flat slab in Ethiopia

3.3.3 Current Practice and Their Implications

It's been few years since post-tension system has begun to be applied in earnest to buildings in Ethiopia. In the meantime, post-tension system has been used in few buildings in Addis Ababa as main structural system including tall buildings. The post-tensioned building market of Ethiopia has been slowly grown in recent years with such advantages. Recently, post-tensioned technology is adapted for buildings; the authors would like to explain design and construction of tall buildings in Ethiopia using post-tensioned slab technology.

This sub topic introduces state of prospects of post-tensioned slab technology in Ethiopia through introducing few tall buildings where post-tension flat slab technology has been applied.

3.3.3.1 U-Street building

U-Street building is a mixed tall building with 15 stories above ground and 3 basement floors has been under construction at the backside of Friendship building along Bole road in Addis Ababa.

As project Coordinator mr.Kriss.D explained, the reason way the client needs to redesign the floor system to post-tensioned flat slab without perimeter beams over other types of slabs are: - Long span and high ceiling height within limited story height, minimize reinforcement steel and concrete amount and increasing the floor number within existing foundation system.

Fig. 3.2:- post-tensioned flat slab and Anchorage device at Bole around Friendship complex

3.3.3.2 ENE SHITA MINALE MIXED USE BUILDING

ENE SHITA MINALE MIXED USE BUILDING is a mixed building located in Addis Ababa around Uraile church it has 19 stories above ground and 3 basemen floors. Originally, it was designed as reinforced concrete solid slab floor system then, the design is redesigned to post-tensioned flat slab without perimeter beam by East Africa Specialized Engineering (EASE) consulting p.l.c in order to obtain the optimum construction cost

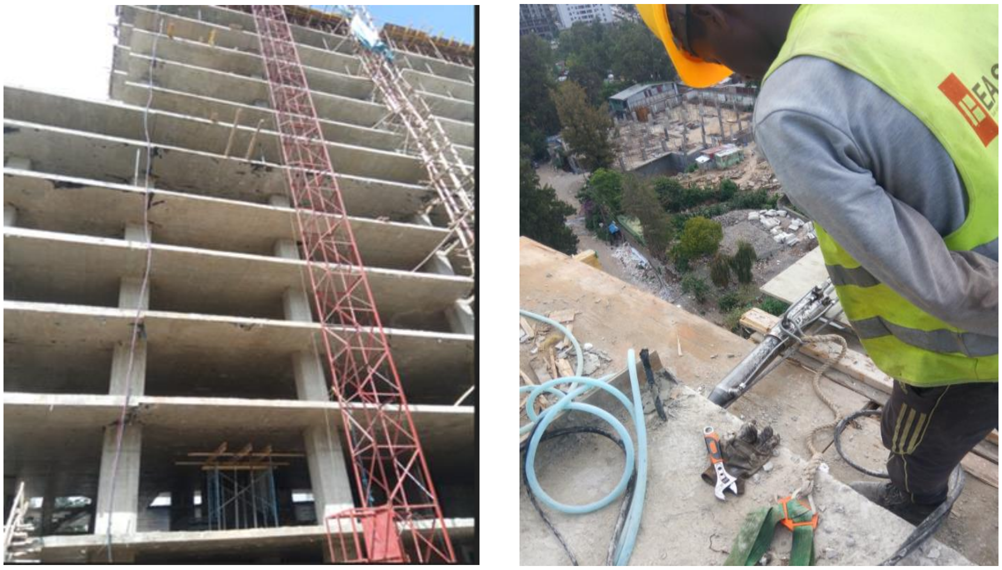


Fig. 3.3:- Post-tensioned flat slab building and Tensioning process at Kasanchis around Uraile Church

3.4 DISCUSSION

Introducing a new technology in construction industry everywhere faces problems and challenges. However, in Ethiopia, these difficulties and challenges are present alongside a general situation of socio-economic stress, chronic resource shortages, institutional weakness and a general inability to deal with the new technology.

The data collected through questionnaires (Table 3.1) using semi-structured question guide was meant to make suggestions for mitigating the design and construction practice of post-tensioned flat slab discussed above.

Their responses may be summarized as follows:-

1. The majority of them did not do any design of prestrssed concrete building, except some of them knew only the basic principles. Similarly, a few of them have the exposure and the general knowledge regarding the advantages of presterssed concrete construction methods.
2. All did not clearly specified which organization or individual groups have a potential knowledge of presterssed concrete design and construction.
3. An appreciable work is not done in the design and construction of post-tensioned flat slab concrete in buildings.

The study suggests that measures shall be taken for the top identified factors hindering the wide application of post-tensioned flat slab in our construction industry widely like; Universities has not been covered the topics deeply in under-graduate sections, Shortage of research literatures, seminars and related design examples, Lack of experienced consultant, contractor and other engineering professionals and Lack of professionals and expertise.

In respect to the future proposal, many of them agree that at least the following has to done:-

- ➢ The merits and demerits of post-tensioned flat slab concrete design and construction have to be assessed.
- ➢ Engineers have to be given more training and short courses.
- ➢ The main organizations and universities responsible for the realization of post-tensioned concrete have to take the major task in assessing the possibilities and means of applications of the system.
- ➢ Continuing the efforts started with respect to giving design and construction works in association with the local consultants and contractors in the sense of technology transfer in post-tensioned concrete systems.

The recent intense construction experience has provided fertile ground for the development of innovative and practical solutions to enhance the acceptability of these slabs in the construction industry. A key factor in the successful use of PT slab in the future is early involvement with buildings designers to ensure that structural details are compatible with the PT slab system.

PT slabs present an excellent vehicle for designers and contractors to provide their client with maintenance free, high performance floor which live up to expectations. It is then author's expectation that we will see more extra dosed post-tensioned flat slab building construction in the near future.

CHAPTER FOUR

DESIGN CONSIDERATION

4.1 General Design Considerations

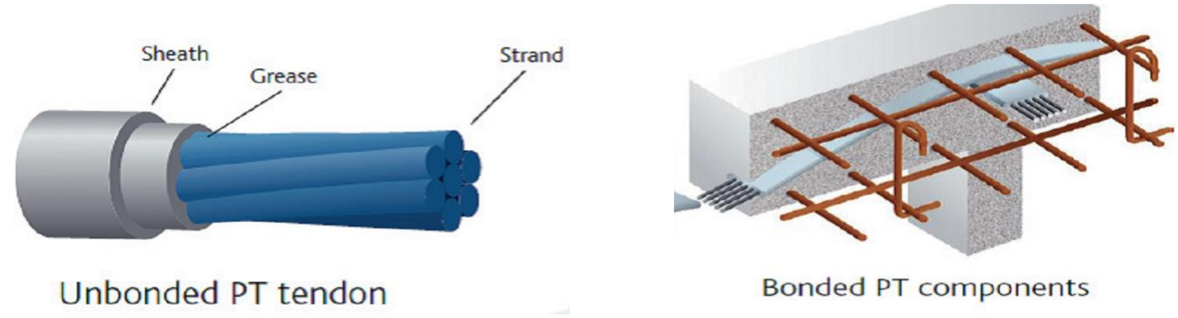
Every major metropolitan area like Addis Ababa is getting a facelift old buildings are being renovated and converted, while new construction continues to add to the skyline. Downtown living is making its resurgence, and with this booming office, shopping malls, and residential construction, post-tensioned flat plates are the structural system of choice.

Post-tensioned (PT) concrete floors are now widely used in worldwide, particularly for high-rise buildings. PT flat slabs provide the thinnest readily available structural option for spans of 7m or more, and can economically be used for spans up to 13mx13m. For longer spans, a one-way spanning slab onto band beams is frequently used.

4.1.1 The post tensioning system

The most common post-tensioning system for two-way slab building construction uses mono-strand unbounded tendons. In this type of construction, the prestressing steel is composed generally of high-strength, single-wire steel, wrapped with another six wires to form a seven-wire strand. By design unbounded tendons have a continuous plastic sheathing to prevent the strand from bonding with the concrete along its length and limits intrusion of corrosive elements. Corrosion inhibiting grease coats the strand to reduce friction between the strand and the sheathing during stressing.

There are two types of post-tensioning system available to the engineer (fib, 1980): bonded and un-bonded (fig.4.1). Most of the post-tensioning work in worldwide is bonded, being about 90% of the market. Both systems can be used in the same slab if the design dictates it. Table 4.1 gives a comparison between the two systems.



Unbonded PT tendon

Bonded PT components

Fig.4.1:- Bonded and unbonded PT tendons

Table 4.1:- Comparison of PT systems

Bonded	Unbonded
➢ Localizes effect of accidental damage ➢ Develops higher ultimate strength ➢ Does not depend on anchorages after grouting ➢ Can be demolished in same way as reinforced concrete	➢ Reduced covers to strand ➢ Reduced prestressing force ➢ Tendons can be prefabricated leading to faster construction. ➢ Tendons can be deflected around obstructions more easily. ➢ Greater eccentricity of strand. ➢ Grouting not required. ➢ Useful when only strand is required, e.g. in rib in ribbed slab.

The force in a stressed tendon is transferred to the concrete via serrated wedges that lock into anchor plates provided at its ends. Anchors are classified as either live (stressing) ends or dead ends. Dead end anchors are embedded into the concrete and will not be stressed. These anchors are mounted to the tendon at the fabrication.

4.1.2 Restraint

At the early stages of a project using PT floors, care must be taken to avoid the problems of restraint. This is where the free movement in the length of the slab under the prestress forces is restrained, e.g. by the unfavorable positioning of shear walls or lift cores (Fig.4.2).

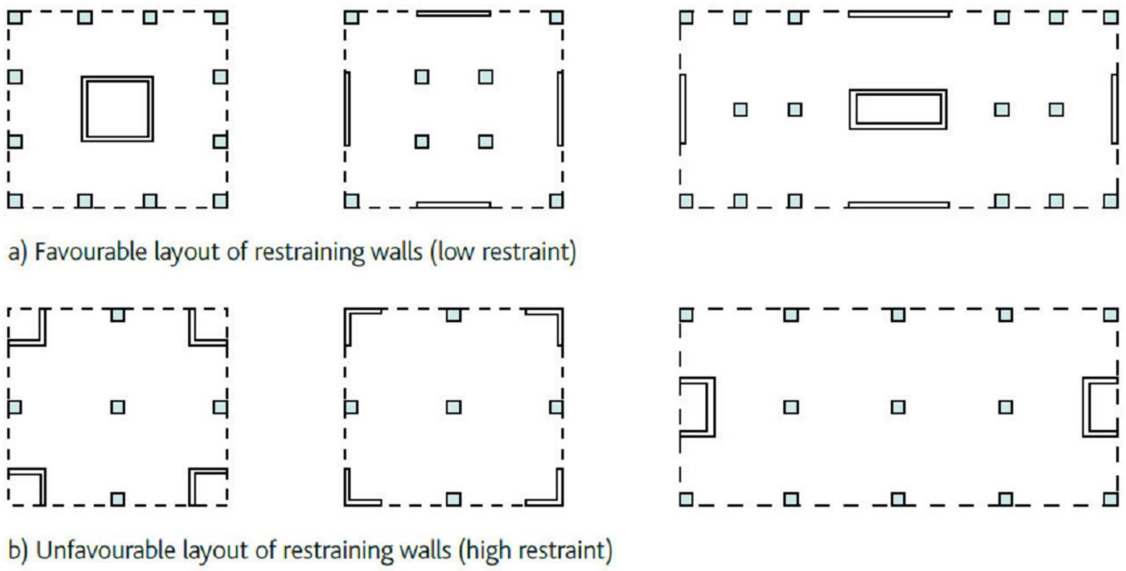


Fig.4.2:- Typical favourable and unfavourable layout of restraining walls

All concrete elements shrink due to drying and early thermal effects but, in addition, prestressing causes elastic shortenings and ongoing shrinkage due to creeps. Stiff vertical members, such as stability walls, restrain the floor slab from shrinking, which prevents the prestress from developing and thus reduces the strength of the floor. Where the walls are unfavorably arranged, a calculation of the effects of movement should be carried out and suitable measures taken to overcome them. (See Excel design template part 9.3) this could involve:

- Using infill strips, also known as pour strips, which are usually cast around 28 days after the remainder of the floor, to allow initial shrinkage to occur.
- Increasing the quality of conventional reinforcement, to control the cracking.
- Using temporary release details.
- Using a proprietary temporary release detail.
- Reducing the stiffness of the restraining elements.

4.1.3 Design process

The figure 5.3 presents a flow chart for the design of PT flat slab. Recommendations for the design of prestressed concrete are given in Ethiopian code (ES-EN-1992:2015). Design methods for PT flat slabs are relatively straightforward and the design procedures are as follows. (See ANNEX 1).

- Assume a set of loads to be balanced by parabolic tendons.
- Analyze an equivalent frame subjected to the net downward loads.
- Check flexural stresses at critical sections, and revise load balancing tendon forces as required to obtain permissible flexural stresses.
- When the final forces are determined, obtain frame moments for factored dead and live loads.
- Calculate secondary moments induced in the frame by post-tensioning forces, and combine with factored load moments to obtain design factored moments,
- Provide minimum bonded reinforcements in accordance with the code.
- Check design flexural strength and increase non-prestressed reinforcements if required by strength criteria,
- Investigate shear strength, including shear due to vertical load and due to moment transfer, and compare total to permissible values.

At the serviceability condition (SLS), the concrete section is checked at all positions to ensure that both the compressive and tensile stresses lie within the acceptable limits given in the code. Stresses are checked in the concrete section at the initial condition when the prestress is applied, and at serviceability conditions when calculations are made to determine the deflections for various load combinations.

At the ultimate limit state (ULS) the pre-compression in the section is ignored and checks are made to ensure that the section has sufficient moment capacity. Shear stresses are also checked at the ULS in a similar manner to that for reinforced concrete design, although the benefit of the prestress across the shear plane may be taken in to account.

4.1.4 Analysis

Computers have increased the speed of post-tensioning design significantly, but it is still important to understand the concepts and calculations to arrive at an accurate output. When performing manual calculations, the equivalent frame method (EFM) often is used for the structure analysis of post-tensioned flat slab structure, EFM models a 3-D slab system as a series of equivalent 2-D frames along the supports lines, taken longitudinal and transversely through the structure (Lin,T.Y.,1963,pp. 719-742). Each equivalent frame then can be analyzed individually as an isolated plane frame, consisting of a row of columns or supports and the corresponding tributary slab panels.

The analysis and design of post-tensioned flat slabs incorporate the full tributary slab width without the distribution of forces and reinforcement between column strips and middle strips, synonymous with mild reinforced slab designs. This allowance eases the structural engineer’s design process and ultimately simplifies construction.

4.1.5 Preliminary sizing

Before design can begin on the structure there needs to be a starting point for the slab thickness. For common occupancy structures with live load to dead load ratio less than 1.0, a preliminary slab thickness can be estimated using a longest span to thickness ratio of 45 for floors and 48 for roofs. Table 4.2 gives span/depth ratios recommended by the post-tensioning institute of the USA. (ACI-ASCE, 1974)

Post-tensioned slabs can be used their own or combined with reinforced concrete to provide a range of in-situ concrete floor options. The slabs can be one-way or two-way spanning depending on the circumstance, in general if the aspect ratio length/width ≤ 2, then one can assume two-way spanning.

Table 4.2:- Typical span-to-depth ratio (ACI-ASCE, 1974)

Type of slab	Span/depth ratio
➢ One-way slab	48
➢ Two-way slab	45
➢ Two-way slab with drop panel (maximum drop panel span/6 each way)	50
➢ Two-way slab with two-way beams	55
➢ Waffle (5*5 grid)	35
➢ Beams b = h/3	20
➢ Beams b = 3*h	30

Because of post-tensioning’s ability to balance loads and greatly reduce service load deflections, there is a 25% to 35% reduction in slab thickness in post-tensioned structures compared with mild reinforced structures. Therefore, in addition to the ability to span further, the reduced structure depth economized materials quantities for the slabs and consequently the columns and foundations.

4.1.6 Design for prestressing

The amount of prestressing in a slab system is guided by parameters and requirements given in ES-EN-1992:2015 as well as other references, but the engineer has flexibility in adjusting the design for optimization of an individual project.

Tendons for building construction usually are placed with a parabolic vertical profile to counteract a portion of the gravity loads on the structure (Fig.4.3). this undulating profile places the center of gravity, CGS of the tendon force P, eccentric to the neutral axis (e) of the concrete section creating the primary moment, P*e.

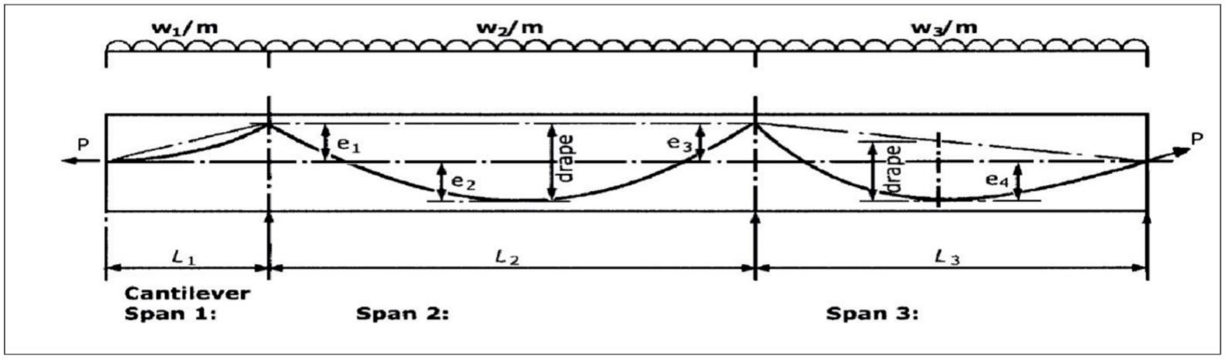


Fig 4.3:- Idealized tendon profile

The high and low points are governed by several parameters. The ends of the tendons usually are located at the section's neutral axis (mid-height for a slab), so as to not induce additional moment at the anchors.

In building construction minimum cover requirement per ES-EN-1992:2015 are sufficient for slabs not exposed to a corrosive environment and fire protection often governs the minimum concrete cover of these structures.

When the design is performed manually, the post-tensioning force typically is selected to balance a specified percentage of the floor self- weight. Superimposed dead loads such as partitions, flooring, mechanical equipment, and live loads are not included, since they are not present at the time of prestressing. Common load balancing percentages are in the 65% to 80% range and should be kept relatively consistent between spans, but engineers still need to design to appropriate balancing loads to limit slab deflections and cracking.

The load balancing effect reduces the amount of flexural stress for ultimate requirements, helping economize member sizes and materials. Another advantage is the significantly reduced deflections. These advantages are significant, but excessive use of post-tensioning can be detrimental. Note that it is not considered conservative to over-balance the gravity loads or over-pre-compress the section because unwanted camber and excessive cracking may occur. It is imperative to remember that post-tensioning is active reinforcement, exerting its load for the life of the structure.

The effect of prestressing force on a member can be evaluated by replacing the tendon with equivalent externally applied loads. The designed force in the tendons P, will be a function of the designer specified equivalent balancing load w_b, the span length l, and its associated maximum drape, f. for a simply supported determinate span with a parabolic tendon profile, the formula for balancing a uniformly distributed load is

$$P = \frac{W_b l^2}{8f} \tag{4.1}$$

For a multi span indeterminate structure, the design process has additional considerations. Since the eccentricity and length may vary between spans, the prestressing force P needs to be determined for each span. The greatest force P_{max} typically is selected for the entire equivalent frame, but code requirements and other guidelines may influence the final effective force.

Post-tensioning design is an iterative process to determine an optimized solution. From the selected equivalent frame force, the resulting balancing load in each span must be checked to ensure the percentages are within the range selected by designer. The percentage is determined from:

$$\frac{w_b}{w_{DL}}, \text{ Where, } \frac{8 p_{\max} f}{l^2} \tag{4.2}$$

If the balanced loads are above the acceptable limits in a given span, in lies of adjusting the force, the engineer has option to alter the tendon drape, f, to reduce the balanced load w_b. It is more efficient to alter the drape by raising the bottom tendon ordinate while maintaining the top ordinates. By doing so, the only change in the construction process is to provide higher supporting chair heights, instead of altering the entire top steel configuration at the supports.

4.1.7 Prestress losses

Although unbounded tendons can be stressed to the force per ES-EN-1992:2015, they will not retain this maximum force for the life of the structure. The force calculated from the balancing load is an effective value is found after calculating prestress loses. Instantaneous prestress losses arise from the seating of the wedges into the anchor, elastic shortening of the concrete, and friction along the length of the tendon. The long-term stress losses are caused by creep and shrinkage of the concrete and relaxation of the prestressed steel. Further explanations of prestress losses can be found in sub topic 4.3.

4.1.8 Additional design parameters and required

With the required effective tendon force calculated, the following code parameters and requirements may influence the final design values.

- Limitation of the average prestress:- For slabs not exposed to corrosive environment, the concrete compressive stress in the structure resulting from the prestressing force and other loads acting at time of tensioning or release of prestress, should be limited to as ES-EN-1992:2015.

$$\sigma_c \leq 0.6 \times f_{ck}(t) \qquad (4.3)$$

- Limitation for service load stresses:- The ES-EN-1992:2015 provisions limit tensile stresses in the concrete to control the development of flexural cracking. The code classifies flexural members based upon computed extreme fiber stress in tension f_t at service loads as class U (un-cracked), class T (Transition) or class C (Cracked). Depending upon the classification, members are assumed to behave as cracked or uncracked sections for service loads stress and deflection calculations. Per ES-EN-1992:2015 prestressed two-way slabs are to be designed as class U with $f_t \leq 6\sqrt{f'_c}$, Where, f_c is the specified concrete strength. This requirement permits service load stresses to be computed using uncracked section properties.

Generally, the most economical design for flexural strength will be obtained by using the maximum permissible tensile stresses. Some limitations may result from serviceability restriction, such as deflection.

Table 4.3 Permissible stresses in concrete in prestressed flexural members (H.Nilson, David Darwin and Charles W.Dolan, "Design of Concrete structures".)

Condition	Class		
	U	T	C
a. Extreme fiber stress in compression immediately after Transfer (except as in b.)	$0.60f'_{ci}$	$0.60\ f'_{ci}$	$0.60\ f'_{ci}$
b. Extreme fiber stress in compression at ends of simply supported members	$0.70\ f'_{ci}$	$0.70\ f'_{ci}$	$0.70\ f'_{ci}$
c. Extreme fiber stress in tension immediately after transfer (except as in d)	$3\sqrt{f'ci}$	$3\sqrt{f'ci}$	$3\sqrt{f'ci}$
d. Extreme fiber stress in tension immediately after transfer at ends of simply supported members	$6\sqrt{f'ci}$	$6\sqrt{f'ci}$	$6\sqrt{f'ci}$
e. Extreme fiber stress in compression due to prestress plus sustained loads	$0.45\ f'_{ci}$	$0.45\ f'_{ci}$	_
f. Extreme fiber stress in compression due to prestress plus total loads	$0.60f'_{ci}$	$0.60f'_{ci}$	_
g. Extreme fiber stress in tension f_t in recompressed tensile Zone under service load.	$\leq 7.5\sqrt{f'ci}$	$>7.5\sqrt{f'ci}$ and $\leq 12\sqrt{f'ci}$	_

- Limitation for initial stresses:-Compressive and tensile stresses also needed to be checked at the time of stress transfer, also referred to as the initial stage. These stresses typically are computed with self -weight and the forces induced by the prestressing, since live loads and superimposed dead loads will not be on the structure during the stressing procedure. The compressive stresses shall be no greater than $0.6f'_c$, and the tensile stresses are not to exceed $3\sqrt{f'_c}$. If the tensile stresses exceed this limit, additional mild reinforcement needs to provide to resist the total tensile force in the concrete.(ACI Committee 423,1974)

4.2 Bases of Design Principles

This clause deals with general design considerations, such as tendon layout and structural effects of prestress losses, serviceability and ultimate limit states, anchorages and restraint from adjacent structure components, with focus on particular aspects for buildings. (FIP, 1980,21pp)

4.2.1 Structural effects and tendon profiles

Prestress has two effects:

- The axial effect, causing compression in the concrete, with favorable effects on cracking and deflections and also contributing to shear, torsion and punching resistances.
- The transverse effect causing by deviation forces, directly carrying part of the external load to the supports.

With an appropriate tendon layout, the transverse forces can more or less balance part of the external load. A simple example is shown in the fig. 4.5 for best efficiently, the tendon curve should correspond to the bending moment diagram as far as possible. The transverse forces will than have the same distribution as the external load.

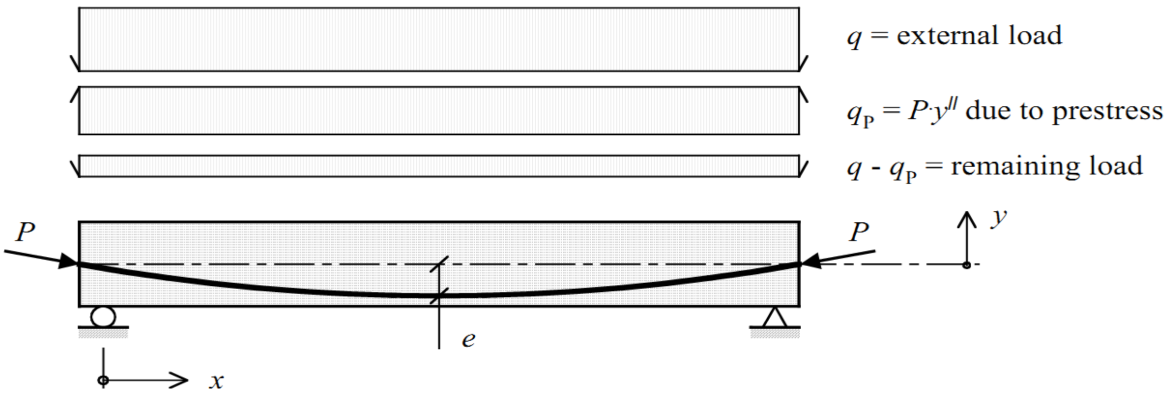


Fig. 4.4:- The transverse effect

Where, P = prestressing force, $q_p = P*y''$ = transverse load due to prestress, e = eccentricity of prestressing force, $y'' = d^2y/dx^2$ = tendon curvature.

In continuous members, the minimum curvature required for the tendon, will necessitate a certain distribution of the downward transverse effect over support. This means that the external moment diagram cannot be completely followed, see fig.4.5 below. However, this is not a major problem as far as the global behavior is concerned.

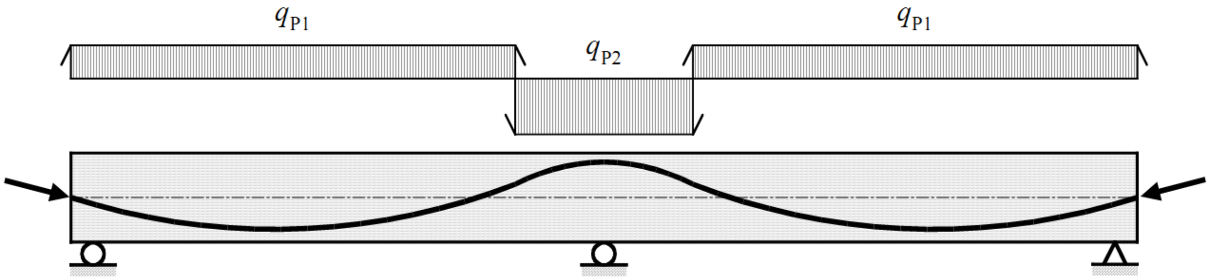


Fig. 4.5:- Tendon layout in a continuous member

Normally, tendons are located centrically at simply supported ends. If not, the eccentricities will give end moments M_p = P*e that should be added to the effect of the transverse load q_p, taking into account boundary conditions if the structure is hyper static. See Fig. 4.6 the effect of the eccentricities can also be seen as the “eccentric axial effect”.

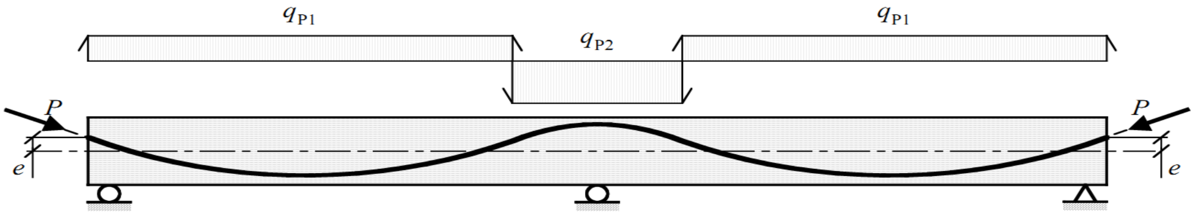


Fig.4.6:- Effect of end eccentricities

End eccentricities can be used to enhance a certain effects of prestress. Thus, an upward end eccentricity as shown in Fig.4.6 is favorable with regard to shear, whereas a downward end eccentricity will reduce deflections.

If there are large concentrated loads with fixed position, the tendons can be bent in a consternated curvature at these positions, and be more or less straight in between. This gives concentrated lifting forces to directly balance (part of) the external loads. See fig. 4.7.

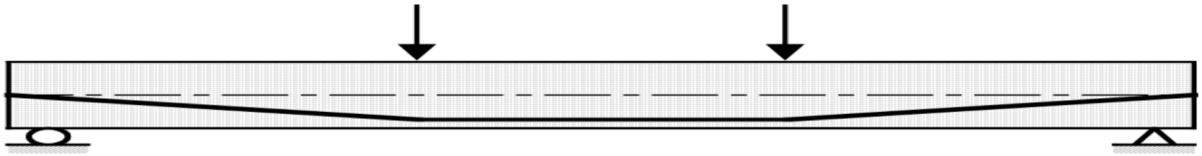


Fig. 4.7:- Tendons layout with straight parts

A tendons layout with straight parts is sometimes used also for practical reasons, even if there are no concentrated loads parts of the tendons can then be supported directly on the bottom reinforcement, which simplifies execution, especially in slabs.

4.2.3 Materials

4.2.3.1 Concrete

To take the full advantage of prestressed concrete with high compressive strength is usually used. The compressive strength of concrete at an age t depends on the type of cement, temperature, and curing conditions. For mean temperature of 20^{o}c and curing in accordance with the code ES-EN-1992:2015 the compressive strength of concrete at various ages $f_{cm}(t)$ may be estimated from.

$$f_{cm}(t) = \beta_{cc} f_{cm}, \text{ with } \beta_{cc}(t) = e^{\left\{s\left(1-\frac{28^{0.5}}{t}\right)\right\}} \qquad (4.4)$$

Where: $f_{cm}(t)$ is the mean concrete compressive strength at an age of t days.

f_{cm}: - is the mean compressive strength at 28 days according to table 3.1 from ES-EN-1992:2015

$\beta_{cc}(t)$: - is a coefficient which depends on the age of the concrete.

t:- is the age of the concrete in days.

S: - is a coefficient which depends on the type of cement.

The development of tensile strength with time is strongly influenced by curing and drying conditions as well as by the dimensions of the structural members. The tensile strength of concrete is $f_{ctm}(t) = \beta_{cc}(t)^{\alpha}.f_{ctm}$

Where: $\beta_{cc}(t)$ follows from the above expression (4.4)

$\alpha = 1$ for $t < 28$

$\alpha = 2/3$ for $t \geq 28$, the value of f_{ctm} are given table 3.1 in ES-EN-1992:2015

The advantage of using high-strength concrete in prestressed concrete construction can be summarized as:

- Due to its speed in gaining strength, the shattering can be removed faster reducing time and cost.
- It minimizes losses in prestressing forces by reducing creep, elastic shortening and shrinkage.
- It reduces the size and weight of the member.

- It reduces the required area for shear reinforcement.
- It produces the high bond strength required to anchor the strands used in pre-tensioned construction.

4.2.3.2 Prestressing Reinforcemen.

Prestressing reinforcement is available in different forms such as cold drawn wires, cables, and alloyed steel bars. The most common type of prestressing reinforcement is the seven wire strands cable. The ultimate tensile strength of this cable is seven times that of non-prestressing reinforcement.

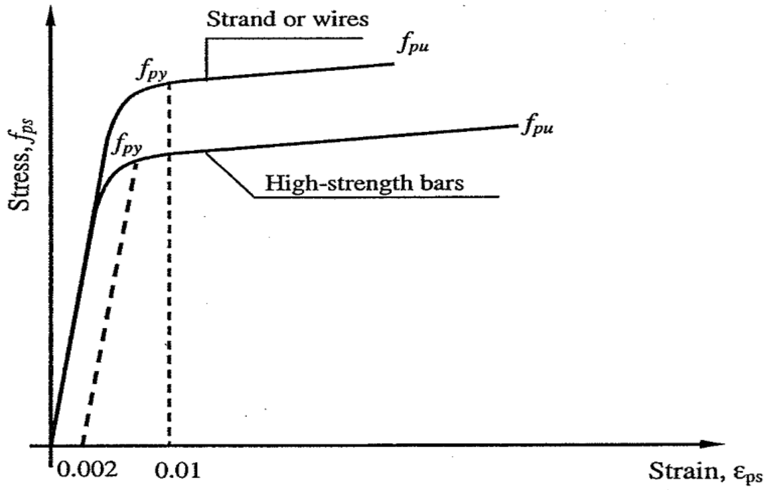


Fig. 4.8:- stress and deformation characteristics for concrete

A typical stress- strain relation for prestressing reinforcement is clear that the prestressing steels lack a sharply defined yield point. Therefore, most codes including ES-EN-1992:2015, specifies the yielding point as the stress associated with a 1% strain, for high strength bars, the yield strength is frequently specified as the stress associated with the intersection of the curve and a line parallel to the initial slope starting at strain of 0.002. The yield stress f_{py} for stress relieved steel equals to approximately 85% of f_{pu} and equal to 90% of f_{pu} for low relaxation steel.

The design value for the modulus of elasticity, E_p may be assumed equal to 205Gpa for wire and bars. The actual value can range from 195Gpa to 210Gpa, depending on the manufacturing process. And the design value for strand is equal to 195Gpa. See table 4.4. (Michael p. Collins, Denis Mitchell, 1997)

Table 4.4:- maximum permissible stresses in steel(H.Nilson, David Darwin and Charles W.Dolan, "Design of Concrete structures".)

Maximum permissible stress in prestressing steel	Stress
1. Due to tendon jacking force but greater than the Lesser of $0.8f_{pu}$ and the maximum value Recommended By the manufacturer of the prestressing steel.	$0.94f_{pu}$
2. Immediately after prestress transfer but not greater than $0.74f_{pu}$	$0.82f_{pu}$
3. Post-tensioning tendons, at anchorage devices and couplers, Immediately after tendon anchorage.	$0.70f_{pu}$

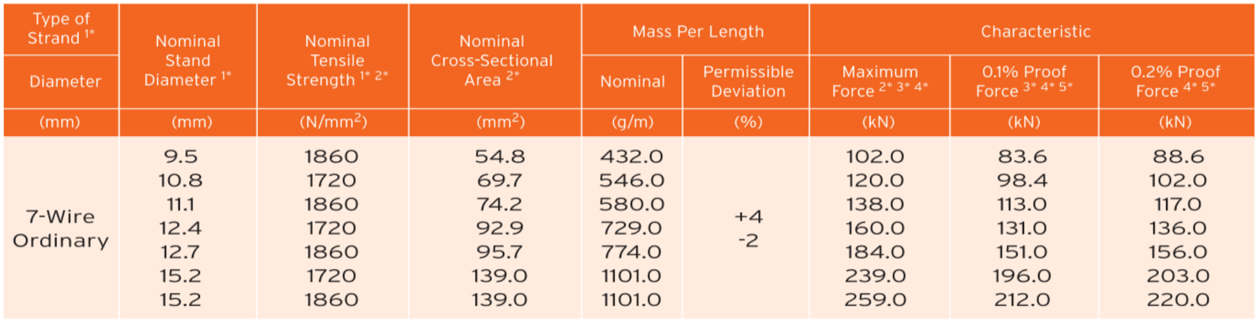
4.2.3.3 Prestressing force

The maximum prestress is limited to values, which are given in codes as an example the following stress values can be mentioned in ES-EN-1992:2015:

- Maximum stress during tensioning : $\sigma_p \leq 0.90f_{yk}$ and $\leq 0.8f_{uk}$
- Maximum stress after tensioning and anchoring: $\sigma_p \leq 0.85f_{yk}$ and $\leq 0.75f_{uk}$

Here f_{yk} and f_{uk} are the characteristic values of yield and ultimate tensile strength respectively. For common types of strands used in the SPEC-ISO 6934-4:1991(E): Dimension, mass and Tensile properties of 7 wire strand are given it table 4.5. It is recommended that only one of these strand types is used on any project.

Table 4.5:- Specification of commonly used strand (SPEC-ISO 6934-4:1991(E)



Type of Strand ^{1*}	Nominal Stand Diameter ^{1*}	Nominal Tensile Strength $^{1*\,2*}$	Nominal Cross-Sectional Area ^{2*}	Mass Per Length		Characteristic		
Diameter				Nominal	Permissible Deviation	Maximum Force $^{2*\,3*\,4*}$	0.1% Proof Force $^{3*\,4*\,5*}$	0.2% Proof Force $^{4*\,5*}$
(mm)	(mm)	(N/mm^2)	(mm^2)	(g/m)	(%)	(kN)	(kN)	(kN)
7-Wire Ordinary	9.5	1860	54.8	432.0	+4 -2	102.0	83.6	88.6
	10.8	1720	69.7	546.0		120.0	98.4	102.0
	11.1	1860	74.2	580.0		138.0	113.0	117.0
	12.4	1720	92.9	729.0		160.0	131.0	136.0
	12.7	1860	95.7	774.0		184.0	151.0	156.0
	15.2	1720	139.0	1101.0		239.0	196.0	203.0
	15.2	1860	139.0	1101.0		259.0	212.0	220.0

Note:-

1. The type of strand, Nominal diameter and Nominal tensile strength are for designation purposes only
2. The nominal tensile strength is calculated from the nominal cross section area and the specified characteristic maximum force
3. No single test result shall be less than 85% of the specified characteristic value.
4. Considering the small tolerance on mass per length, characteristic forces have been specified rather than stresses.

5. The 0.1% proof force is mandatory and the 0.2% proof force is for information only (see ISO 6934-1), except when otherwise agreed.

4.3 Loss of Pre-stressed forces

4.3.1 Introduction

The applied prestressing force after jacking undergoes a number of reductions, some of these reductions occur immediately and others occur over a period of time. Therefore, it is important to establish the level of prestressing at each loading stage as shown below.

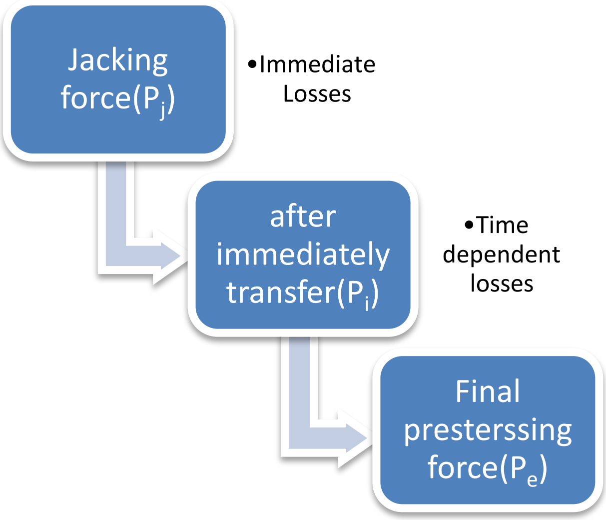


Fig.4.9:- Level of prestressing losses

Presteressing losses can be categorized in two groups:

- Immediate losses:- these are losses that occur during fabrication including elastic shortening, anchorage loss, and frictional losses.
- Time dependent losses:- these are the losses that increase over time including creep, shrinkage and steel relaxation.

4.3.2 Elastic shortening losses

During tensioning the concrete is subjected to compression and a corresponding shortening. If there are several tendons which cannot tensioned at the same time, the force in tendons already tensioned will increase each time another tendon is tensioned.

The average loss can related to half the total prestress. The concrete shortening ε_c and the corresponding loss of prestress $\Delta\sigma_{cp}$ is then

$$\varepsilon_c = \frac{0.5P}{A_c E_c} = \frac{0.5\sigma_c}{E_c}, \quad \Delta\sigma_{cp} = \varepsilon_c E_c = 0.5\frac{E_s}{E_c}\sigma_c = 3\sigma_c \qquad (4.5)$$

Where:- P total prestressing force

A_c - concrete area

$\sigma_c = P/A_c$

E_c Elastic modulus of concrete

E_s Elastic modulus of steel

Account should be taken of the loss in tendon force corresponding to the deformation of concrete, taking account the order in which the tendons are stressed.

The loss ΔP_{el} from ES-EN-1992:2015 may be assumed as a mean loss in each tendon as follows:

$$\Delta P_{el} = A_p E_p \sum \left[\frac{j\Delta\sigma_c(t)}{E_{cm}(t)} \right] \tag{4.6}$$

Where:- $\Delta\sigma c(t)$ is the variation of stress at the center of gravity of the tendons applied at time t.

j is a coefficient equal to = (n-1)/2n, where n is the number of identical tendons successively prestressed. As an approximation this may be taken as 0.5and 1.0 for the variations of permanent actions applied after prestressing.

For post-tensioned slabs in building, where typically σc = 1.5Mpa, the loss will be about 5Mpa, which is quite negligible.

4.3.3 Losses due to friction

The prestressing force decrease with increased distance from the active end due to friction, the variation of the force then follows from:-

$$P(t) = P_o e^{\{-\mu(\sum\theta + kx)\}} \tag{4.7}$$

Where:- p_o – prestressing force at the active end.

μ - Coefficient of friction

$\sum\theta$ - Sum of angular deviations over distance x (absolute value)

K- Unintentional angular deviations per unit length.

x- Distance from the active end to the section considered.

The distance x should in principle be measured along the tendon, but a straight length coordinate can normally be used. The reduction of the prestressing force over a distance x is:-

$$\Delta P = P_o\left[1 - e^{\{-\mu(\sum\theta + kx)\}}\right] \tag{4.8}$$

The value of μ depends on the surface characteristics of the tendons and the duct and a straight length is a fundamental difference between unbounded and bonded tendons.(see table 5.1 in ES-EN-1992:2015).

The value of K for unintentional angular displacement depends on the quality of workmanship, on the distance between tendons supports, on the type of duct or sheath employed, and on the degree of vibration used in placing the concrete. In European technical Approval, values for unintended regular displacements for internal tendons will generally be in the range 0.005<K<0.01 per meter. For eternal tendons, the losses of prestress due to unintentional angles may be ignored.

4.3.4 Wedge draw- in losses.

When tendons are locked in the anchorage, a certain displacement (draw-in) occurs before the wedge have full grip, see Fig.4.10.

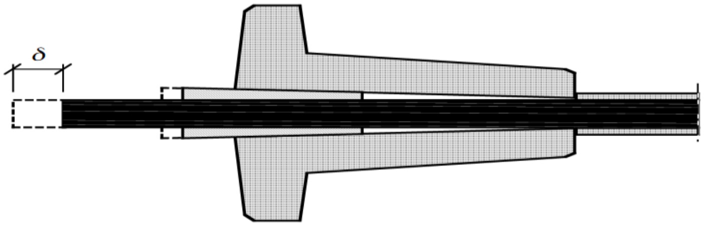


Fig.4.10:- Wedge draw-in

This causes a reduction of the prestressing force, which can be calculated if the magnitude of draw-in is known. Figure 4.24 below shows the variation of the prestressing force before and after wedge draw-in (exaggerated), and for the cases of normal length and short tendons.

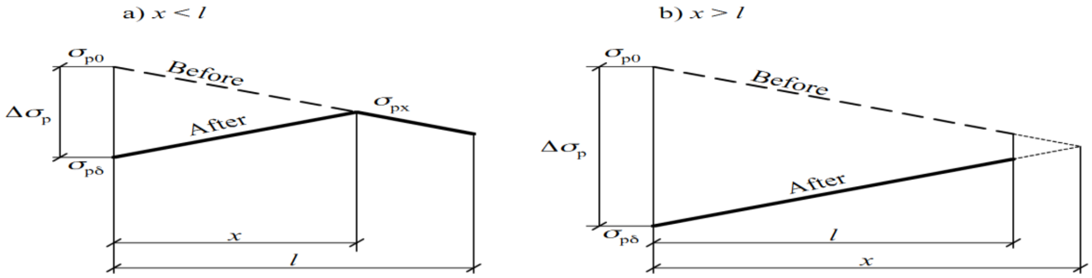


Fig.4.11:- variation of prestress before and after wedge draw-in for a normal length and a short tendon respectively

In case (a) we have:-

$$\delta = \frac{\Delta\sigma_p x}{2E_s} \tag{4.9}$$

, where δ = wedge draw-in

$$\Delta\sigma_p = \sigma_{po} - \sigma_{p\delta} \approx 2\sigma_{po}\mu(\theta + kx) = 2\sigma_{po}\beta x, \beta = \mu\left(\frac{\theta}{x} + k\right) \tag{4.10}$$

Where:- β is the average relative friction loss per unit length

$$x = \sqrt{\frac{\delta}{\beta\varepsilon_{po}}} \quad \text{And} \quad \Delta\sigma_p = 2\sigma_{po}\sqrt{\frac{\beta\delta}{\varepsilon_{po}}} \tag{4.11}$$

Where:- $\varepsilon_{po} = \frac{\sigma_{po}}{E_s}$ is the initial prestrain

In case (b), which occurs if the resulting x is greater than l, the following value of the loss at the active end can be derived:-

$$\Delta\sigma_p = \frac{E_s\delta}{1+\sigma_p\beta l} \tag{4.12}$$

4.3.5 Time dependent losses

The prestress will decrease with time due to shrinkage and creep in the concrete, plus relaxation of the tendons. Different expression for the time dependent loss can be found in ES-EN-1992:2015:-

$$\Delta P_{c+s+r} = A_p\Delta\sigma_{p,c+s+r} = A_p\frac{\varepsilon_{cp}E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(t,t_o)\sigma_{c,Qp}}{1+\frac{E_p}{E_{cm}}\frac{A_p}{A_c}\left(1+\frac{A_c}{I_c}Z_{cp}^2\right)[1+0.8\varphi(t,t_o)]} \tag{4.13}$$

Where:-

$\Delta\sigma_{p,c+s+r}$	is the absolute value of the variation of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t
ε_{cs}	is the estimated shrinkage strain according to 3.1.4(6) in absolute value
E_p	is the modulus of elasticity for the prestressing steel, see 3.3.3 (9)
E_{cm}	is the modulus of elasticity for the concrete (Table 3.1)
$\Delta\sigma_{pr}$	is the absolute value of the variation of stress in the tendons at location x, at time t, due to the relaxation of the prestressing steel. It is determined for a stress of $\sigma_p = \sigma_p(G+P_{m0}+ \psi_2 Q)$ where $\sigma_p = \sigma_p(G+P_{m0}+ \psi_2 Q)$ is the initial stress in the tendons due to initial prestress and quasi-permanent actions.
$\varphi(t,t_0)$	is the creep coefficient at a time t and load application at time t_0
$\sigma_{c,QP}$	is the stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant. The value of $\sigma_{c,QP}$ may be the effect of part of self-weight and initial prestress or the effect of a full quasi-permanent combination of action ($\sigma_c(G+P_{m0}+\psi_2 Q)$), depending on the stage of construction considered.
A_p	is the area of all the prestressing tendons at the level being considered.
A_c	is the area of the concrete section.
I_c	is the second moment of area of the concrete section.
z_{cp}	is the distance between the centre of gravity of the concrete section and the tendons

4.4 Limit states

4.4.1 Ultimate limit states (ULS)

For load bearing structures it is not sufficient to limit deflections, cracks and stresses in serviceability limit state. To ensure a certain safety margin against failure or collapse, also the so-called ultimate limit state (ULS) have to be considered. All possible failure modes should be considered example bending, shear, punching, torsion, anchorage of reinforcement and prestressing tendons, etc.

Design models and criteria for ULS verifications are generally treated in sufficient detail in codes. Therefore, in this document only some particular aspects will be treated, especially those related to the effect of prestress.

Concerning the bending resistance, a major advantage of prestressing is that steel with very high strength can be used. Without prestress, the utilization of high steel stresses will a high strength three to four times that of ordinary reinforcement, the steel area necessary to achieve a certain bending resistance can be reduced proportionally. This, together with the possibility of grouping several strands in few tendons clearly facilitates the detailing of the tension zone.

The ultimate bending resistance is:-

$$M_{RD} = F_T \times Z \tag{4.14}$$

Where:- F_T is the total tensile force and Z is the internal lever arm, see Fig. 4.12. The tensile force F_T is approximately given by:-

$$F_T = A_s f_{yd} + A_p f_{yd} \text{, for bonded tendons} \tag{4.15}$$

$$F_T = A_s f_{yd} + A_p \Delta\sigma_p + P \text{, for unbonded tendons} \tag{4.16}$$

Where:- f_{yd} – design strength of ordinary reinforcement.

f_{pd} - design strength of prestressing tendons.

P - Apσp - orestress force.

σp –Prestress stress

Δσp = 100Mp stress increase above prestress for unbonded tendons.

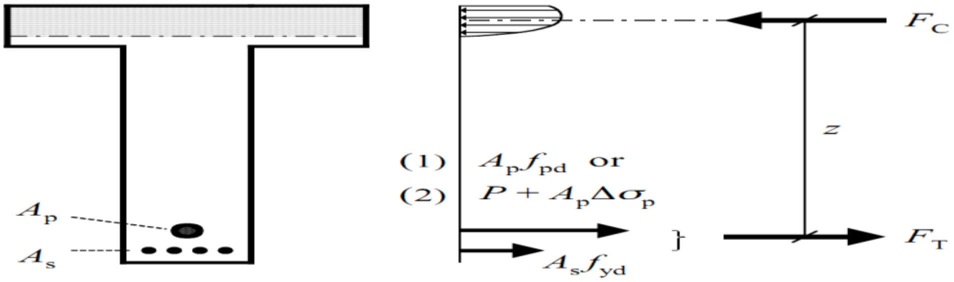


Fig. 4.12:- Stress and forces in a prestressed concrete section for ULS of bending

4.4.1.1 SHEAR RESISTANCE

With curved tendons, the main contribution of prestress to the shear resistance is normally given by the inclination of tendons, as illustrated in Fig.4.31, i.e. the transverse effect of prestress. The contribution to shear resistance is simply the transverse component of the prestressing force. Using a so-called truss model for the design of shear reinforcement, and with prestress considered on the “action side”, the design criterion is:

$$V_{Rdsy} \geq V_{sd} - P\tan\alpha \tag{4.17}$$

$$V_{Rdsy} = \frac{A_{sw}}{s} f_{yd} Z\cot\theta \tag{4.18}$$

Where:- equation 4.18 is the resistance of shear reinforcement.

Alternatively, with prestress consideration as a contribution to the resistance:

$$V_{Rd} = V_{Rdsy} + P \tan \alpha \geq V_{sd} \tag{4.19}$$

Thus, the only difference is whether the contribution of inclined tendons is placed on the right or left hand side of the design equation.

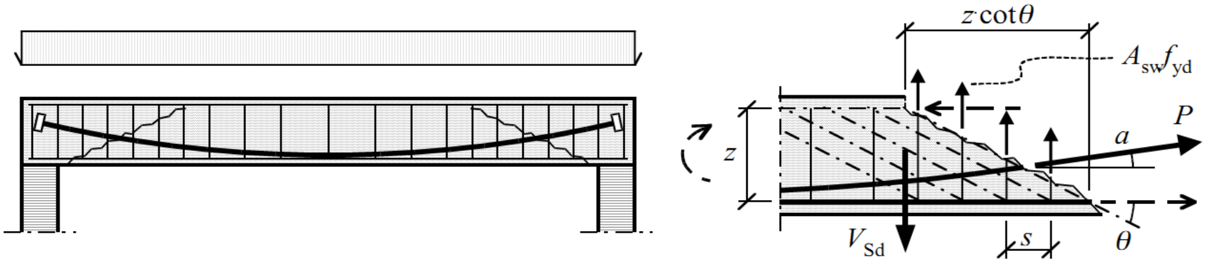


Fig. 4.13:- Shear ultimate limit state of a prestressed concrete element

4.4.1.2 Punching of slabs

In punching of slabs the effect of prestress is similar, but only tendons close to the column can be taken into account. Fig. 4.32 gives an indication of which tendons can be taken into account; the distances x and y can be found in some codes being x generally limited to d/2. The axial effect of prestress is favorable also in punching, but less than in shear.

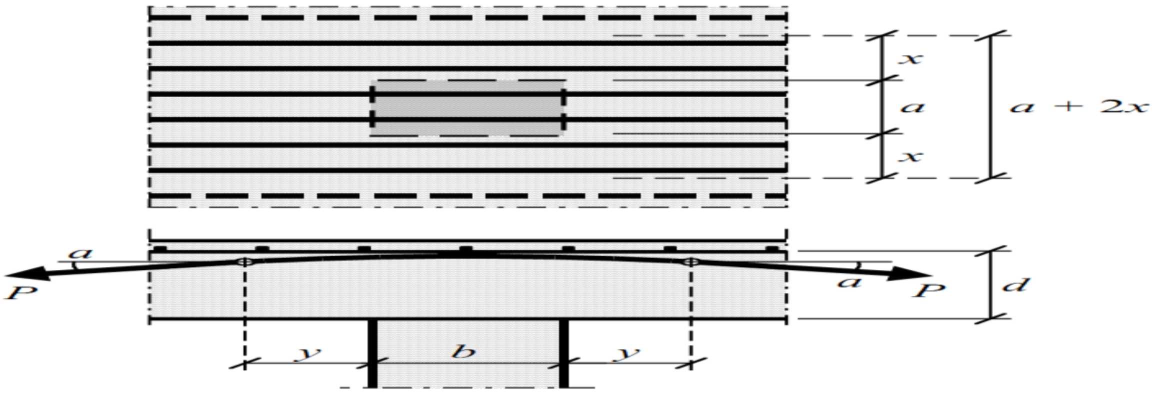


Fig.4.14:- Tendons contributing to the punching resistance in slabs

4.4.2 Serviceability limit states (SLS)

The governing design criteria for prestressed structures are normally are those relating to service conditions, the so-called serviceability limit state (SLS). The reason for choosing prestressed concrete is often a large span and/or a requirement on reduced depth, leading to a high span-depth ratio. In such cases, deflections often become critical.

If concrete is uncracked, the deflection (see Fig. 4.15) can be expressed in the following general way:-

$$a = \frac{k_1 q l^4}{EI} = \frac{k_1 q l^4}{E k_2 b h^3} \tag{4.20}$$

$$\frac{a}{l} = k_3 \frac{q}{bE} \left(\frac{l}{h} \right)^3 \tag{4.21}$$

Where;-

q:-distributed load

L:-span length

E:-concrete modulus of elasticity.

I: - moment of inertia of cross section.

K_1:- coefficient depending on load distribution and boundary conditions.

K_2:- coefficient depending on cross section geometry

$K_3 = K_1/K_2$

b:- width of cross section

h:- depth of cross section.

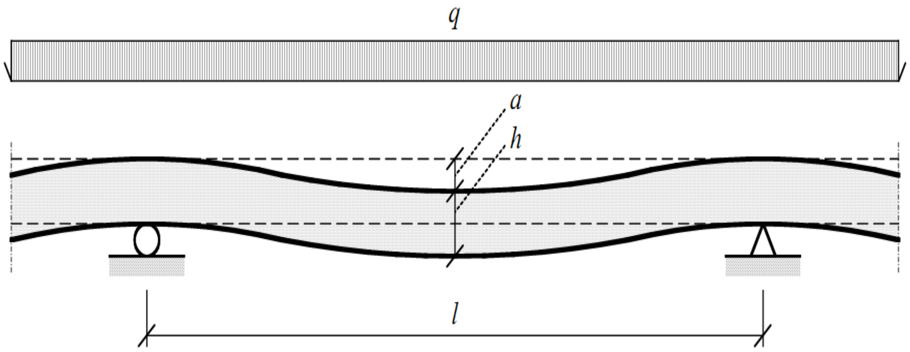


Fig. 4.15:- Illustration of deflection

Prestress can be designed so that the deformation under a certain load, e.g. permanent or quasi-permanent, is partially or totally balanced by the transverse effect. in this way deformations can be kept within acceptable limits.

However, there are limits to the possible slenderness with regard to economy and structural behavior: excessive quantities of prestressing steel should be avoided and the structure should have sufficient stiffness for variable loads, sometimes also with regard to vibrations.

In the final deformation calculation, concrete creep has to be taken into account. The additional deflection due to concrete creep is , provided the structural member is uncracked:

$$a_{creep} = \varphi(a_{go} + a_{po}) \approx 2.5(a_{go} + a_{po}) \quad (4.22)$$

Where:- φ is creep coefficient

a_{go} is downward deflection due to quasi-permanent load

a_{po} is upward deflection due to transverse effect of prestress and the creep coefficient φ depends on concrete composition and quality, ambient humidity etc.

4.5 Stress analysis for pre-stressed concrete flat slab

The stress in the concrete are investigated at different stages in the life of a member. These stresses are calculated on the basis of the materials exhibiting an elastic, uncracked response.

a. Initial stage.

The stress in the concrete immediately after prestress transfer (before time dependent losses have occurred) are limited to prevent crushing or cracking of the young concrete due to the high prestress force.

Eurocode-2 to limit the concrete compressive stress in the structure resulting from the prestressing force at the time of tensioning or release of prestress should be:

$$\sigma_c \leq 0.6 \times f_{ck}(t) \quad (4.23)$$

Where: - $f_{ck}(t)$ is the characteristic compressive strength of the concrete at time t when it is subjected to the presterssing force.(see table 4.3)

b. Final stage.

The stress in the concrete are also investigated at a stage when all prestress losses have occurred and the full service loads are applied. The tensile zones being investigated are those regions of the member where the prestress causes compressive stresses (pre-compressed) but are in tension under service loads.

The consequences of cracking for a structure in a corrosive environment (e.g. a parking garage) are much severe, and hence lower tensile stress limits and larger concrete cover should provide.

4.6 Design spreadsheet preparation of post tensioned flat slab by ES-EN-1992:2015

4.6.1 General Overview

As the floor system plays an important role in the overall cost of a building, a post-tensioned floor system is invented which reduces the time for the construction and finally the cost of the structure. In some countries, a great number of large buildings have been successfully constructed using post-tensioned floors. The reason for this lies in its decisive technical and economic advantages. The most important advantages offered by post-tensioning systems are as follows-

- By comparison with reinforced concrete, a considerable saving in concrete and steel since, due to the working of the entire concrete cross section more slender design are possible.
- Smaller deflections compare to with steel and reinforced concrete structures.
- Good crack behavior and therefore permanent protection of steel against corrosion.
- Almost unchanged serviceability even after considerable overload, since temporary cracks close again after the overload has disappeared.
- High fatigue strength, since the amplitude of the stress changes in the prestressing steel under alternating loads are quite small.
- If a significant part of the load is resisted by post-tensioning the non-presteressed reinforcement can be simplified and standardized to a large degree. Furthermore, material handling is reduced since the total amount of steel (presterssed and non-presteressed) and concrete is less than for a reinforced concrete floor.
- Assembling of precast elements by post tensioning avoids complicated reinforcing bar connections with insitu closure pours, or welded steel connectors, and thus can significantly reduce erection time.
- Usually the permanent floor load is largely balanced b draped post-tensioning tendons so that only the weight of the wet concrete of the floor above induced flexural stresses. These are often of the same order as the design live load stresses. Post-tensioning usually balances most of the permanent loads thus significant reducing deflections and tensile stresses.
- The stress provided by post-tensioning may prevent tensile stresses causing the floor to crack.

For the above reasons post-tensioned construction has also come to used in many situations in building. In addition to the above mentioned general features of post-tensioned construction systems, the following advantages of post-tensioned slab over reinforced concrete slabs are listed as follows:

- More economical structures resulting from the use of prestressing steels with a very high tensile strength instead of normal reinforcing steels.

- Larger spans and greater slenderness, which results in reducing dead load, which also has a beneficial effect upon the columns and foundations and reduces the overall height of buildings or enables additional floors to be incorporated in buildings of a given height.

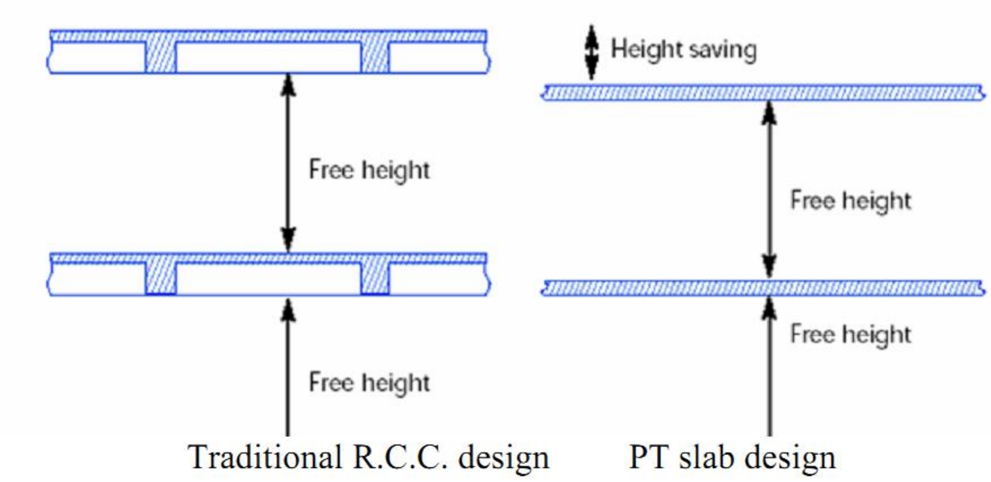


Fig.4.16:- Height comparison of R.C.C. and PT slab design

The design of the post-tension flat slab can be done by using load balancing and equivalent frame method. Among of both the equivalent frame method is widely used. In the load balance method the 65 to 80% of the dead load is carried by the tendons itself. So that there is an upward deflection due to tendon profile resulting the reduction in the overall deflection. In the present study the design of post-tensioned flat slab is done by using equivalent frame method. As the shear and deflection check is the most important for the post-tensioned slab the detail design for shear and deflection (short term deflection and long term deflection due to creep and shrinkage) is carried out. The parametric study of the slab is done and results of the different parameters such as loss due to stress, normal reinforcement, reinforcement for shear, number of tendons, stressing forces and deflection etc. are presented in the graphical and table forms.

4.6.2 Design Methodology

4.6.2.1 General

The analysis of post-tensioned flat slab is complicated problem. The design is basically a trial-and-error process in an effort to reach the best and optimum result. Manual computations of the design may take a time for the engineers to arrive in best design outputs. However, with the possible iterations in steps, some values are assumed; the designers may lose patience and come up with a non-economical proportion.

The Excel Spreadsheet on the other hand is a powerful tool not only in Accounting but also in Engineering. Spreadsheet is mostly used in modification of the traditional hand written method of calculations. The equations are solved exactly the same way in the computer. The computer only makes it easier by doing the calculations and keeping a record for reuse. One has only to become familiar with the Excel functions, many of which are similar to Microsoft Word.

This project seeks to enhance the trial and error design computation of post-tensioned flat slab by developing a spreadsheet. Also, the project attempted to develop an automated analyze process that can be used not only by the designer but also can serve as classroom instructions for handling the subject matter.

4.6.2.2 Program Description

The program developed was a MS Excel Spreadsheet that computes the required prestressing force for the given loads, punching shear preliminarily, tendon profile, prestress losses, concrete stress, number and distribution of tendons. It allow user to have trial-and-error process and instantly see the effect on the design upon input change.

4.6.2.3 Analysis Method

The design of post-tensioned slab is done by two methods, load balancing and equivalent frame methods however; the equivalent frame method of analysis was used in the preparation of excel analysis and design spreadsheet in this project.

4.6.2.4 Load-balancing method

The concept of load balancing is introduced for prestressed concrete structures, as per (T.Y Lin et al) a third approach after the elastic stress and the ultimate strength method of design and analysis. It is first applied to simple beams and cantilevers and then to continuous beams and rigid frames. Its advantage over the elastic stress and ultimate strength methods is not significant for statically determinate structure. When dealing with statically indeterminate systems including flat slabs and certain thin shells. According to load-balancing method, presterssing balances a certain portion of the gravity loads so the flexural members, such as slabs, beams, and girders, will not be subjected to bending stresses under a given load condition. Thus a structure carrying transverse loads is subjected only to axial stresses.

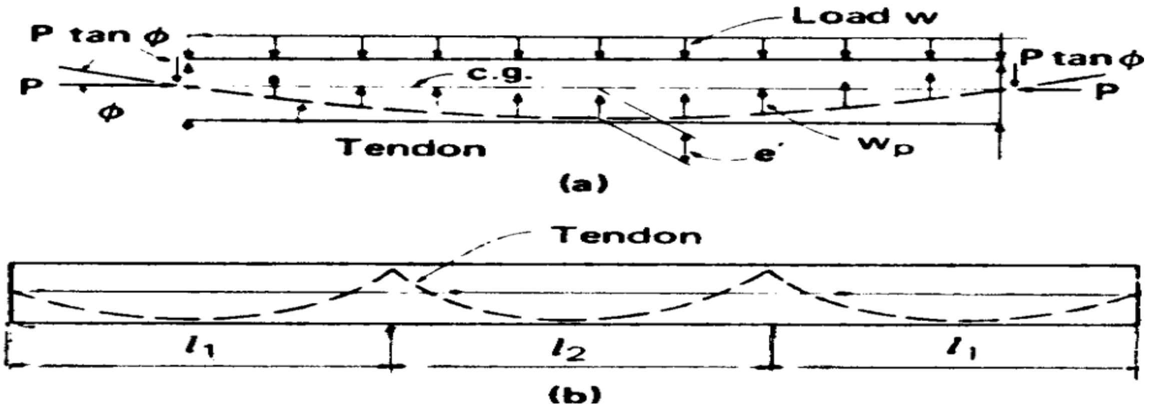


Fig.4.17:- Load balancing

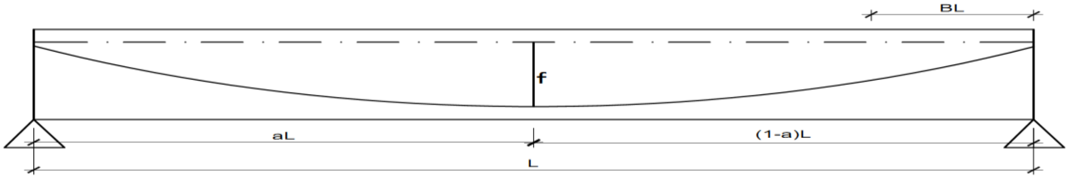
Irrespective of the arrangement of tendons the total prestressing force required to balance

a uniform downward load of w is:-

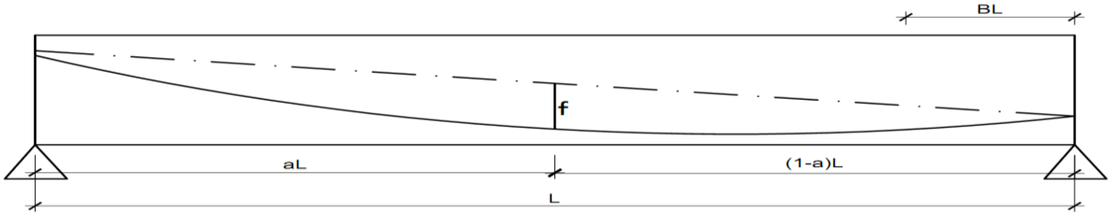
$$P_1 = \frac{w l_1^2}{8 f_1} \text{, for direction 1} \qquad (4.24)$$

$$P_2 = \frac{w l_2^2}{8 f_2} \text{, for direction 2} \qquad (4.25)$$

Where: - P_1 and P_2 are average prestressing force per unit width of slab for tendons in the 1 and 2 directions respectively, w_p is a countering load the tendon applied to the member and θ is the angle formed by the tendon force with respect to the axis of the member.



a) Interior span



b) End span

Fig. 4.18:- Typical tendon profile

4.6.2.5 Equivalent frame method of analysis

The equivalent frame method of analysis is known as the beam method. This method of analysis utilizes the conventional elastic analysis assumption and models the slab or slab and columns, as a beam or as a frame, respectively. This is the most widely used and applied method of analysis for post-tensioned flat slabs.

According to (Y.H.Luo, A. Durrani et al) the effect of vertical of lateral services and design loading on post-tensioned flat slab, boded or unbounded, may be analyzed as for rigid frames in accordance with the provisions of the code (euro code). As per (A.C. Scordelis, Lin, T.Y, and R Itaya et al) the moment induced by prestressing may also determine by a similar analysis of a rigid frame or continuous beam, using equivalent load or load balancing concept. However it should be kept in mind that the distribution of moments due to loads may differ considerably from the distribution of moments due to prestressing.

Service loads produce very pronounced moments peaks at column, whereas the moment curve produced by post- tensioning has a more gentle undulating variation of the same form as the tendon profile.

4.6.2.6 Programming Procedures

The program platform was Microsoft Office 2010 Professional Edition. The user may not see the steps and procedures computed but may follow through the necessary step by step output reflected in the spreadsheet cells. Programing, excel data validation are introduced on cells that contains varies formula.

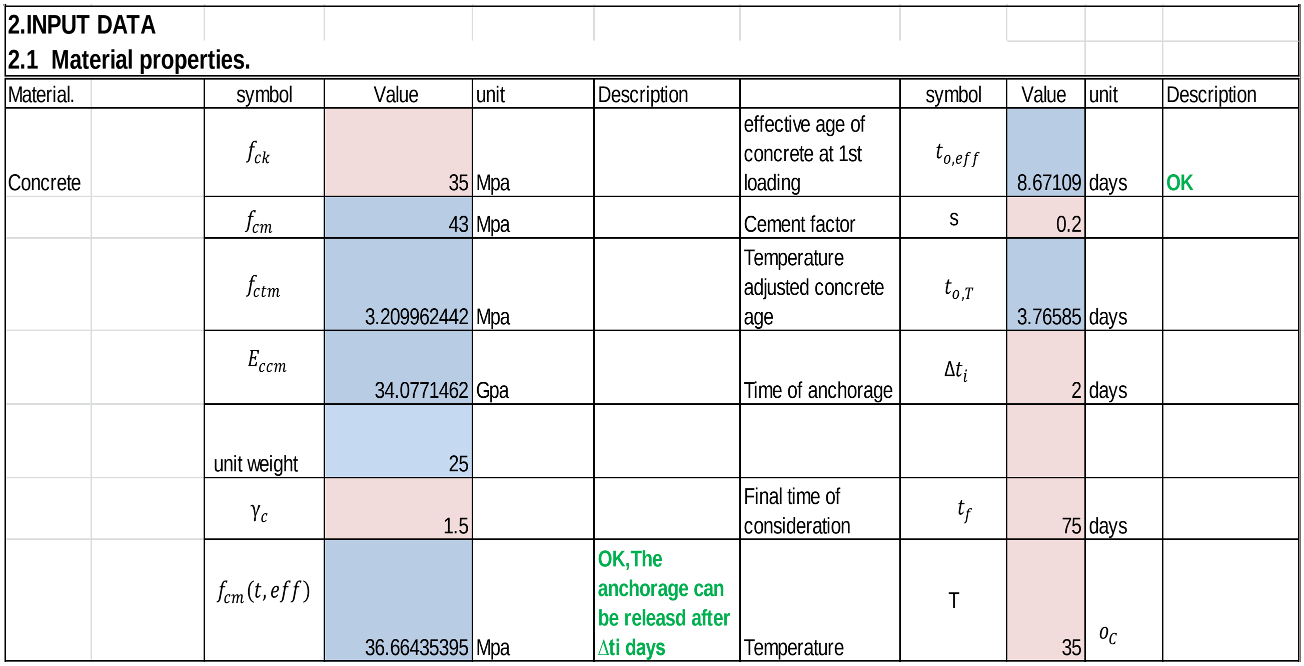
4.6.2.7 Input Cells

The cells of spreadsheet were protected, only cells allocated for user input can be edited. Input cells were formatted red (Accent 2).

4.6.2.8 Output Cells

The output cells are protected and cannot be altered. Output results were preliminary computations, the required prestressing force for the given loads, punching shear preliminarily, tendon profile, prestress losses, concrete stress, number and distribution of tendons etc.

The Excel spread sheet also contain a code descriptions, formulas and comments as red mark give a suggestion and instruction to analyze easily as much as possible (figure 4.19).



2.INPUT DATA

2.1 Material properties.

Material.		symbol	Value	unit	Description		symbol	Value	unit	Description
Concrete		f_{ck}	35	Mpa		effective age of concrete at 1st loading	$t_{o,eff}$	8.67109	days	OK
		f_{cm}	43	Mpa		Cement factor	s	0.2		
		f_{ctm}	3.209962442	Mpa		Temperature adjusted concrete age	$t_{o,T}$	3.76585	days	
		E_{ccm}	34.0771462	Gpa		Time of anchorage	Δt_i	2	days	
		unit weight	25							
		γ_c	1.5			Final time of consideration	t_f	75	days	
		$f_{cm}(t,eff)$	36.66435395	Mpa	OK,The anchorage can be releasd after Δti days	Temperature	T	35	^oC	

Fig. 4.19:- Comment and descriptions on excel spreadsheet

4.6.2.9 Flowchart Development

The analysis of post-tensioned flat slab is generalized in following flowchart shown in fig.4.20.

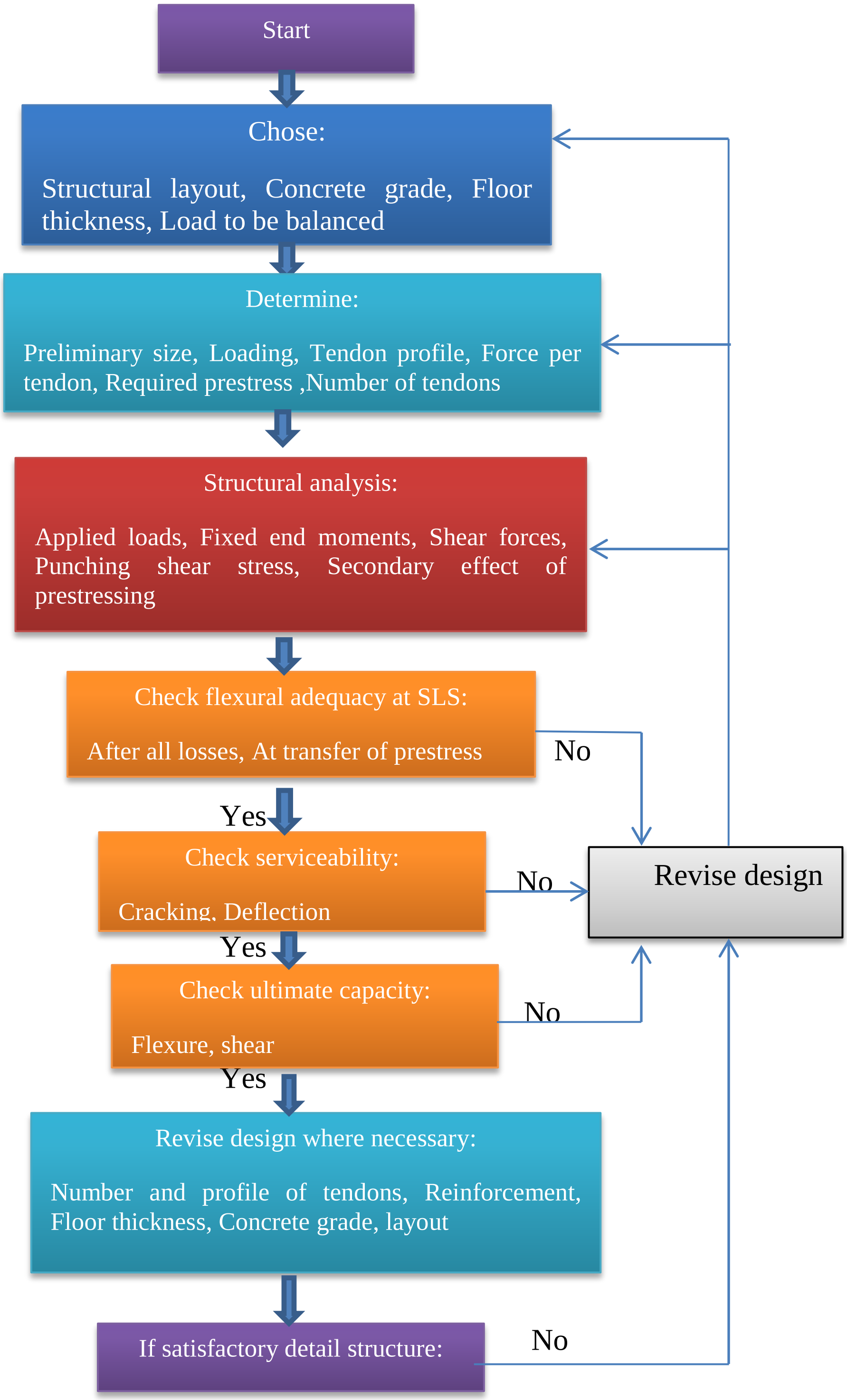


Fig.4.20:-Design flowchart for PT slab

4.6.3 Analysis of post-tension flat slab using excels Spreadsheet by ES -EN:-1992-2015

4.6.3.1 General

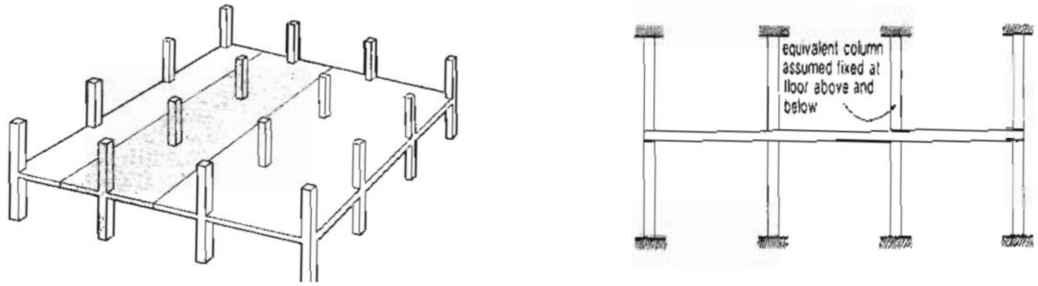
Before the flexural stresses at service load level on the flexural capacity under factored load can be checked, it is necessary to determine the moments at critical sections in the slab caused by loading. The equivalent frame method is the most popular method and this approach involves performing elastic analysis of "equivalent" two-dimensional frames composed of the slab strips connected to the columns above and below the slab being designed. This method is used for analyzing two way slabs for service conditions as well as for ultimate conditions.

4.6.3.2 Design Procedure

- Assume a set of loads to be balanced by parabolic tendons
- Analyze an equivalent frame subjected to the net downward loads according to ES-EN:-1992-2015 or EURO CODE
- Check flexural stresses at critical sections, and revise load balancing tendon forces as required to obtain permissible flexural stresses according to the code.
- When the forces are determined, obtain frame moments for factored live and dead loads.
- Calculate secondary moments induced in the frame by post-tensioning forces, and combine with factored load moments to obtain design factored moments.
- Provide minimum bonded reinforcement in accordance with the code.
- Check design flexural strength and increase non-prestressed reinforcement if required by strength criteria.
- Investigate shear strength, including shear due to vertical load and due to moment transfer, and compare total to permissible values calculated accordance with the code.

4.6.3.3 Design steps

Step 1:- Divide the slab into a series of design strips in the two principal directions of the structure. The width of each design strip is a function of both the center-to-center span l_1, in the direction of the design strip, and the center-to-center span l_2, perpendicular to the design strip illustrated in Fig.4.21.



(a) Design strip in a structure (b) Idealized two dimensional frames for design strip

Fig.4.21:- Equivalent frame idealization

Step 2:-Calculate the stiffness of equivalent frame members.

The stiffness of the slabs and the columns are then determined as illustrated in the Fig.4.22. The stiffness's of the columns are then reduced to account for the twisting of the slab as shown in the Fig.4.23.

The equivalent column stiffness K_{cc} is determined from:

$$\frac{1}{k_{cc}} = \frac{1}{\sum k_c} + \frac{1}{k_t} \tag{4.26}$$

Where: - $\sum K_t$ is the sum of flexural stiffnesses of columns at the joint and K_t are the torsional stiffness's of the assumed torsional members attached to the column and perpendicular to the direction in which moments are being calculated (fig. 5.8)

$$k_t = \frac{\sum 9 E_{CC} C}{l_2\left(1 - \frac{c_2}{l_2}\right)^3} \quad \text{Where: -} \quad C = \frac{\left(1 - 0.63\frac{x}{y}\right)x^3 y}{3} \tag{4.27}$$

And x and y are equal to h and c, for the case of a slab without beams.

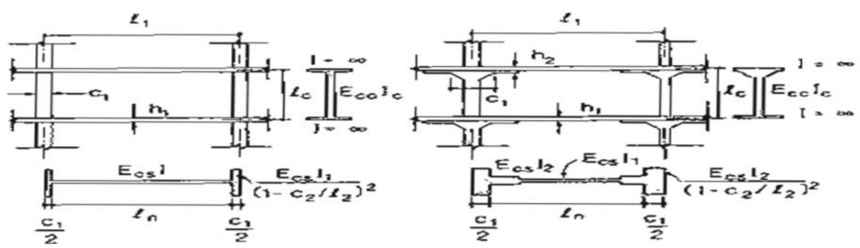


Fig.4.22:- The slab and column stiffness

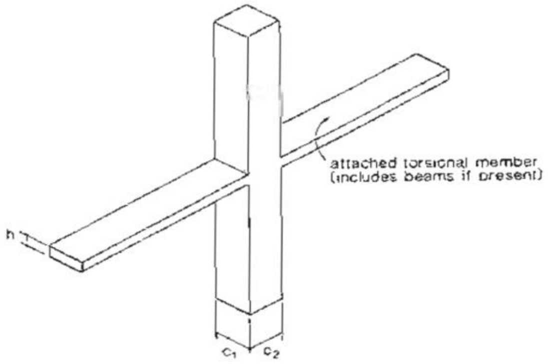


Fig.4.23:- Column with attached torsional members

Step 3:- Analyze equivalent frames.

For different design strips determine the fixed-end moments due to the final presressing after all losses. The final prestressing force can be approximated by assuming a lump sum loss. The equivalent frame is then analyzed to find the restraint moments. Performing separate analysis of the equivalent frame subjected to:

- Slab dead load.
- Additional dead load.
- Live load.

The sequence of construction greatly influences the initial moments in a statically indeterminate structure, creep considerably reduces the influence of sequence of construction, hence it is customary to neglect the influence of the sequence of construction when determining the moments in two-way floor slabs. For this structure the stress under initial conditions are typically not critical. The moments calculated from the equivalent frame method are then used in checking the stresses in the slab at service loads and in checking the required flexural capacities of the slab.

4.6.3.4 Design example by using excels spreadsheet

The design of the flat slab floor system for the office building of G+7 described in the figure will be used to illustrate the steps in the design of a structure. The designer expected to feel the cells having red (Accent 2) only and the output and all the computations are processed and automatically displayed in blue (Accent 1)

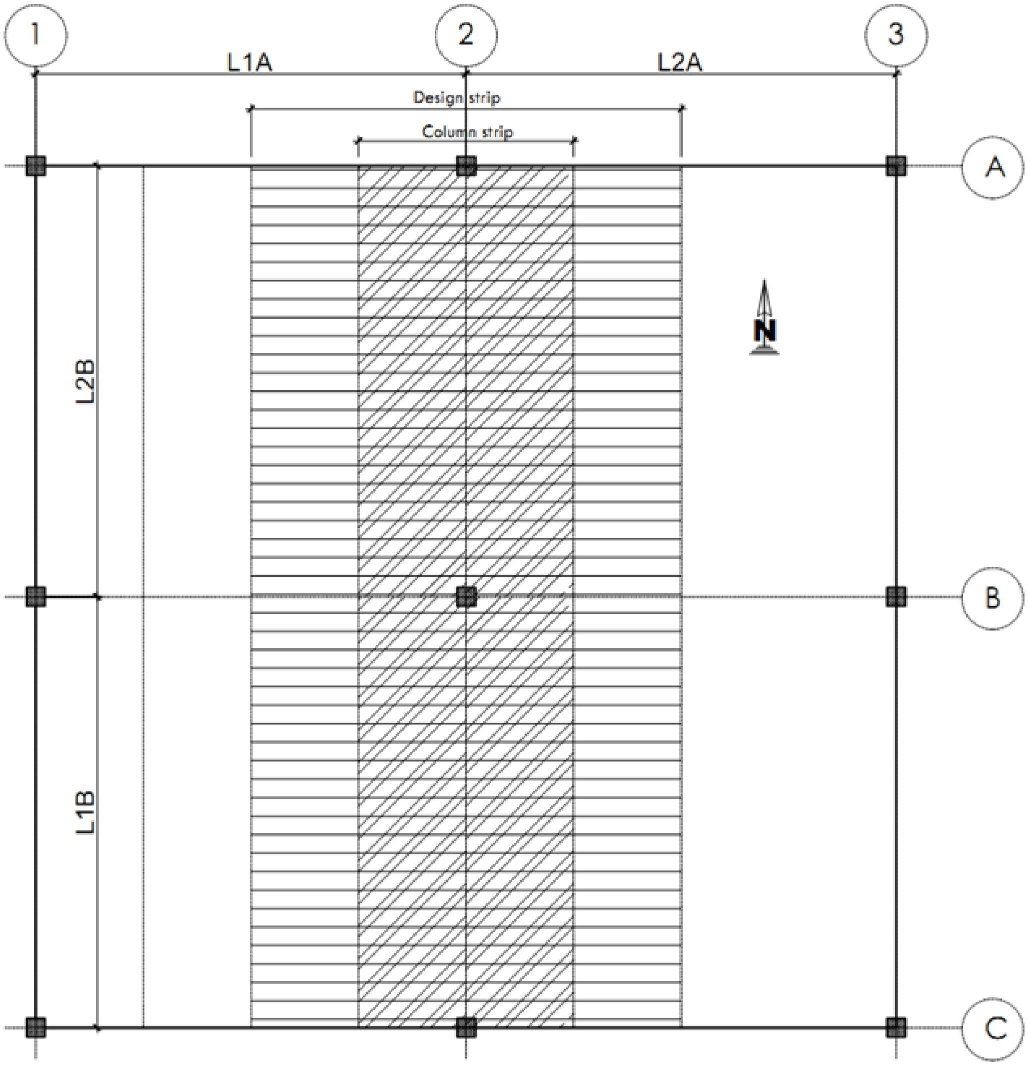
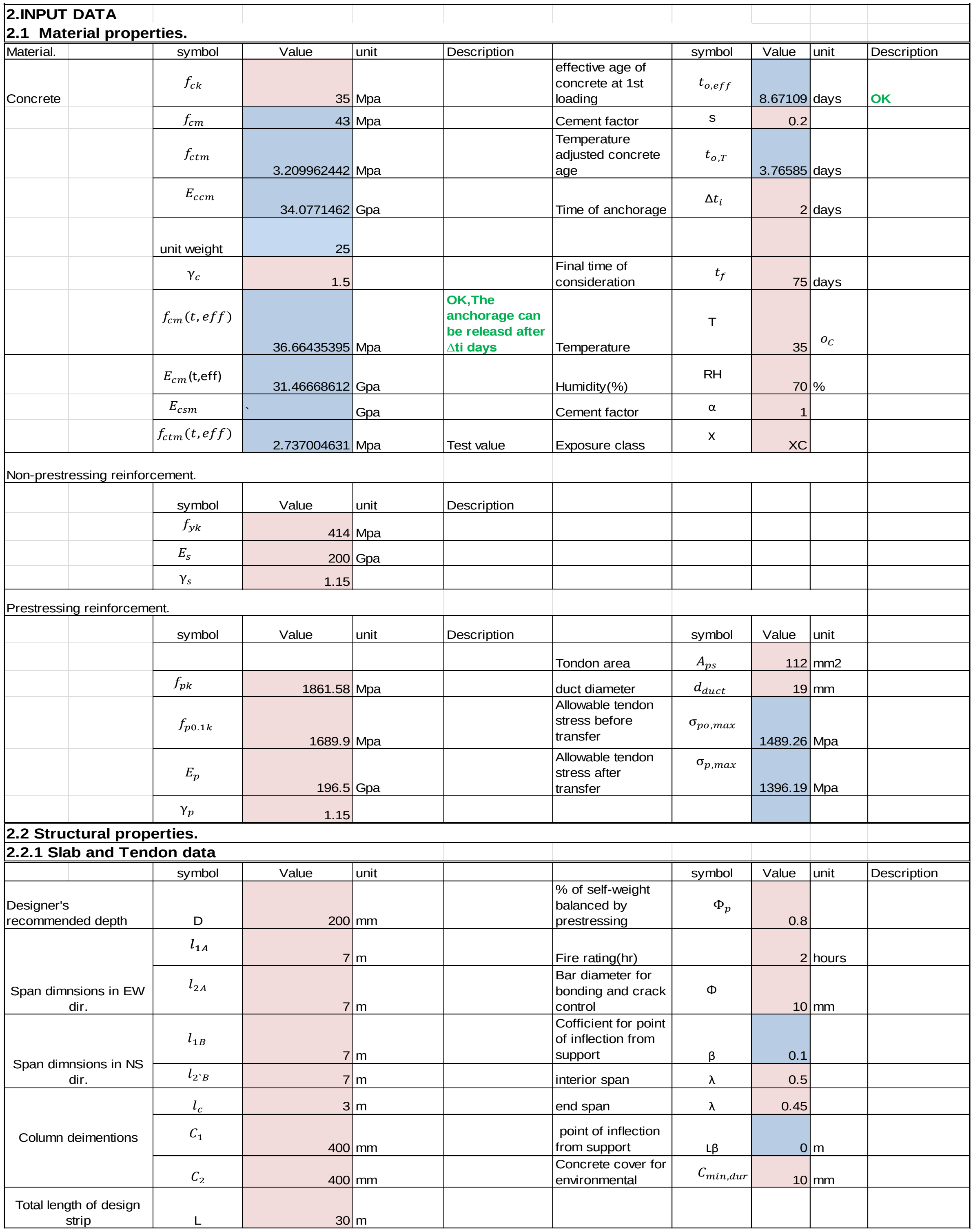


Fig.4.24:- Typical floor plane

- ***Step 1:- Design Data***

1.1.Material properties.



2.INPUT DATA									
2.1 Material properties.									
Material.	symbol	Value	unit	Description		symbol	Value	unit	Description
Concrete	f_{ck}	35	Mpa		effective age of concrete at 1st loading	$t_{o,eff}$	8.67109	days	OK
	f_{cm}	43	Mpa		Cement factor	s	0.2		
	f_{ctm}	3.209962442	Mpa		Temperature adjusted concrete age	$t_{o,T}$	3.76585	days	
	E_{ccm}	34.0771462	Gpa		Time of anchorage	Δt_i	2	days	
	unit weight	25							
	γ_c	1.5			Final time of consideration	t_f	75	days	
	$f_{cm}(t,eff)$	36.66435395	Mpa	OK,The anchorage can be releasd after ∆ti days	Temperature	T	35	^oC	
	E_{cm}(t,eff)	31.46668612	Gpa		Humidity(%)	RH	70	%	
	E_{csm}	`	Gpa		Cement factor	α	1		
	$f_{ctm}(t,eff)$	2.737004631	Mpa	Test value	Exposure class	X	XC		
Non-prestressing reinforcement.									
	symbol	Value	unit	Description					
	f_{yk}	414	Mpa						
	E_s	200	Gpa						
	γ_s	1.15							
Prestressing reinforcement.									
	symbol	Value	unit	Description		symbol	Value	unit	
					Tondon area	A_{ps}	112	mm2	
	f_{pk}	1861.58	Mpa		duct diameter	d_{duct}	19	mm	
	$f_{p0.1k}$	1689.9	Mpa		Allowable tendon stress before transfer	$\sigma_{po,max}$	1489.26	Mpa	
	E_p	196.5	Gpa		Allowable tendon stress after transfer	$\sigma_{p,max}$	1396.19	Mpa	
	γ_p	1.15							
2.2 Structural properties.									
2.2.1 Slab and Tendon data									
	symbol	Value	unit			symbol	Value	unit	Description
Designer's recommended depth	D	200	mm		% of self-weight balanced by prestressing	Φ_p	0.8		
Span dimnsions in EW dir.	l_{1A}	7	m		Fire rating(hr)		2	hours	
	l_{2A}	7	m		Bar diameter for bonding and crack control	Φ	10	mm	
Span dimnsions in NS dir.	l_{1B}	7	m		Cofficient for point of inflection from support	β	0.1		
	$l_{2`B}$	7	m		interior span	λ	0.5		
Column deimentions	l_c	3	m		end span	λ	0.45		
	C_1	400	mm		point of inflection from support	Lβ	0	m	
	C_2	400	mm		Concrete cover for environmental	$C_{min,dur}$	10	mm	
Total length of design strip	L	30	m						

2.2.2 Estimated slab thickness

	symbol	Value	unit	Description		symbol	Value	unit	Description
Cofficient for structural system	K	1.2		ES-EN-1992: 2015	Allowable shear resistance		1.44437	Mpa	
Reference reinforcment ratio(%)	ρ_o	0.591607978			Shear stress on critical area due to factored load		1.34643	Mpa	
The required tension reinforcment(%)	ρ	0.5			Minimum shear resistance v_{min}		0.68991		OK.
The required compression reinforcment(%)	ρ′	0			v_{ED}	<	$v_{Rd,c}$		THE DEPTH IS ADEQUATE
Span to depth ratio	$\frac{l}{d}$	27.58159994	Designer's recommended ratio	35	The sum of angular displacment over the distance x	θ	1.32578	rad	
Minimum depth required	d_{min}	0.2	m		coefficient of friction	μ	0.2		
Minimum depth required for given fire rating	$d_{min,fire}$		m		an unitentional angular displacement	k	0.005	0.005<k<0.01/m	
Minimum bonded tension rebar	ρ	0.05			The distance along the tendon from active to end	x	7	m	

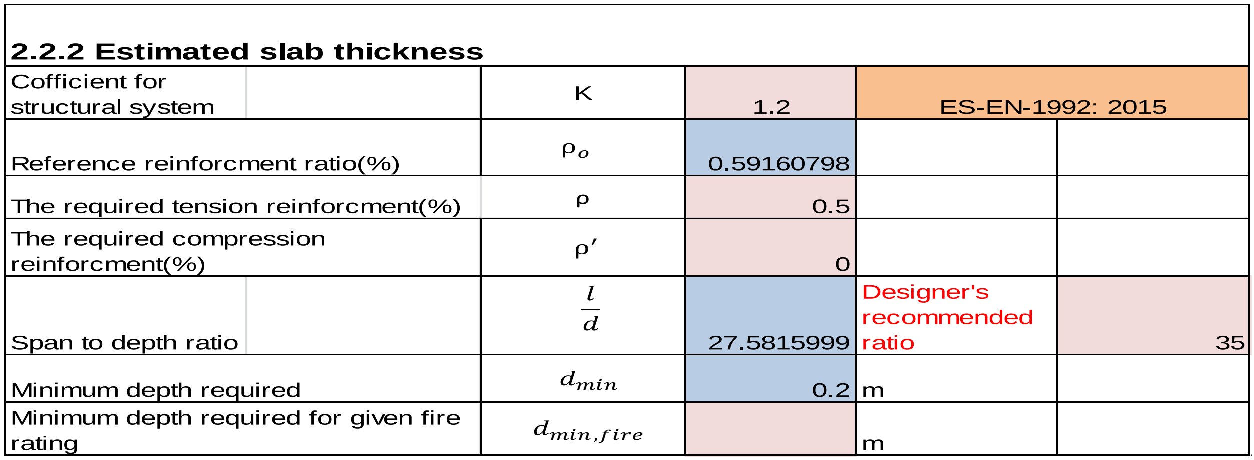
2.2.3 Tendon profil data

	symbol	Value	unit	Description		symbol	Value	unit	Description
Concrete cover for bonding	$C_{min,b}$	19	mm		Eccentricity of upper point of the profile	e_2	61.5	mm	
Concrete cover for environmental	$C_{min,dur}$	10	mm		Point of inflection near to support for interior-span	$h_{2,int}$	26.6	mm	
Required concrete cover at supoprt	C_{min}	19	mm		near to support for end-span	$h_{2,end}$	29.5556	mm	
depth of tendon centroid from top at support	$d_{pl,sup}$	38.5	mm		Tendon drape in mid span	f_{mid}	159.6	mm	
depth of tendon centroid from bottom at mid-span	$d_{pl,mid}$	28.5	mm		Tendon drape in end-span	f_{end}	117.028	mm	
Centroid of slab from top	Z_t	100	mm		Tendon force end-span	P_{end}	209.352	KN/m	
Centroid of slab from bottom	Z_B	100	mm		Tendon force interior-span	P_{int}	250.253	KN/m	
Eccentricity of lower point of the profile	e_1	71.5	mm		Concrete stress due to Prestress	$\Delta\sigma_c(t)$	1.25126	Mpa	

mm	Dead load	D	2.4	KN/m2	Slab partition load		Dss	1	KN/m2
	live load	L	2.4	KN/m2	Beam partition load		DsB	1	KN/m2
	super Dead load	Ds	1.2	KN/m2					
	effective age of concrete at 1st loading	$t_{o,eff}$	6.826526076	days					
	Final time of consideration	t_f	75	days					
	Temperature	T	40	^oC					
	Factored load(1.35D+1.6L)	w_f	13.32	KN/m2					
For Interior design strip	self-weight(end-span,EW)	w_{end}		KN/m2	For edge design strip	self-weight(end-span,NS)	w_{end}		KN/m2
	self-weight(int-span,EW)	w_{int}		KN/m2		self-weight(int-span,NS)	w_{int}		KN/m2
	self-weight(end-span,NS)	w_{end}		KN/m2		self-weight(end-span,EW)	w_{end}		KN/m2
	self-weight(int-span,NS)	w_{int}	5	KN/m2		self-weight(int-span,EW)	w_{int}		KN/m2

Fig.4.25:-Design Data

Estimating the slab thickness.

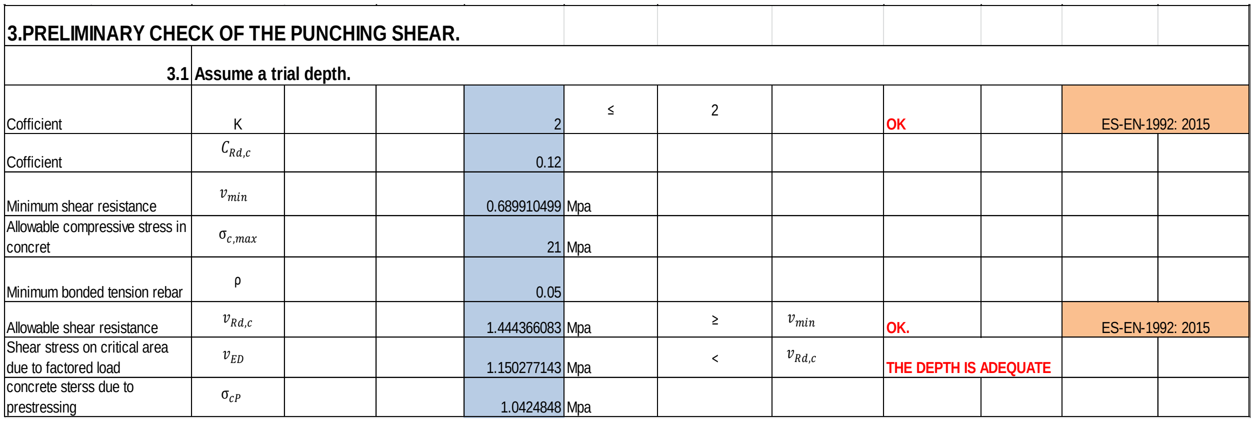


2.2.2 Estimated slab thickness

Cofficient for structural system	K	1.2	ES-EN-1992: 2015	
Reference reinforcment ratio(%)	ρ_o	0.59160798		
The required tension reinforcment(%)	ρ	0.5		
The required compression reinforcment(%)	ρ'	0		
Span to depth ratio	$\frac{l}{d}$	27.5815999	Designer's recommended ratio	35
Minimum depth required	d_{min}	0.2	m	
Minimum depth required for given fire rating	$d_{min,fire}$		m	

Fig.4.26:- Slab thickness estimation

- ***Step 2:- A preliminary check of the punching shear.***



3.PRELIMINARY CHECK OF THE PUNCHING SHEAR.

3.1 Assume a trial depth.

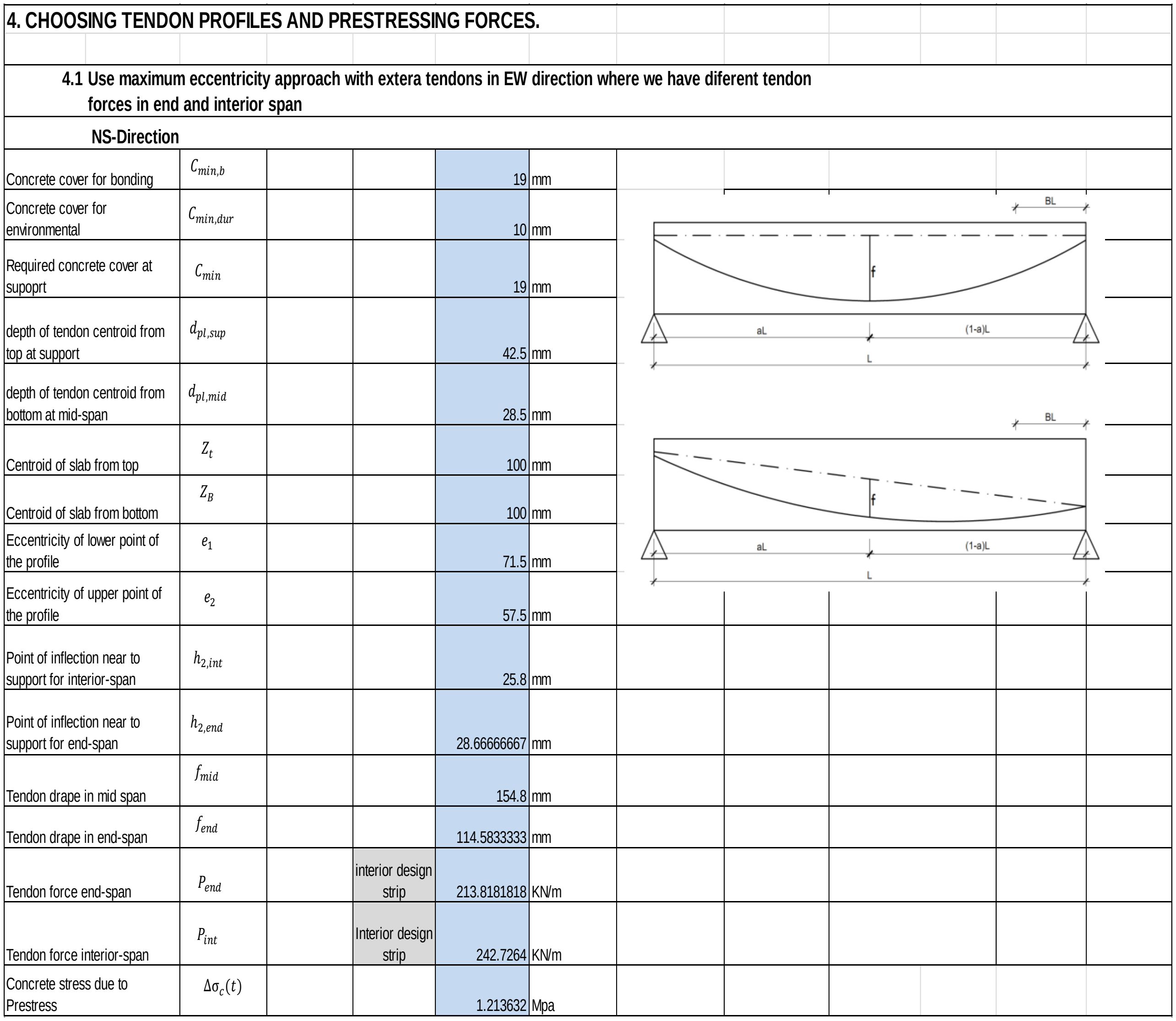
Cofficient	K			2	≤	2		OK		ES-EN-1992: 2015	
Cofficient	$C_{Rd,c}$			0.12							
Minimum shear resistance	v_{min}			0.689910499	Mpa						
Allowable compressive stress in concret	$\sigma_{c,max}$			21	Mpa						
Minimum bonded tension rebar	ρ			0.05							
Allowable shear resistance	$v_{Rd,c}$			1.444366083	Mpa	≥	v_{min}	OK.		ES-EN-1992: 2015	
Shear stress on critical area due to factored load	v_{ED}			1.150277143	Mpa	<	$v_{Rd,c}$	THE DEPTH IS ADEQUATE			
concrete sterss due to prestressing	σ_{cP}			1.0424848	Mpa						

Fig.4.27:- Preliminary punching shear check

- ***Step 3:- Determine the load to be balanced.***

Practical design experience has shown that if the superimposed service loads are less than the self-weight of the slab, then an economical design satisfying serviceability and ultimate requirements will be achieved if the load balanced by presterssing after all losses have occurred is between 80 and 100% of the slab self-weight.

- ***Step 4:- choosing tendon profiles***



4. CHOOSING TENDON PROFILES AND PRESTRESSING FORCES.

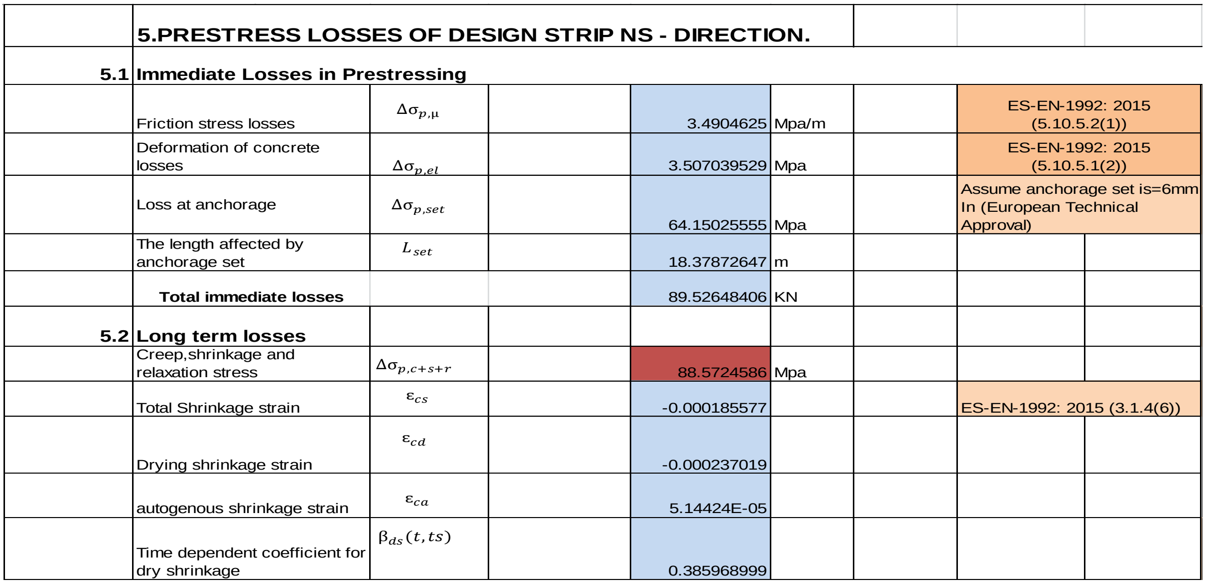
4.1 Use maximum eccentricity approach with extera tendons in EW direction where we have diferent tendon forces in end and interior span

NS-Direction

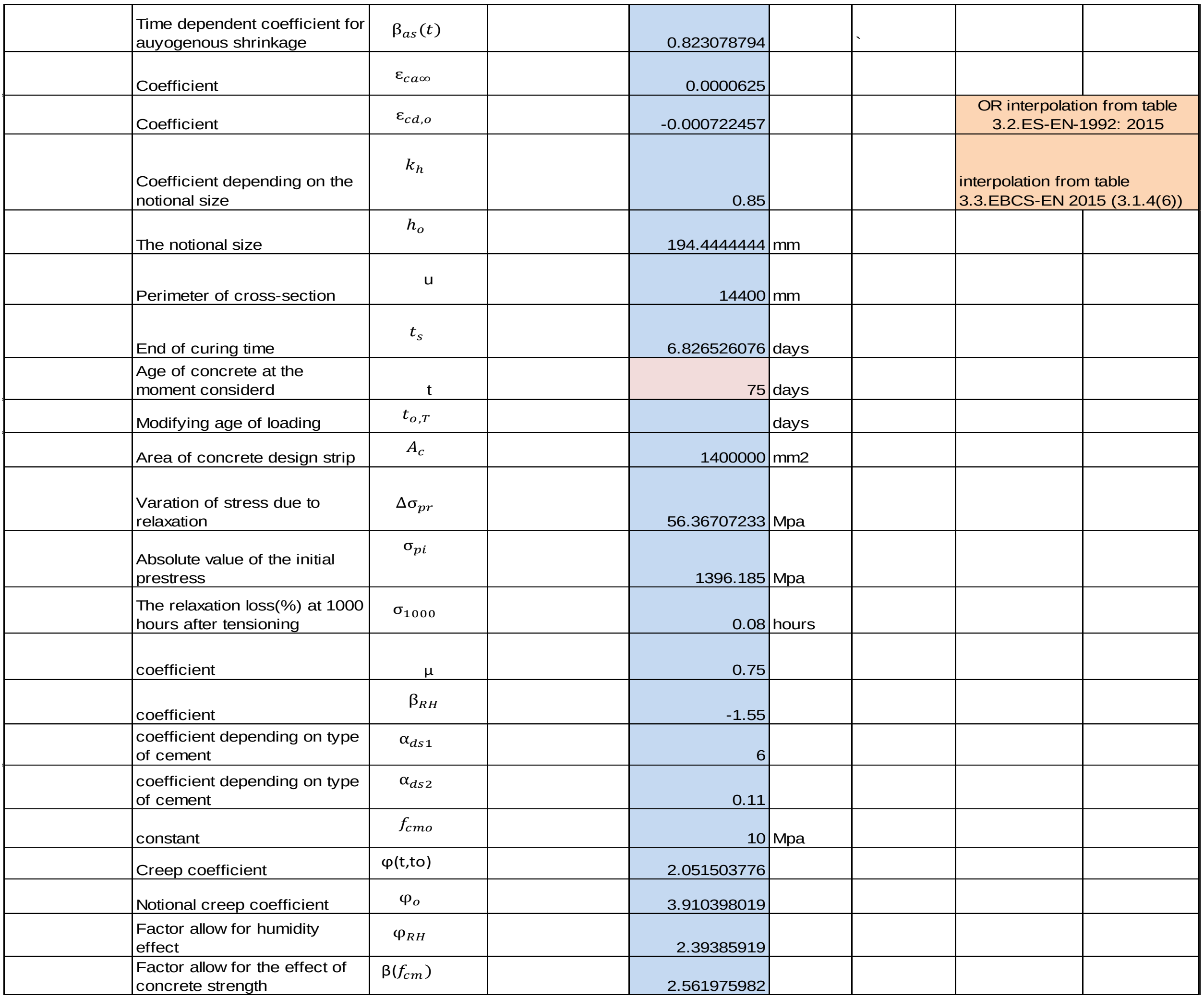
Concrete cover for bonding	$C_{min,b}$			19	mm
Concrete cover for environmental	$C_{min,dur}$			10	mm
Required concrete cover at supoprt	C_{min}			19	mm
depth of tendon centroid from top at support	$d_{pl,sup}$			42.5	mm
depth of tendon centroid from bottom at mid-span	$d_{pl,mid}$			28.5	mm
Centroid of slab from top	Z_t			100	mm
Centroid of slab from bottom	Z_B			100	mm
Eccentricity of lower point of the profile	e_1			71.5	mm
Eccentricity of upper point of the profile	e_2			57.5	mm
Point of inflection near to support for interior-span	$h_{2,int}$			25.8	mm
Point of inflection near to support for end-span	$h_{2,end}$			28.66666667	mm
Tendon drape in mid span	f_{mid}			154.8	mm
Tendon drape in end-span	f_{end}			114.5833333	mm
Tendon force end-span	P_{end}		interior design strip	213.8181818	KN/m
Tendon force interior-span	P_{int}		Interior design strip	242.7264	KN/m
Concrete stress due to Prestress	$\Delta\sigma_c(t)$			1.213632	Mpa

Fig.4.28:- Tendon profile determination

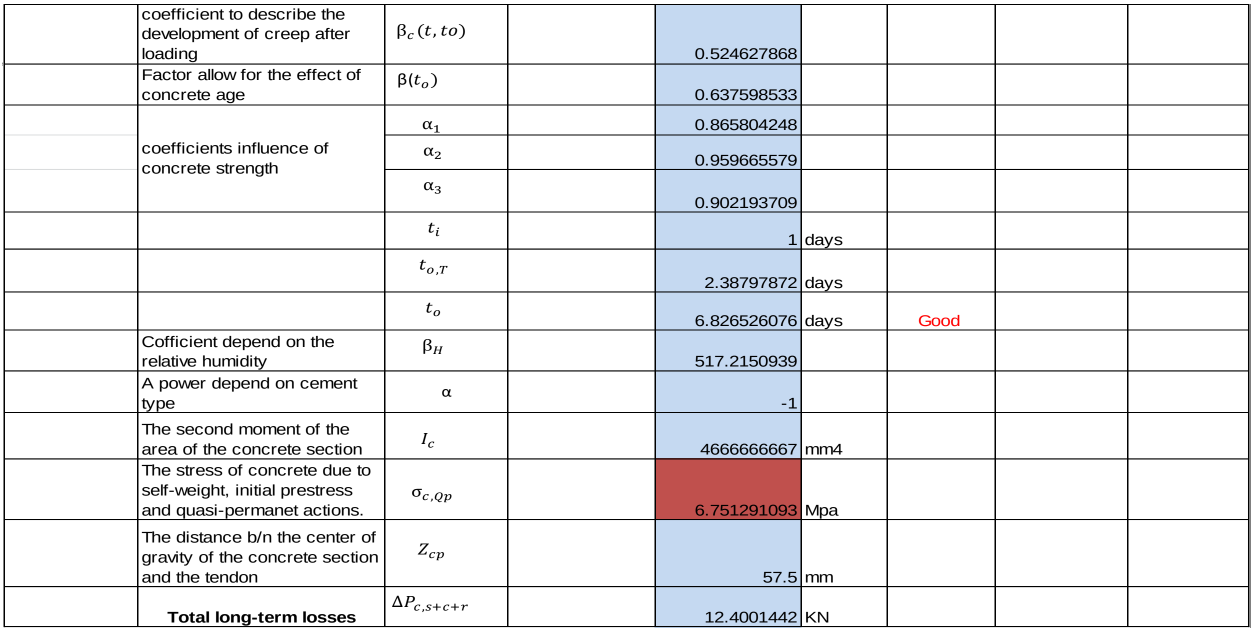
- ***Step 5:-Prestress Losses of design strip.***



	5.PRESTRESS LOSSES OF DESIGN STRIP NS - DIRECTION.							
5.1	**Immediate Losses in Prestressing**							
	Friction stress losses	$\Delta\sigma_{p,\mu}$		3.4904625	Mpa/m		ES-EN-1992: 2015 (5.10.5.2(1))	
	Deformation of concrete losses	$\Delta\sigma_{p,el}$		3.507039529	Mpa		ES-EN-1992: 2015 (5.10.5.1(2))	
	Loss at anchorage	$\Delta\sigma_{p,set}$		64.15025555	Mpa		Assume anchorage set is=6mm In (European Technical Approval)	
	The length affected by anchorage set	L_{set}		18.37872647	m			
	Total immediate losses			89.52648406	KN			
5.2	**Long term losses**							
	Creep,shrinkage and relaxation stress	$\Delta\sigma_{p,c+s+r}$		88.5724586	Mpa			
	Total Shrinkage strain	ε_{cs}		-0.000185577			ES-EN-1992: 2015 (3.1.4(6))	
	Drying shrinkage strain	ε_{cd}		-0.000237019				
	autogenous shrinkage strain	ε_{ca}		5.14424E-05				
	Time dependent coefficient for dry shrinkage	$\beta_{ds}(t,ts)$		0.385968999				



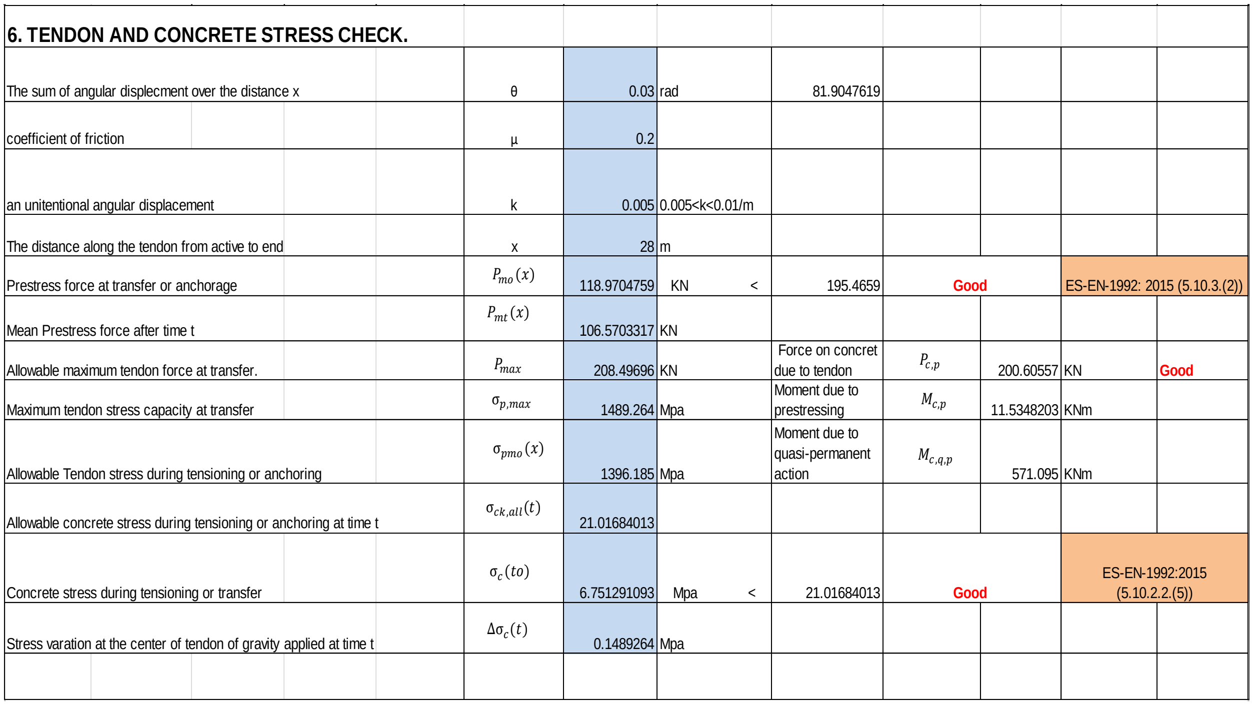
	Time dependent coefficient for auyogenous shrinkage	$\beta_{as}(t)$		0.823078794		`		
	Coefficient	$\varepsilon_{ca\infty}$		0.0000625				
	Coefficient	$\varepsilon_{cd,o}$		-0.000722457			OR interpolation from table 3.2.ES-EN-1992: 2015	
	Coefficient depending on the notional size	k_h		0.85			interpolation from table 3.3.EBCS-EN 2015 (3.1.4(6))	
	The notional size	h_o		194.4444444	mm			
	Perimeter of cross-section	u		14400	mm			
	End of curing time	t_s		6.826526076	days			
	Age of concrete at the moment considerd	t		75	days			
	Modifying age of loading	$t_{o,T}$			days			
	Area of concrete design strip	A_c		1400000	mm2			
	Varation of stress due to relaxation	$\Delta\sigma_{pr}$		56.36707233	Mpa			
	Absolute value of the initial prestress	σ_{pi}		1396.185	Mpa			
	The relaxation loss(%) at 1000 hours after tensioning	σ_{1000}		0.08	hours			
	coefficient	μ		0.75				
	coefficient	β_{RH}		-1.55				
	coefficient depending on type of cement	α_{ds1}		6				
	coefficient depending on type of cement	α_{ds2}		0.11				
	constant	f_{cmo}		10	Mpa			
	Creep coefficient	$\varphi(t,to)$		2.051503776				
	Notional creep coefficient	φ_o		3.910398019				
	Factor allow for humidity effect	φ_{RH}		2.39385919				
	Factor allow for the effect of concrete strength	$\beta(f_{cm})$		2.561975982				



	coefficient to describe the development of creep after loading	$\beta_c(t, to)$		0.524627868				
	Factor allow for the effect of concrete age	$\beta(t_o)$		0.637598533				
	coefficients influence of concrete strength	α_1		0.865804248				
		α_2		0.959665579				
		α_3		0.902193709				
		t_i		1	days			
		$t_{o,T}$		2.38797872	days			
		t_o		6.826526076	days	Good		
	Cofficient depend on the relative humidity	β_H		517.2150939				
	A power depend on cement type	α		-1				
	The second moment of the area of the concrete section	I_c		4666666667	mm4			
	The stress of concrete due to self-weight, initial prestress and quasi-permanet actions.	$\sigma_{c,Qp}$		6.751291093	Mpa			
	The distance b/n the center of gravity of the concrete section and the tendon	Z_{cp}		57.5	mm			
	Total long-term losses	$\Delta P_{c,s+c+r}$		12.4001442	KN			

Fig.4.29:- Determining losses of prestressing

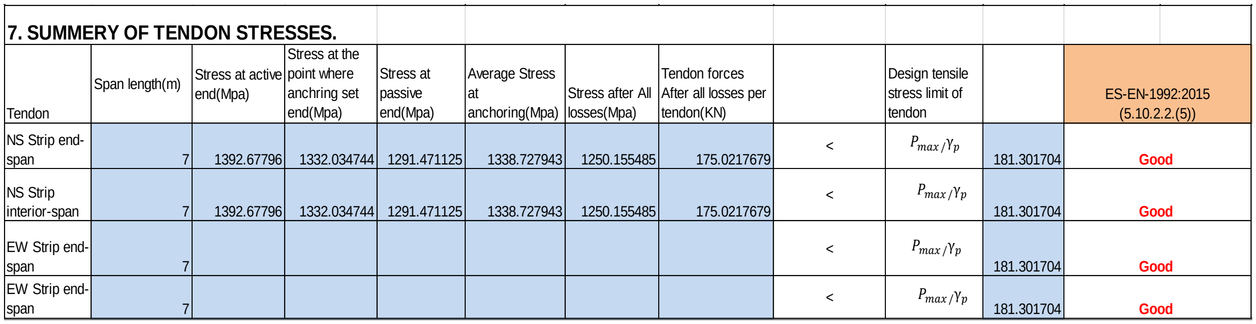
- ***Step 6:- Tendon and concrete stress check***



6. TENDON AND CONCRETE STRESS CHECK.								
The sum of angular displecment over the distance x	θ	0.03	rad	81.9047619				
coefficient of friction	μ	0.2						
an unitentional angular displacement	k	0.005	0.005<k<0.01/m					
The distance along the tendon from active to end	x	28	m					
Prestress force at transfer or anchorage	$P_{mo}(x)$	118.9704759	KN <	195.4659	Good		ES-EN-1992: 2015 (5.10.3.(2))	
Mean Prestress force after time t	$P_{mt}(x)$	106.5703317	KN					
Allowable maximum tendon force at transfer.	P_{max}	208.49696	KN	Force on concret due to tendon	$P_{c,p}$	200.60557	KN	Good
Maximum tendon stress capacity at transfer	$\sigma_{p,max}$	1489.264	Mpa	Moment due to prestressing	$M_{c,p}$	11.5348203	KNm	
Allowable Tendon stress during tensioning or anchoring	$\sigma_{pmo}(x)$	1396.185	Mpa	Moment due to quasi-permanent action	$M_{c,q,p}$	571.095	KNm	
Allowable concrete stress during tensioning or anchoring at time t	$\sigma_{ck,all}(t)$	21.01684013						
Concrete stress during tensioning or transfer	$\sigma_c(to)$	6.751291093	Mpa <	21.01684013	Good		ES-EN-1992:2015 (5.10.2.2.(5))	
Stress varation at the center of tendon of gravity applied at time t	$\Delta\sigma_c(t)$	0.1489264	Mpa					

Fig.4.30:- tendon and concrete stress

- ***Step 7:- Tendon stress summery***

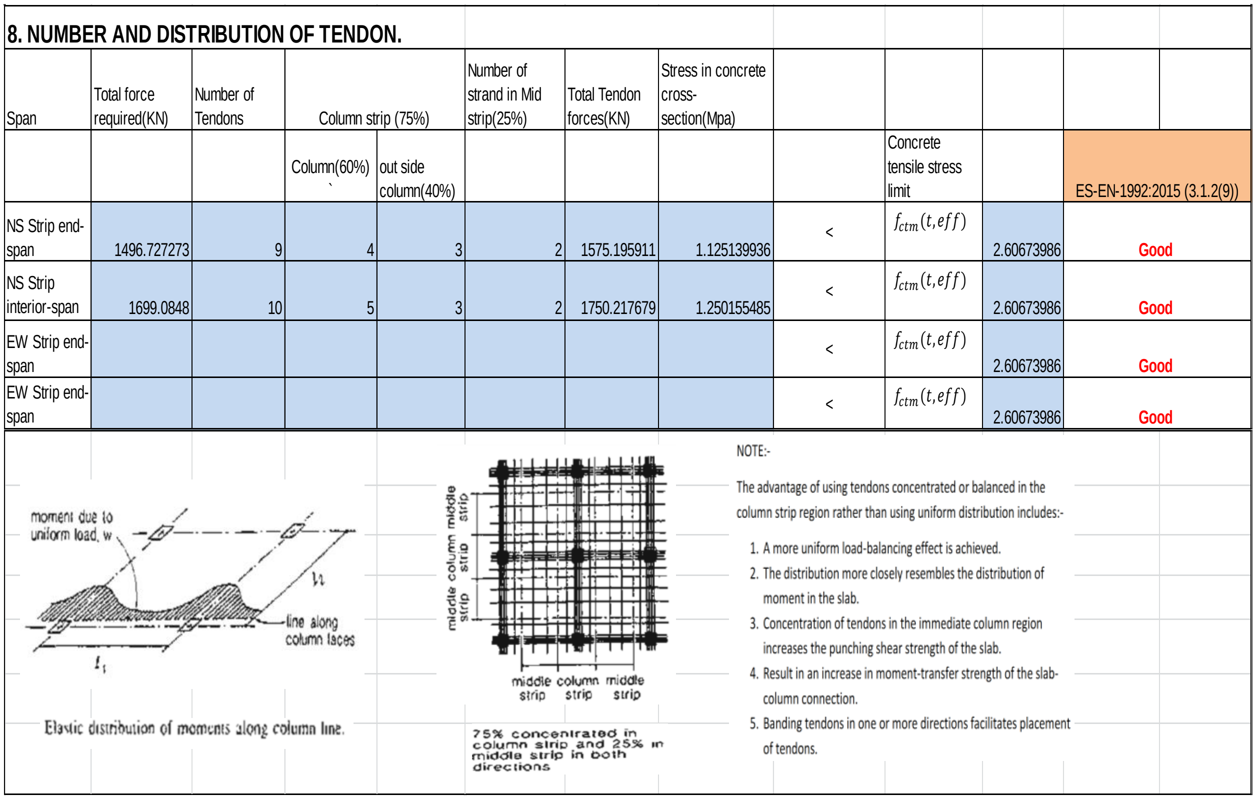


7. SUMMERY OF TENDON STRESSES.

Tendon	Span length(m)	Stress at active end(Mpa)	Stress at the point where anchring set end(Mpa)	Stress at passive end(Mpa)	Average Stress at anchoring(Mpa)	Stress after All losses(Mpa)	Tendon forces After all losses per tendon(KN)		Design tensile stress limit of tendon		ES-EN-1992:2015 (5.10.2.2.(5))
NS Strip end-span	7	1392.67796	1332.034744	1291.471125	1338.727943	1250.155485	175.0217679	<	P_{max}/γ_p	181.301704	Good
NS Strip interior-span	7	1392.67796	1332.034744	1291.471125	1338.727943	1250.155485	175.0217679	<	P_{max}/γ_p	181.301704	Good
EW Strip end-span	7							<	P_{max}/γ_p	181.301704	Good
EW Strip end-span	7							<	P_{max}/γ_p	181.301704	Good

Fig.4.31:- tendon stress summery

- ***Step 8:- Number and distribution of tendons***



8. NUMBER AND DISTRIBUTION OF TENDON.

Span	Total force required(KN)	Number of Tendons	Column strip (75%)		Number of strand in Mid strip(25%)	Total Tendon forces(KN)	Stress in concrete cross-section(Mpa)				
			Column(60%) `	out side column(40%)					Concrete tensile stress limit		ES-EN-1992:2015 (3.1.2(9))
NS Strip end-span	1496.727273	9	4	3	2	1575.195911	1.125139936	<	$f_{ctm}(t,eff)$	2.60673986	Good
NS Strip interior-span	1699.0848	10	5	3	2	1750.217679	1.250155485	<	$f_{ctm}(t,eff)$	2.60673986	Good
EW Strip end-span								<	$f_{ctm}(t,eff)$	2.60673986	Good
EW Strip end-span								<	$f_{ctm}(t,eff)$	2.60673986	Good

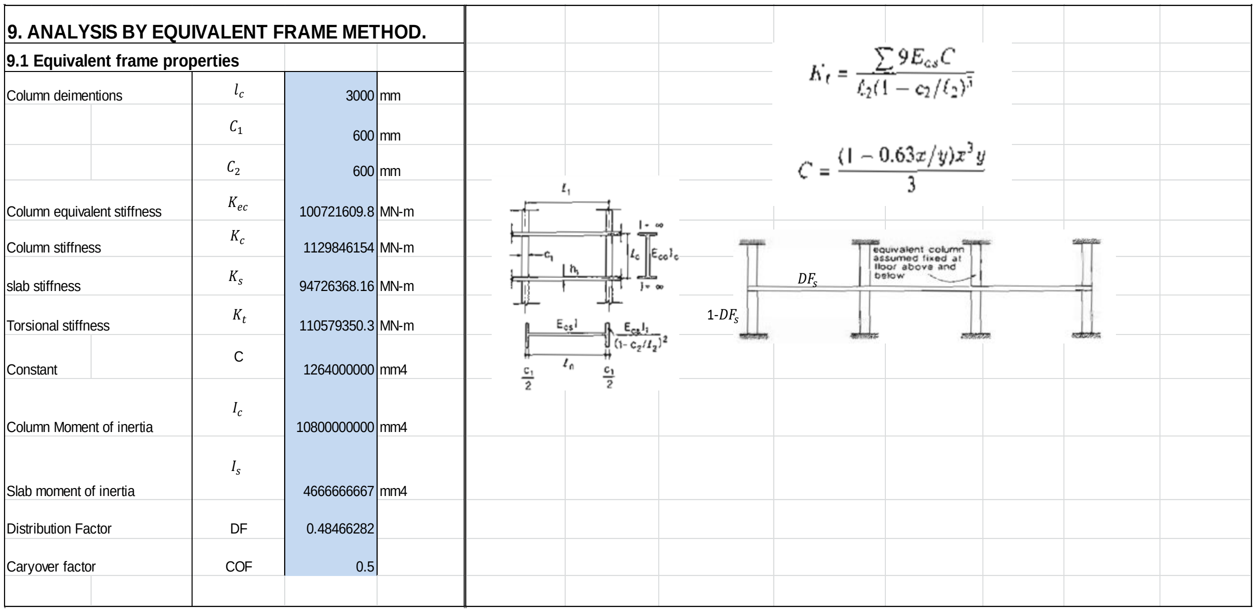
NOTE:-

The advantage of using tendons concentrated or balanced in the column strip region rather than using uniform distribution includes:-

1. A more uniform load-balancing effect is achieved.
2. The distribution more closely resembles the distribution of moment in the slab.
3. Concentration of tendons in the immediate column region increases the punching shear strength of the slab.
4. Result in an increase in moment-transfer strength of the slab-column connection.
5. Banding tendons in one or more directions facilitates placement of tendons.

Fig.4.32:- Number of tendons

- ***Step 9:- Analysis of slab by equivalent frame method***



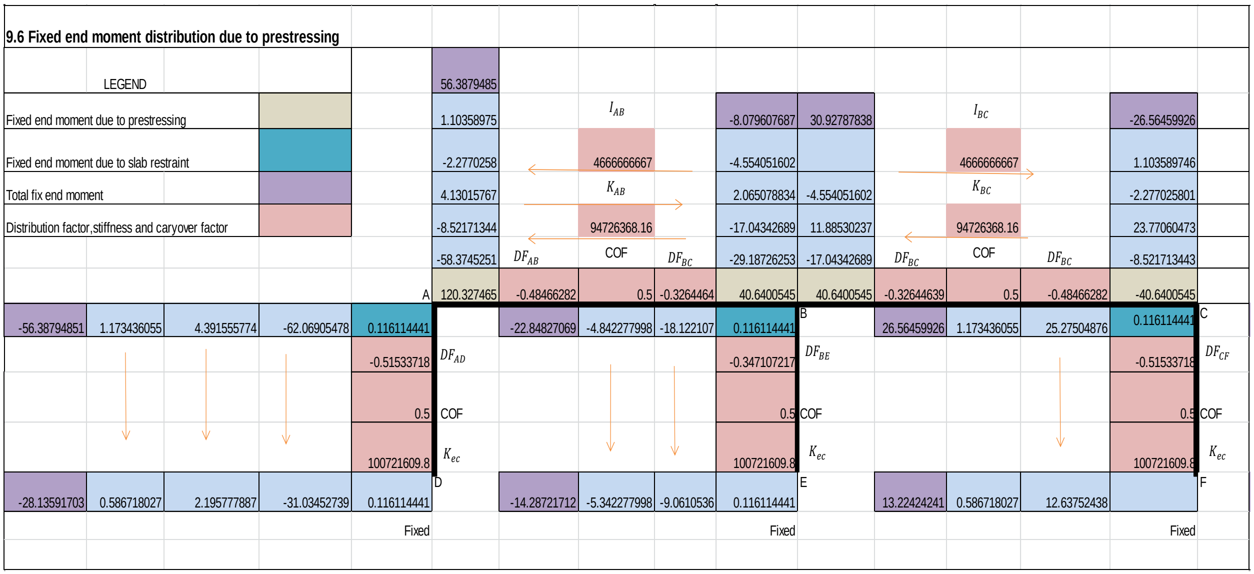
9. ANALYSIS BY EQUIVALENT FRAME METHOD.

9.1 Equivalent frame properties

Column deimentions	l_c	3000	mm
	C_1	600	mm
	C_2	600	mm
Column equivalent stiffness	K_{ec}	100721609.8	MN-m
Column stiffness	K_c	1129846154	MN-m
slab stiffness	K_s	94726368.16	MN-m
Torsional stiffness	K_t	110579350.3	MN-m
Constant	C	1264000000	mm4
Column Moment of inertia	I_c	10800000000	mm4
Slab moment of inertia	I_s	4666666667	mm4
Distribution Factor	DF	0.48466282	
Caryover factor	COF	0.5	

$$K_t = \frac{\sum 9E_{cs}C}{\ell_2(1-c_2/\ell_2)^3}$$

$$C = \frac{(1-0.63x/y)x^3y}{3}$$

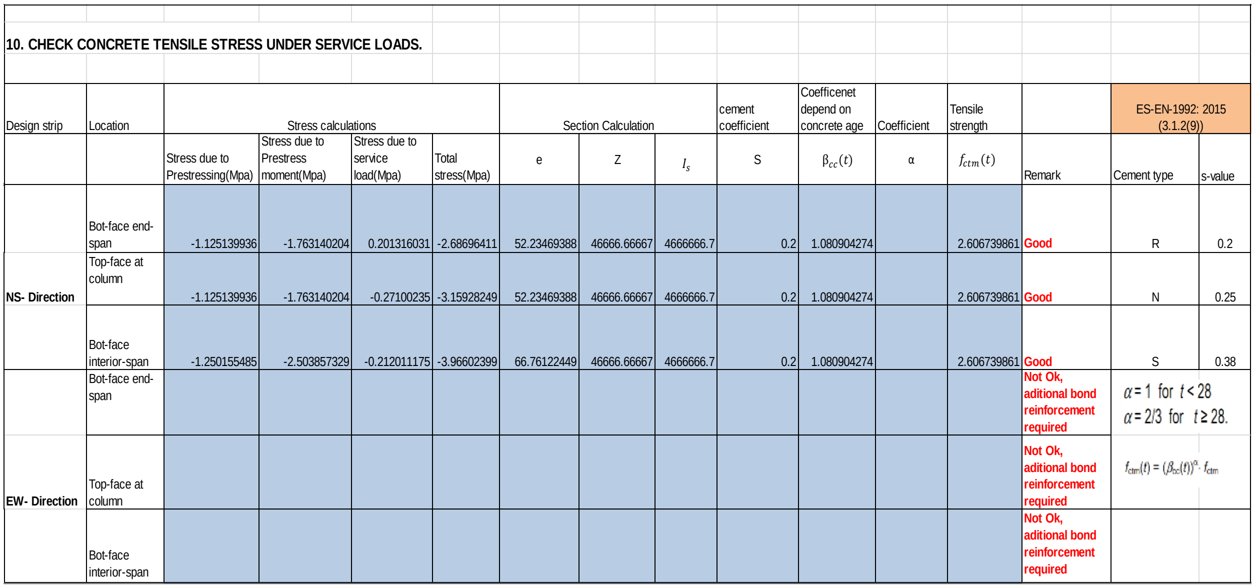


9.6 Fixed end moment distribution due to prestressing

	LEGEND				56.3879485										
Fixed end moment due to prestressing			🟨		1.10358975		I_{AB}		-8.079607687	30.92787838		I_{BC}		-26.56459926	
Fixed end moment due to slab restraint			🟦		-2.2770258		4666666667		-4.554051602			4666666667		1.103589746	
Total fix end moment			🟪		4.13015767		K_{AB}		2.065078834	-4.554051602		K_{BC}		-2.277025801	
Distribution factor,stiffness and caryover factor			🟥		-8.52171344		94726368.16		-17.04342689	11.88530237		94726368.16		23.77060473	
					-58.3745251	DF_{AB}	COF	DF_{BC}	-29.18726253	-17.04342689	DF_{BC}	COF	DF_{BC}	-8.521713443	
				A	120.327465	-0.48466282	0.5	-0.3264464	40.6400545	40.6400545	-0.32644639	0.5	-0.48466282	-40.6400545	
-56.38794851	1.173436055	4.391555774	-62.06905478	0.116114441		-22.84827069	-4.842277998	-18.122107	0.116114441	B	26.56459926	1.173436055	25.27504876	0.116114441	C
				-0.51533718	DF_{AD}				-0.347107217	DF_{BE}				-0.51533718	DF_{CF}
				0.5	COF				0.5	COF				0.5	COF
				100721609.8	K_{ec}				100721609.8	K_{ec}				100721609.8	K_{ec}
-28.13591703	0.586718027	2.195777887	-31.03452739	0.116114441	D	-14.28721712	-5.342277998	-9.0610536	0.116114441	E	13.22424241	0.586718027	12.63752438		F
				Fixed					Fixed					Fixed	

Fig.4.33:-Analysis of design strip

- ***Step 10:- Concrete tensile stress chec.***

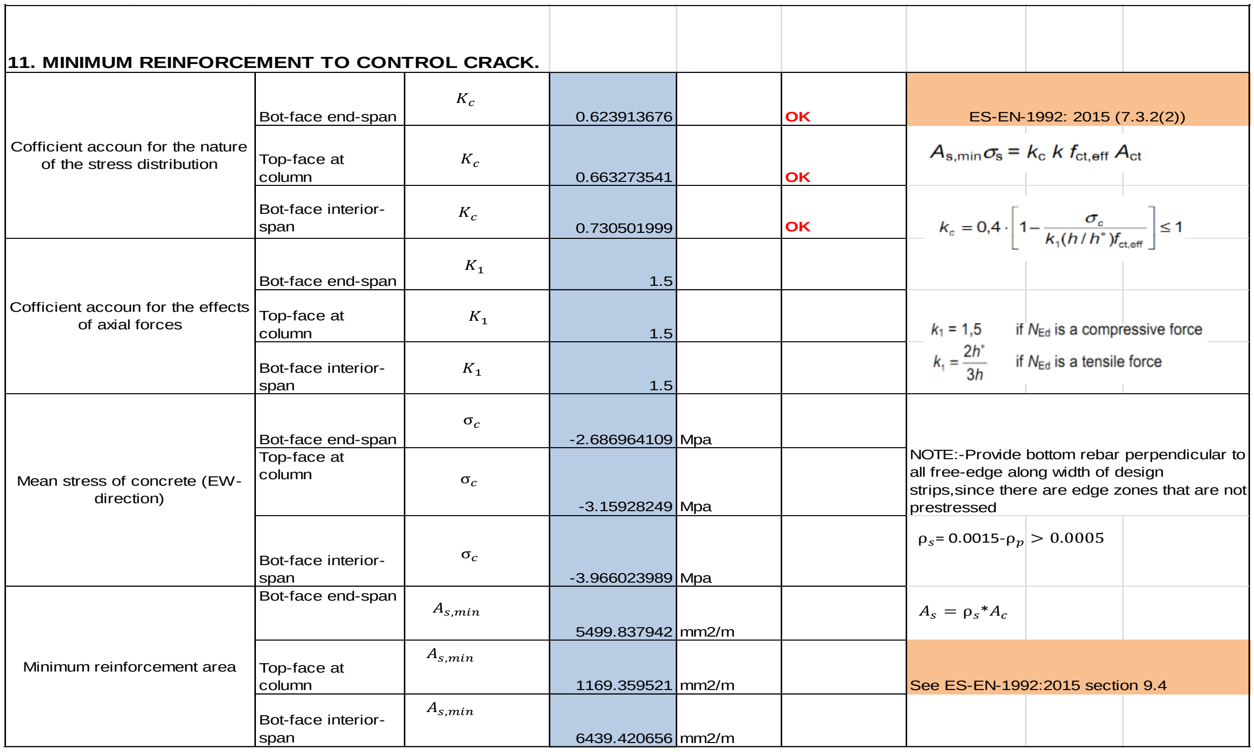


10. CHECK CONCRETE TENSILE STRESS UNDER SERVICE LOADS.

Design strip	Location	Stress calculations				Section Calculation			cement coefficient	Coefficenet depend on concrete age	Coefficient	Tensile strength		ES-EN-1992: 2015 (3.1.2(9))	
		Stress due to Prestressing(Mpa)	Stress due to Prestress moment(Mpa)	Stress due to service load(Mpa)	Total stress(Mpa)	e	Z	I_s	S	$\beta_{cc}(t)$	α	$f_{ctm}(t)$	Remark	Cement type	s-value
NS- Direction	Bot-face end-span	-1.125139936	-1.763140204	0.201316031	-2.68696411	52.23469388	46666.66667	4666666.7	0.2	1.080904274		2.606739861	Good	R	0.2
	Top-face at column	-1.125139936	-1.763140204	-0.27100235	-3.15928249	52.23469388	46666.66667	4666666.7	0.2	1.080904274		2.606739861	Good	N	0.25
	Bot-face interior-span	-1.250155485	-2.503857329	-0.212011175	-3.96602399	66.76122449	46666.66667	4666666.7	0.2	1.080904274		2.606739861	Good	S	0.38
EW- Direction	Bot-face end-span												Not Ok, aditional bond reinforcement required	$\alpha = 1$ for $t < 28$ $\alpha = 2/3$ for $t \geq 28.$	
	Top-face at column												Not Ok, aditional bond reinforcement required	$f_{ctm}(t) = (\beta_{cc}(t))^{\alpha} \cdot f_{ctm}$	
	Bot-face interior-span												Not Ok, aditional bond reinforcement required		

Fig.4.34:-Concrete tensile stress

- ***Step 11:- Crack control reinforcement***



11. MINIMUM REINFORCEMENT TO CONTROL CRACK.

Cofficient accoun for the nature of the stress distribution	Bot-face end-span	K_c	0.623913676		OK	ES-EN-1992: 2015 (7.3.2(2))
	Top-face at column	K_c	0.663273541		OK	$A_{s,min}\sigma_s = k_c\, k\, f_{ct,eff}\, A_{ct}$
	Bot-face interior-span	K_c	0.730501999		OK	$k_c = 0,4 \cdot \left[1 - \frac{\sigma_c}{k_1(h/h^*)f_{ct,eff}}\right] \leq 1$
Cofficient accoun for the effects of axial forces	Bot-face end-span	K_1	1.5			
	Top-face at column	K_1	1.5			$k_1 = 1,5$ if N_{Ed} is a compressive force $k_1 = \frac{2h^*}{3h}$ if N_{Ed} is a tensile force
	Bot-face interior-span	K_1	1.5			
Mean stress of concrete (EW-direction)	Bot-face end-span	σ_c	-2.686964109	Mpa		
	Top-face at column	σ_c	-3.15928249	Mpa		NOTE:-Provide bottom rebar perpendicular to all free-edge along width of design strips,since there are edge zones that are not prestressed
	Bot-face interior-span	σ_c	-3.966023989	Mpa		$\rho_s = 0.0015 - \rho_p > 0.0005$
Minimum reinforcement area	Bot-face end-span	$A_{s,min}$	5499.837942	mm2/m		$A_s = \rho_s * A_c$
	Top-face at column	$A_{s,min}$	1169.359521	mm2/m		See ES-EN-1992:2015 section 9.4
	Bot-face interior-span	$A_{s,min}$	6439.420656	mm2/m		

Fig.4.35:-Crack control rebar

- ***Step 12:- Flexural strength check***

12. CHECK FLEXURAL STRENGTH.

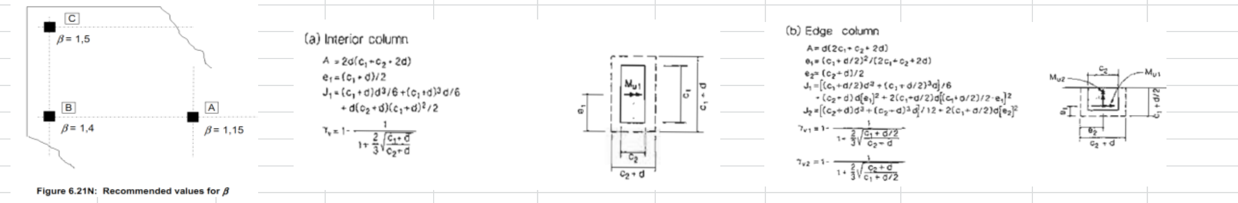
Design strip	Location	Tendon area total in the section	Min.rebar area	Tendon depth	Depth of rebar	Individual tendon stress	Stress block depth	Value of ϕ	Design flexural strength	Factored moment	Remark
		A_{ps}	$A_{s,min}$	d_p	d_s	f_{ps}	a	ϕ	ϕM_n(KNm)	M_u(KNm)	
NS-Direction	End-span column-strip	945	5499.837942	162	181	1250.155485	16.59331952	0.85	488.5957875	-474.259562	OK
	End-span middle-strip	315	5499.837942	148	181	1250.155485	12.81436693	0.85	385.3009505	352.307103	OK
	Interior-span column-strip	1050	6439.420656	116.3	181	1250.155485	19.0895342	0.85	507.6366876	475.086792	OK
	Interior-span middle-strip	350	6439.420656	119.1666667	181	1250.155485	14.89069798	0.85	434.8319482	-292.361103	OK

Fig.4.36:-Flexural strength check

- ***Step 13:- Shear and moment transfer strength checks***

13. CHECK SHEAR AND MOMENT TRANSFER STRENGTH.

Column location	Critical perimeter	Critical area	Tributary area			Critical section properties				Unbalanced moment EW-direction
NS- Direction	U_1(mm)	A_c(mm2)	A_t(mm2)	e_1(mm)	e_2(mm)	J_1(mm4)	J_2(mm4)	γ_{v1}	γ_{v2}	M_{u1}(KNm)
Interior column	2800	640000	49000000	400	400	69333333333	69333333333	0.4	0.4	
Edge column	2400	440000	22050000	222.7272727	400	1.12148E+11	9290666667	0.384088378	0.41612544	
Corner column	2000	280000	11025000	175	175	49058333333	49058333333	0.4	0.4	



Unbalanced moment NE-direction	Factored shear force in control perimeter	Support load eccentricity factor	Factored shear stress	Maximum factored shear stress	Punching shear stress capacity with out shear rebar	Punching shear stress capacity with shear rebar	Perimeter	Remark
M_{u2}(KNm)	F_{vf}(KN)	β	V_f(Mpa)	$V_{max,f}$(Mpa)	$V_{Rd,c}$(Mpa)	$V_{Rd,cs}$(Mpa)	$U_{out,ef}$(mm)	
	644.1552	1.15	1.150277143	1.322818714	0.79507435	0.596305763	4658.548474	Not Ok, Provide shear rebar
	287.8452	1.4	0.5996775	0.8395485	0.782572795	0.586929596	2574.73351	Not Ok,Provide shear rebar
	143.1234	1.5	0.3578085	0.53671275	0.782572795	0.586929596	1371.662172	OK

ES-EN-1992: 2015 (6.4.4 & 6.4.5)

$v_{Rd,c} = C_{Rd,c}\, k\, (100 \rho_l f_{ck})^{1/3} + 0{,}10\sigma_{cp} \geq (v_{min} + 0{,}10\sigma_{cp})$

where:

f_{ck} is in MPa

$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 \quad d$ in mm

$\rho_l = \sqrt{\rho_{ly} \cdot \rho_{lz}} \leq 0{,}02$

$v_{Ed} = \beta \frac{V_{Ed}}{u_1 d}$

$v_{Ed} = \frac{\beta V_{Ed}}{u_0 d} \leq v_{Rd,max} = 0{,}5 \nu f_{cd}$

where

u_0 for an interior column u_0 = length of column periphery

for an edge column $u_0 = c_2 + 3d \leq c_2 + 2c_1$

for a corner column $u_0 = 3d \leq c_1 + c_2$

c_1, c_2 are the column dimensions as shown in Figure 6.20

ν see Expression (6.6)

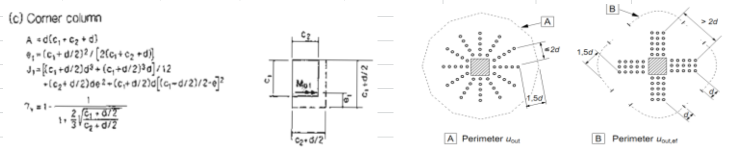
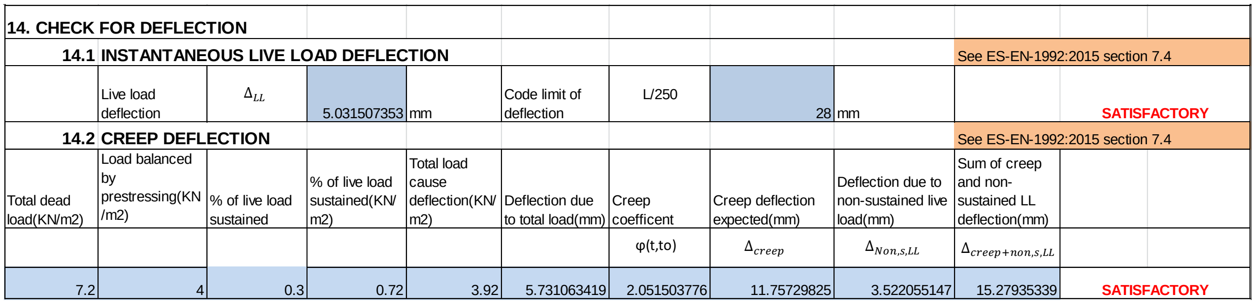


Figure 6.22: Control perimeters at internal columns

Fig.4.37:-Shear and moment transfer

- ***Step 14:- Deflection check***



14. CHECK FOR DEFLECTION											
14.1	**INSTANTANEOUS LIVE LOAD DEFLECTION**								See ES-EN-1992:2015 section 7.4		
	Live load deflection	Δ_{LL}	5.031507353	mm	Code limit of deflection	L/250	28	mm		SATISFACTORY	
14.2	**CREEP DEFLECTION**								See ES-EN-1992:2015 section 7.4		
Total dead load(KN/m2)	Load balanced by prestressing(KN/m2)	% of live load sustained	% of live load sustained(KN/m2)	Total load cause deflection(KN/m2)	Deflection due to total load(mm)	Creep coefficent	Creep deflection expected(mm)	Deflection due to non-sustained live load(mm)	Sum of creep and non-sustained LL deflection(mm)		
						φ(t,to)	Δ_{creep}	$\Delta_{Non,s,LL}$	$\Delta_{creep+non,s,LL}$		
7.2	4	0.3	0.72	3.92	5.731063419	2.051503776	11.75729825	3.522055147	15.27935339	SATISFACTORY	

Fig.4.38:-Deflection check

4.6.3.5 The layout

Designing and detailing with constructability in mind will improved the success of post-tensioned concrete project. The constructability aspect is arguably the most important component of a post-tensioned concrete project. There are several code provisions and recommended guidelines to help facilitate construction through design.

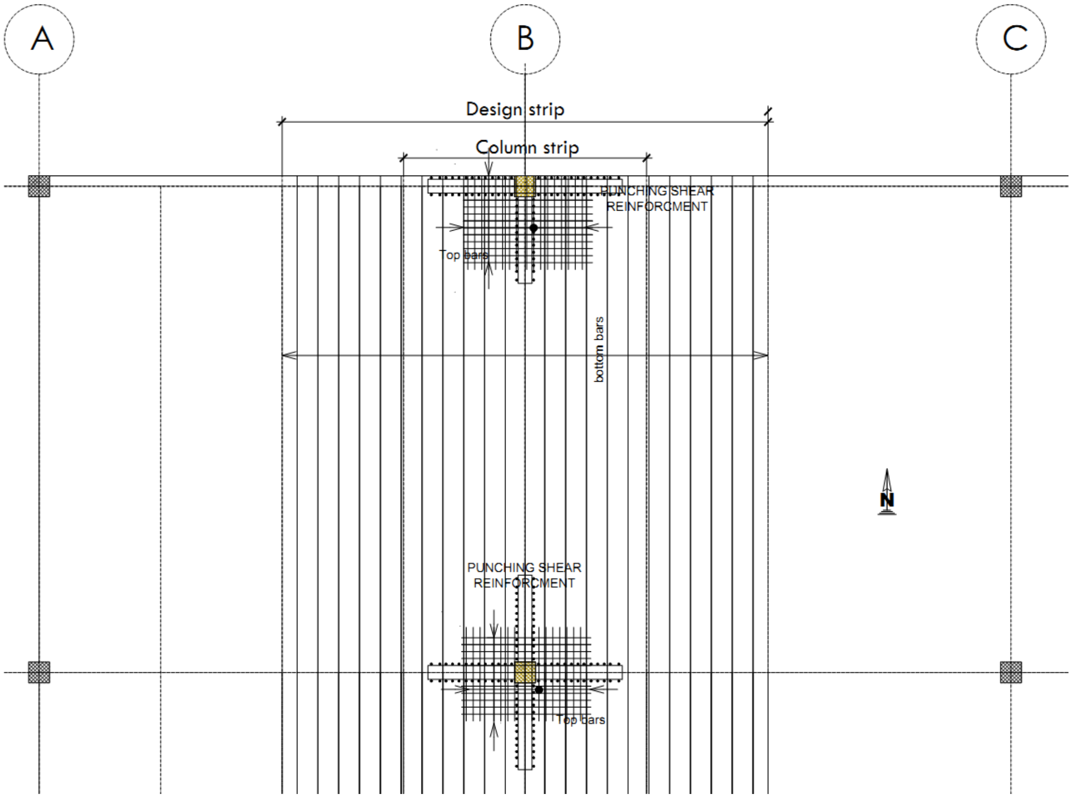


Fig.4.39:-Typical reinforcement bar layout in NS-direction

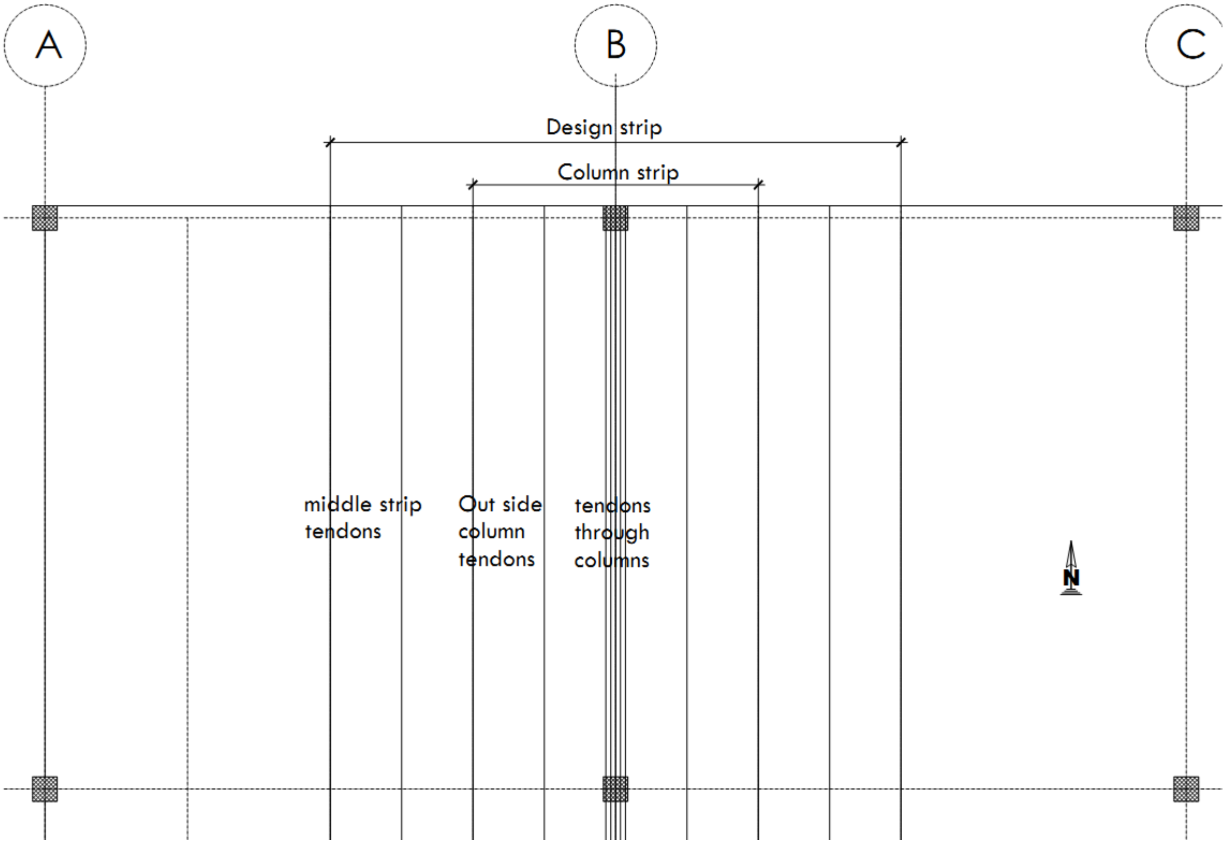


Fig.4.40:-Typical tendons layout in NS-direction

4.7 Analysis of Post-tension flat slab and cost comparison using ETABS by Euro code

4.7.1 Methodology

This study presents the structural engineering calculations of a post-tensioned flat slab supported on columns and reinforced concrete flat slab, using Eurocode-2002. The general criteria used for the design, such as details of materials properties are given and features of the geometry and details of loading applicable to the floor slab covered by this study. The design uses the program ETABS 2016 and the analysis and design are done by equivalent frame method.

The results are tabulated and compared. The cost analysis of the post tensioned and reinforced concrete flat slab are calculated and compared, based on the comparisons the discussions and recommendations are drawn.

For the application of design procedure office building (G+7) typical floor 6 with dimensions 28.40m*21.40m, with largest spans of 7m*7m flat slab. The column dimensions are 400*400mm is consider as a comparison with the output of reinforced flat slab. This building is designed by considering typical design strip shown below. For each case the outputs are presented in tabular form for comparison along with tendon and reinforcement details.

Grade of concrete considered is $C_{35/40}$, grade for tendons is A416Gr270 and grade for rebar is A615Gr60.

4.7.2 Design Inputs

Table 4.6:-ETABS material inputs

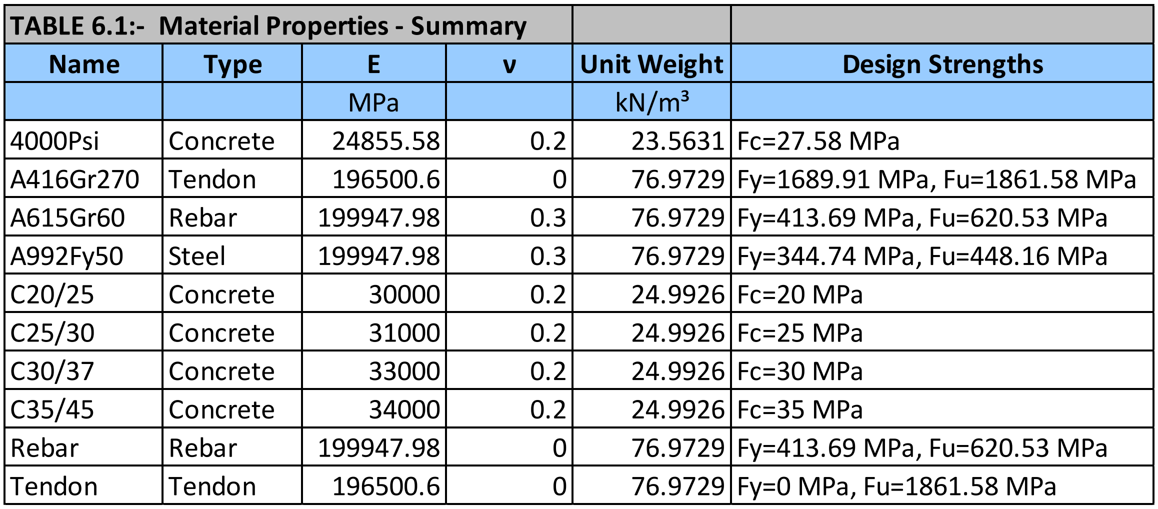


TABLE 6.1:- Material Properties - Summary

Name	Type	E	ν	Unit Weight	Design Strengths
		MPa		kN/m³	
4000Psi	Concrete	24855.58	0.2	23.5631	Fc=27.58 MPa
A416Gr270	Tendon	196500.6	0	76.9729	Fy=1689.91 MPa, Fu=1861.58 MPa
A615Gr60	Rebar	199947.98	0.3	76.9729	Fy=413.69 MPa, Fu=620.53 MPa
A992Fy50	Steel	199947.98	0.3	76.9729	Fy=344.74 MPa, Fu=448.16 MPa
C20/25	Concrete	30000	0.2	24.9926	Fc=20 MPa
C25/30	Concrete	31000	0.2	24.9926	Fc=25 MPa
C30/37	Concrete	33000	0.2	24.9926	Fc=30 MPa
C35/45	Concrete	34000	0.2	24.9926	Fc=35 MPa
Rebar	Rebar	199947.98	0	76.9729	Fy=413.69 MPa, Fu=620.53 MPa
Tendon	Tendon	196500.6	0	76.9729	Fy=0 MPa, Fu=1861.58 MPa

4.7.3 Design Criteria

- **Load Patterns:-**

Table 4.7:-ETABS Load patterns

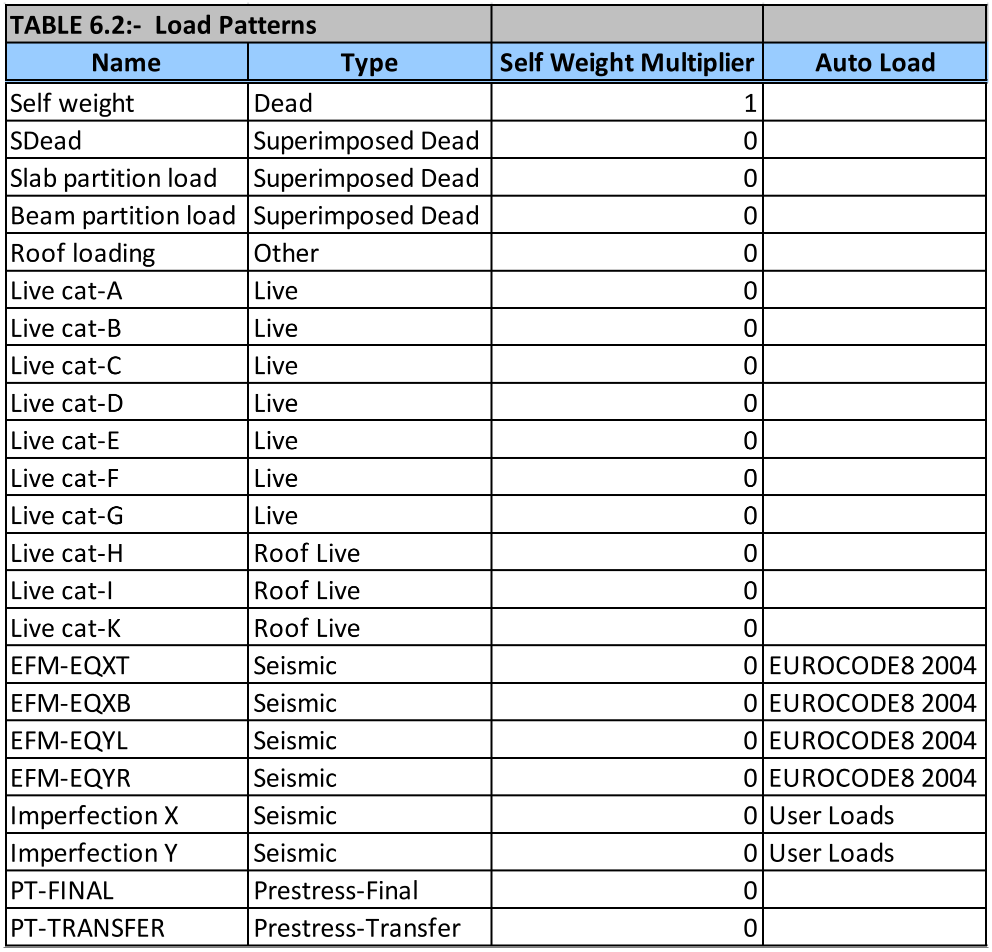


TABLE 6.2:- Load Patterns

Name	Type	Self Weight Multiplier	Auto Load
Self weight	Dead	1	
SDead	Superimposed Dead	0	
Slab partition load	Superimposed Dead	0	
Beam partition load	Superimposed Dead	0	
Roof loading	Other	0	
Live cat-A	Live	0	
Live cat-B	Live	0	
Live cat-C	Live	0	
Live cat-D	Live	0	
Live cat-E	Live	0	
Live cat-F	Live	0	
Live cat-G	Live	0	
Live cat-H	Roof Live	0	
Live cat-I	Roof Live	0	
Live cat-K	Roof Live	0	
EFM-EQXT	Seismic	0	EUROCODE8 2004
EFM-EQXB	Seismic	0	EUROCODE8 2004
EFM-EQYL	Seismic	0	EUROCODE8 2004
EFM-EQYR	Seismic	0	EUROCODE8 2004
Imperfection X	Seismic	0	User Loads
Imperfection Y	Seismic	0	User Loads
PT-FINAL	Prestress-Final	0	
PT-TRANSFER	Prestress-Transfer	0	

- **Load cases:-**

Table 4.8:-Load cases

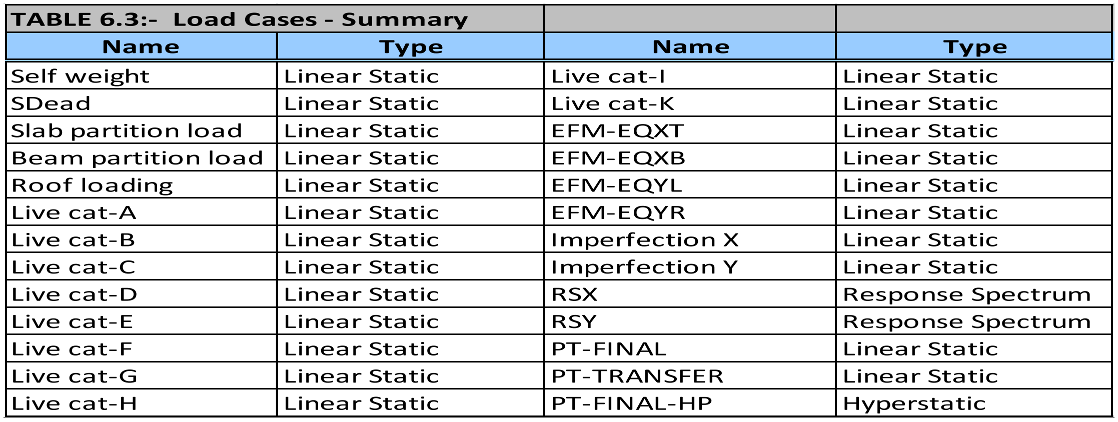


TABLE 6.3:- Load Cases - Summary

Name	Type	Name	Type
Self weight	Linear Static	Live cat-I	Linear Static
SDead	Linear Static	Live cat-K	Linear Static
Slab partition load	Linear Static	EFM-EQXT	Linear Static
Beam partition load	Linear Static	EFM-EQXB	Linear Static
Roof loading	Linear Static	EFM-EQYL	Linear Static
Live cat-A	Linear Static	EFM-EQYR	Linear Static
Live cat-B	Linear Static	Imperfection X	Linear Static
Live cat-C	Linear Static	Imperfection Y	Linear Static
Live cat-D	Linear Static	RSX	Response Spectrum
Live cat-E	Linear Static	RSY	Response Spectrum
Live cat-F	Linear Static	PT-FINAL	Linear Static
Live cat-G	Linear Static	PT-TRANSFER	Linear Static
Live cat-H	Linear Static	PT-FINAL-HP	Hyperstatic

- **Load Combinations:-**

Table 4.9:-Load combinations

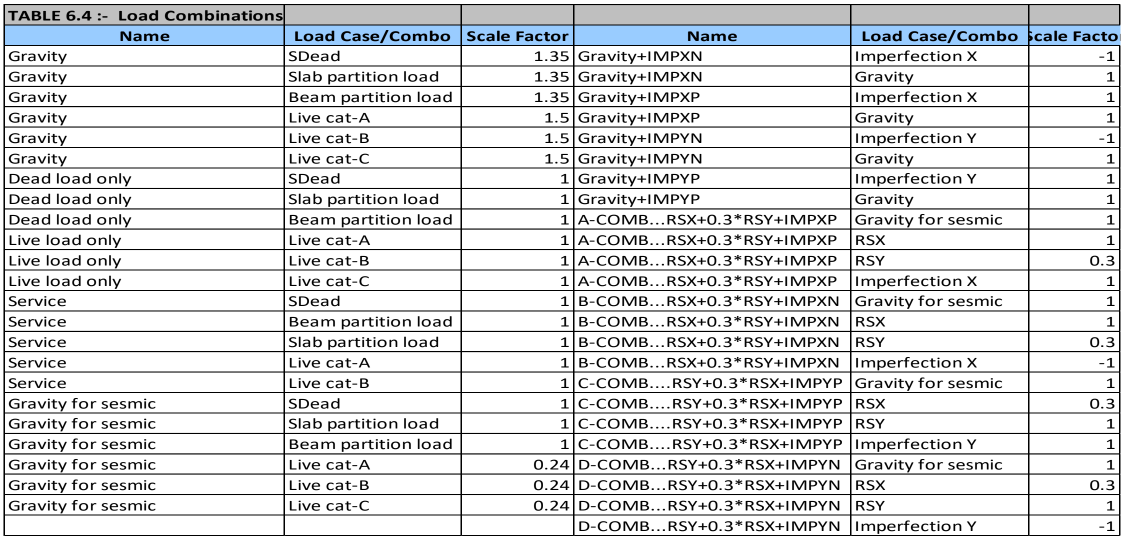


TABLE 6.4 :- Load Combinations

Name	Load Case/Combo	Scale Factor	Name	Load Case/Combo	Scale Facto
Gravity	SDead	1.35	Gravity+IMPXN	Imperfection X	-1
Gravity	Slab partition load	1.35	Gravity+IMPXN	Gravity	1
Gravity	Beam partition load	1.35	Gravity+IMPXP	Imperfection X	1
Gravity	Live cat-A	1.5	Gravity+IMPXP	Gravity	1
Gravity	Live cat-B	1.5	Gravity+IMPYN	Imperfection Y	-1
Gravity	Live cat-C	1.5	Gravity+IMPYN	Gravity	1
Dead load only	SDead	1	Gravity+IMPYP	Imperfection Y	1
Dead load only	Slab partition load	1	Gravity+IMPYP	Gravity	1
Dead load only	Beam partition load	1	A-COMB...RSX+0.3*RSY+IMPXP	Gravity for sesmic	1
Live load only	Live cat-A	1	A-COMB...RSX+0.3*RSY+IMPXP	RSX	1
Live load only	Live cat-B	1	A-COMB...RSX+0.3*RSY+IMPXP	RSY	0.3
Live load only	Live cat-C	1	A-COMB...RSX+0.3*RSY+IMPXP	Imperfection X	1
Service	SDead	1	B-COMB...RSX+0.3*RSY+IMPXN	Gravity for sesmic	1
Service	Beam partition load	1	B-COMB...RSX+0.3*RSY+IMPXN	RSX	1
Service	Slab partition load	1	B-COMB...RSX+0.3*RSY+IMPXN	RSY	0.3
Service	Live cat-A	1	B-COMB...RSX+0.3*RSY+IMPXN	Imperfection X	-1
Service	Live cat-B	1	C-COMB....RSY+0.3*RSX+IMPYP	Gravity for sesmic	1
Gravity for sesmic	SDead	1	C-COMB....RSY+0.3*RSX+IMPYP	RSX	0.3
Gravity for sesmic	Slab partition load	1	C-COMB....RSY+0.3*RSX+IMPYP	RSY	1
Gravity for sesmic	Beam partition load	1	C-COMB....RSY+0.3*RSX+IMPYP	Imperfection Y	1
Gravity for sesmic	Live cat-A	0.24	D-COMB...RSY+0.3*RSX+IMPYN	Gravity for sesmic	1
Gravity for sesmic	Live cat-B	0.24	D-COMB...RSY+0.3*RSX+IMPYN	RSX	0.3
Gravity for sesmic	Live cat-C	0.24	D-COMB...RSY+0.3*RSX+IMPYN	RSY	1
			D-COMB...RSY+0.3*RSX+IMPYN	Imperfection Y	-1

4.7.4 Structural Plan

The following consecutive figures shows the structural model of the slab, the outline of the post-tensioned geometry, and 3D view of the post-tensioned slab and its support arrangement in the analysis model used for analysis and design.

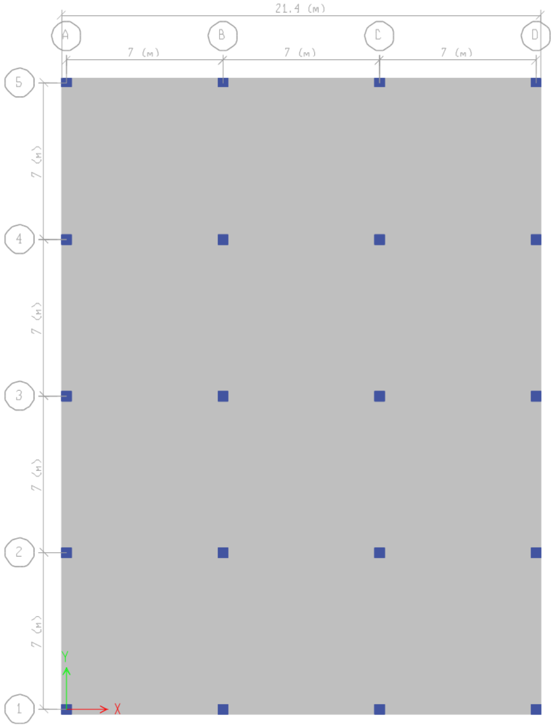


Fig.4.41:-General Layout of the floor system

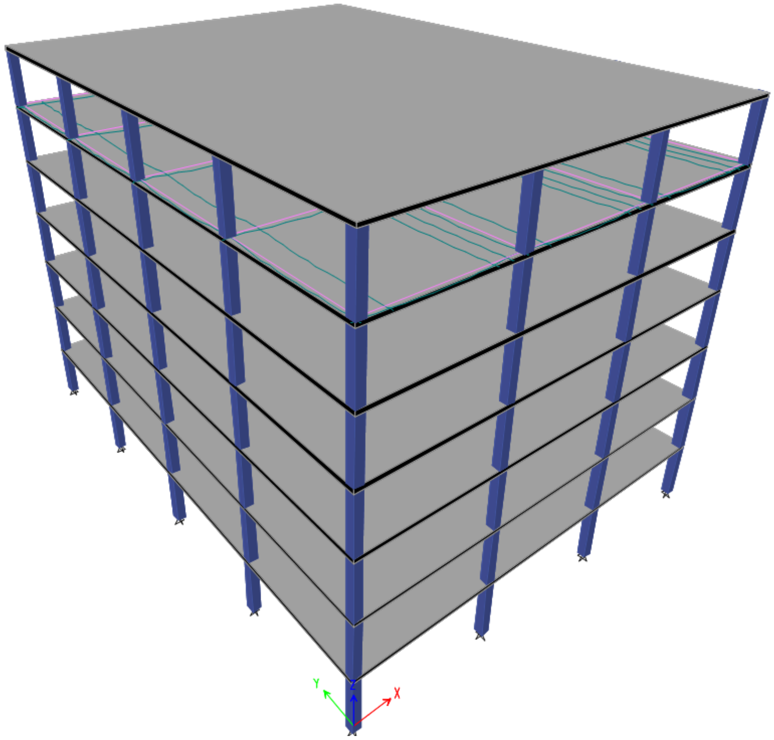


Fig.4.42:- 3D view of the post-tensioned slab and its support arrangement.

4.7.5 Loading Assumptions and Values

For the design criteria, the post-tensioned slab is designed for a uniformly distributed super imposed dead load of 1.2KN/m^2 and uniformly distributed live load of 2.50KN/m^2. The loads generally in the design model are illustrated in the following views from the analysis model.

4.7.6 Reinforcement plan from computation

The figures below show the tendon layout and reinforcement as reported by the software in both cases PT flat slab and R.C flat slab in A and B strip directions respectively.

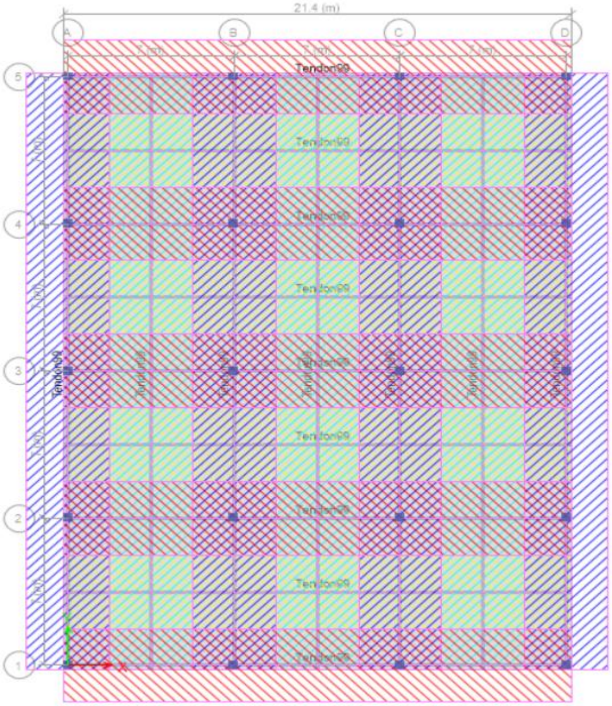


Fig. 4.43: - PT flat slab Tendons layout in layer A and B strip directions

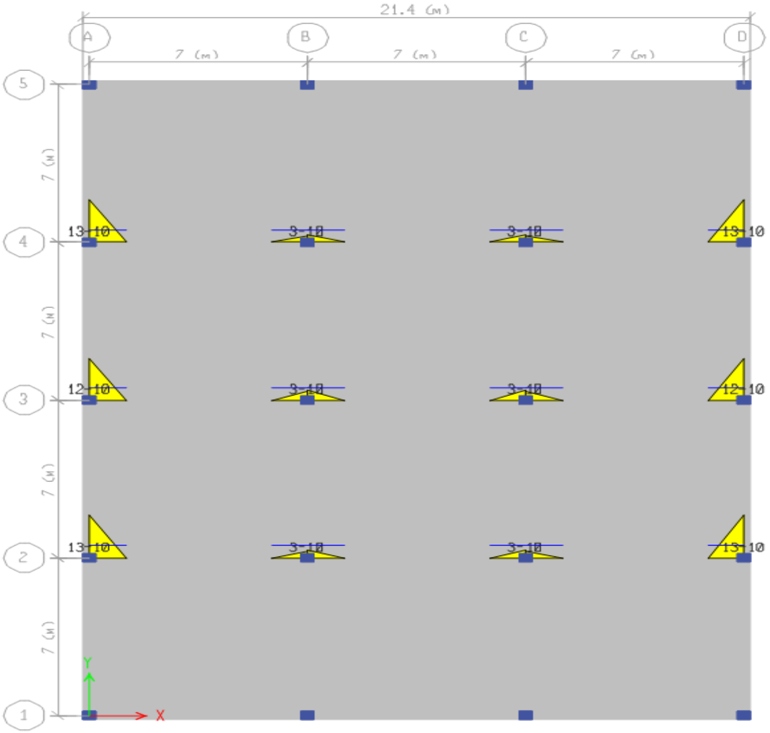


Fig. 4.44: - PT flat slab Top reinforcement layout in layer A strip direction

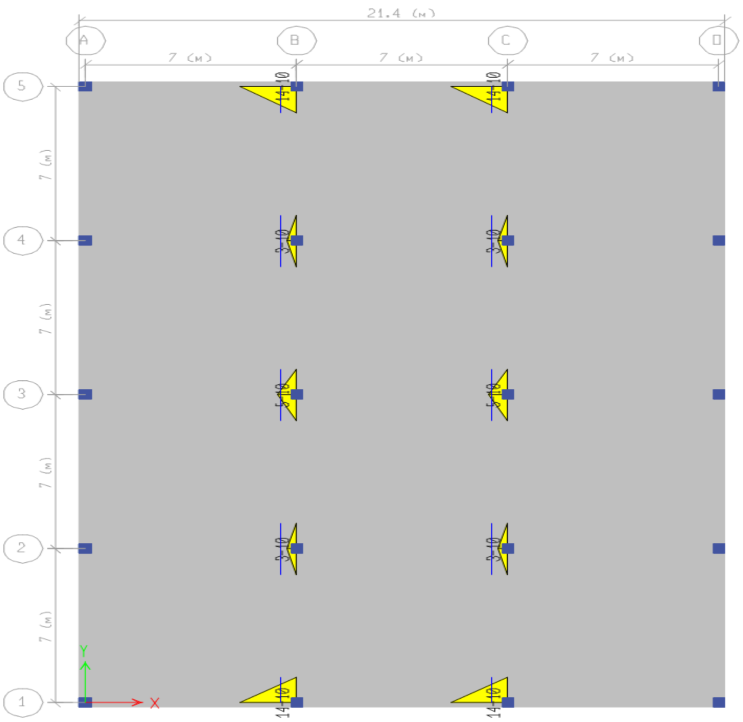


Fig. 4.45: - PT flat slab Top reinforcement layout in layer B strip direction

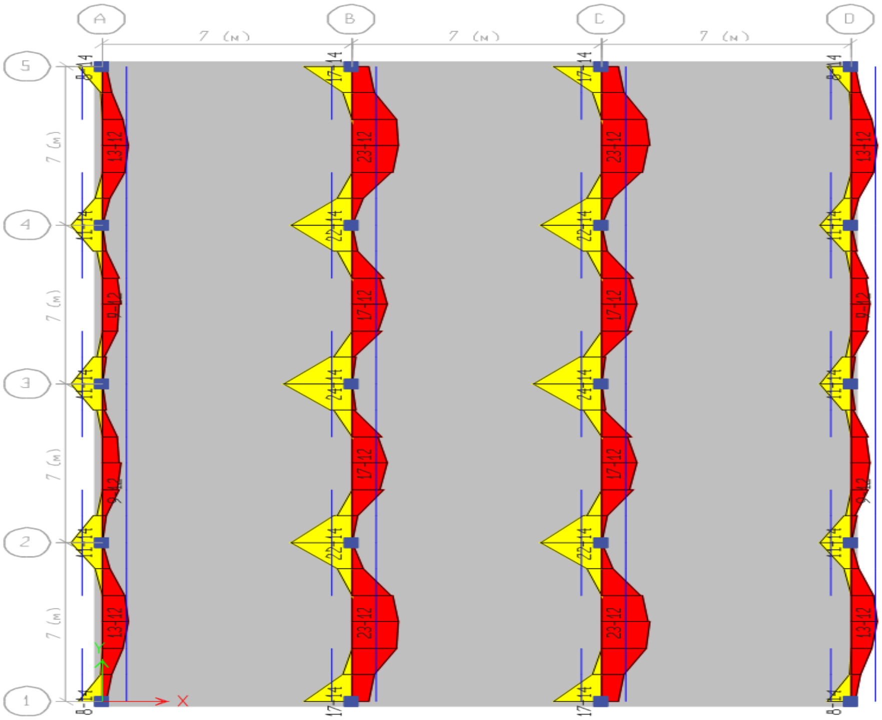


Fig. 4.46: - R.C flat slab Top and Bottom reinforcement layout in layer B strip direction

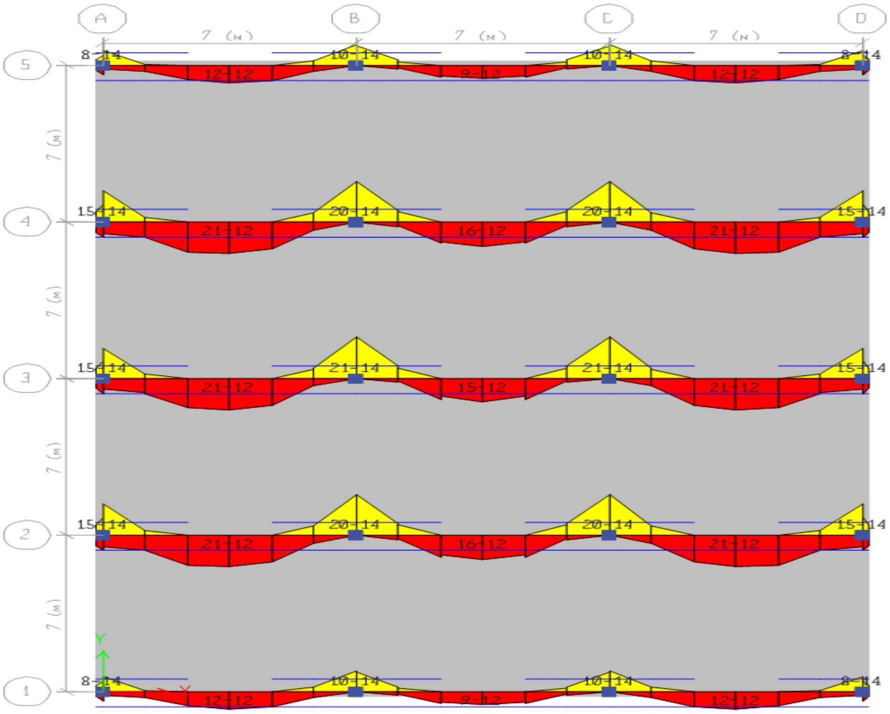


Fig. 4.47: - R.C flat slab Top and Bottom reinforcement layout in layer A strip direction

4.7.7 Comparison of cost

For the analysis and design results of the office building the total estimation for a typical floor is calculated. The quantities of concrete, reinforcing steel, pre-stressing steel and their cost according to the current Alibaba.com online rate excluding the labor charges for the two cases and the results are tabulated and compared. Therefore based on comparisons the conclusions are drawn.

4.7.7.1 Steel comparisons

Table 4.10:- R.C flat slab rebar steel kg and its cost in A and B strip directions

	No of bars	Length(m)	Kg
	A strip direction		
Top(Φ14)	144	2.33	402.624
	192	4.66	1073.66
Bottom(Φ12)	92	21.4	1771.92
	B strip direction		
Top(Φ14)	174	2.33	486.5
	378	4.66	2113.77
Bottom(Φ12)	92	28.4	2351.52
	Total		8200.0
	Kg/sqm		13.49
	Cost/kg	0.45USD	
	Total cost	3690 USD	

Table 4.11:- PT flat slab rebar steel kg and its cost in A and B strip directions

	No of bars	Length(m)	Kg
	A strip direction		
Top(Φ10)	112	2.33	169.62
	40	4.66	121.16
Bottom(Φ8)	113	21.4	798.00
	B strip direction		
Top(Φ10)	84	2.33	127.21
	45	4.66	136.30
Bottom(Φ8)	86	28.4	805.99
	Total		2,158.3
	Kg/sqm		3.55
	Cost/kg	0.45USD	
	Total cost	971.23USD	

Table 4.12:- PT flat slab pre-stressed tendon kg and its cost in A and B strip directions

	No of tendons	Length(m)	Kg
	A strip direction		
7 strand Bonded tendons	7	23	161
	B strip direction		
7 strand Bonded tendons	15	30	450
	Total		611
	Cost/kg	0.395USD	
	Total cost	241.345USD	

Table 4.13:- PT and R.C flat slab total steel cost comparison in USD

	Slab area(m2)	Total cost(USD)
R.C flat slab	607.76	3690
PT flat slab	607.76	1212.57

4.7.7.2 Concrete comparisons

Table 4.14:- PT and R.C flat slab concrete volume comparison

	Slab depth(mm)	Slab area(m2)	Slab volume(m3)
R.C flat slab	250	607.76	151.94
PT flat slab	200	607.76	121.552

4.7.8 Analysis result and Discussions

1. From the quantity estimations and costing it is observed that concrete needed for R.C flat slab construction is 151.92m3 and that for PT flat slab is 121.55m3. Cost of steel required for the R.C flat slab construction is 3690USD and the cost of steel and tendons required for PT flat slab construction is 1212.27USD.
2. Hence, for the structure considered, as the concrete needed and the cost of steel required is much less in case of PT flat slab construction than in the case of R.C flat slab construction, it is more economical to construct the structure considered with PT flat slab than the R.C flat slab.
3. When a concrete flat slab is stressed by the post-tensioning method, it means the steel is being tensioned and the concrete is being compressed. As a building material, concrete is very strong in compression but relatively weak in tension. Steel is very strong in tension. Putting a concrete slab into compression and the steel into tension.
4. There are many other benefits of using PT flat slab. As the thickness of the slab is much lesser than the R.C flat slab, aesthetic look of the building may get enhanced leading to a clear height for a longer distance. Hence, using a PT slab is more advisable for a commercial building than using a R.C flat slab. Construction of a structure using PT flat slab also leads to a lighter structure as the dead load gets reduced this intern increase the story number within a given foundation.
5. Putting a concrete slab into compression and the steel into tension, before any substantial service loads are applied puts both building materials into their strongest states. The result is a “stiffer concrete slab” that actively is compressed and has more capacity to resist tensile forces. Therefore, the stiffness and strength of the structure using post-tensioned slab will be more than the structure constructed using R.C slab.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATION

Post-tensioned flat slab in tall buildings has been used as alternative of reinforced concrete all over the world for decades. Now, this technology provides efficient solutions for various structural members and situations; floor structure, vertical elements, lateral load resisting system, etc. However, in Ethiopia, prestressed concrete, especially post-tension slab system in buildings, has been used as the major floor system starting from around 2009E.C. Post-tension system is in the process of going to be a mainstream in the Ethiopia construction sector.

The study suggests that measures shall be taken for the top identified factors hindering the wide application of post-tensioned flat slab in our construction industry widely like; Universities has not been covered the topics deeply in under-graduate sections, Shortage of research literatures, seminars and related design examples, Lack of experienced consultant, contractor and other engineering professionals and Lack of professionals and expertise.

Despite the successful use of post-tensioned slab construction in buildings in many countries most owners and engineers remained reluctant to apply this technology in Ethiopia. However, today’s demand for office and commercial buildings will also bring needs for more technology. The post-tensioned slab system is one of extremely suitable and economical to satisfy the needs.

The project on spreadsheet analysis of post-tensioned flat slab using Excel spreadsheet was developed. The program requires the user to input loads, dimensions, and allowable stress limits, and material properties. The program then calculates the required prestressing force, section properties, tendon profile properties, prestress losses and check stress criterion of concrete and tendon, and deflection criterion by using Ethiopian design code.

The analysis process had reduced in its duration and complexity by the interaction of the designer at various stages of the analysis, and the ability to selectively automate those components of the analysis process that were repetitive and time consuming. Proper judgment from the user/designer could be applied and be rectified almost instantaneously. The developed program may serve as academic aid since the computation process was systematically reflected on the spreadsheet.

The post-tensioned flat slab technology of Ethiopia has developed slowly reflecting the characteristics of Ethiopian construction practice. But, however, in the near future, a post-tensioned technology will be one of the most competitive solution for certain limited circumstances in the country. This paper could promote exchange of ideas for sustainable development of post-tensioned floor system in Ethiopia.

ANNEX A

A.1 Implementation Procedures of Post-tensioned flat slab

- Confirm site drawings are the latest revision and approved for construction.
- Complete formwork/false work and stop-ends for relevant concrete pour (by others) and check that access and required work areas are clear and safe for installation requirements.
- Check and confirm that stressing material deliveries are as per delivery docket issued and retain docket in site file.
- Set out location of stressing anchorages and pockets in accordance with the issued ‘for construction’ drawings. If drawing is issued ‘for approval’ only, confirm with supervisor if OK to use.
- Slot and drill forms (if required).
- Install anchorages onto edge board as per approved drawings. Ensure that all necessary personal protective equipment is worn at all times.
- Mark out tendon spacing and lay duct as approved drawings.
- Lay pre-stressing duct, install any anti-burst reinforcement and tap duct to anchorages.
- Carefully place strand coil in strand frame prior to cutting retaining straps with tin snips. If strand frame needs to be lifted into position (by crane), use lifting frame. If frame has no lifting lugs, ensure slings pass through coil and not off frame only.
- Position strand frame (coil) adjacent to work area and away from thoroughfares. Use safety goggles whenever strand is being cut. Ensure area is of sufficient strength to support weight of coil.
- Push strand for all tendons per drawing details. Ensure pushing area is barricaded and signage is placed to maintain safe work areas. Stand clear of strand being drawn from coil into duct.
- After correct amount of strand has been placed into duct for each tendon, create dead-end (onion) on each strand.
- Straighten and profile duct as per drawing details. Check straights and profile before fixing to deck using staples. Place grout tube into anchor head grout port, and into end of duct at dead end.
- Ensure that anchorage are positioned according to the design and fixed so they do not move during normal construction activities. Ensure that anti-burst reinforcement is present and fixed central to anchorage zone.
- Check installation has been completed to the best of your ability, and advice design engineer or builder/contractor that installation is complete. And ready for engineer’s inspection.
- At completion of concrete pour, check that all anchors are clear and slurry etc. check that test cylinders have been taken during pour and stored on site in conditions similar to slab being poured.

- After edge board/pan has been stripped, grease holes in anchor block/coupling block. Place anchorage components over protruding strands and hard up against casting. Place two-piece wedges such join is vertical in seat. Seat firmly with hammer.
- Check stress jack and gauge are accompanied by valid and up-to-date calibration (kept in gauge box).check nose on jack (or curved nose) to ensure recess is maintained.
- Check the required load on the calibration chart, and confirm the gauge pressure required.
- Stress tendons in order from further to closest reachable position. Place stressing jack into position over chosen strand. Connect hydraulic pump to jack, and pressure gauge to pump.
- Stress tendons to the required load and in accordance with the calibration report and check seating of wedges at live anchorages after lock-off. Ensure that stressing load is applied gradually and evenly.
- Measure tendon extensions, complete standard stressing from and forward to the principal contractor for review and approval.
- At completion of each tendon stressing, ensure barricading and signage is maintained for a period of at least two hours after full stress has been completed. Notify builder/client that no drilling/coring is to be performed on the area until grouting is completed or unless otherwise approved.
- Cut all tendons, ensuring that specified concrete cover can be achieved, using friction cutter, and place off-cuts in bins provided by others.
- Dry pack any edge recesses with sand/cement mix and fill ‘top of slab’ recesses with concrete.
- Grout tendons. Before commencing grouting, all vents/drains should be opened and blown through to check for clear.
- Commence grouting tendon from one end through standard hose connected to dead end grout tube. Grout injection should be continuous, to avoid blockages or the formation of voids in the grout column.
- Outward vent is closed, pressurize duct to minimum of about 100kpa, maximum of 250kpa and hold for a period of 1 minute using pressure relief valve on pump.
- Remove grout tubes not less than 20 hours after tendon grouting is completed. Top up any vents as necessary.
- Clean up works areas and make safe any obstruction caused by stressing works.
- Forward all records and documentation to supervisor/project engineer for filling in project file.

ANNEX B

B.1 Technology of prestressing in Buildings

Application of prestress in building concrete structures poses no major difficulty in comparison with any other type of structure. Nevertheless, in some aspects it has its particularities concerning the type of geometry, loads and/or border constraints; it is the case of thin slab, heavy concentrated loads on transition floors or rigid vertical elements that restrain the presterssed effects.

In this sub topic the basic structural concepts of prestress are reviewed, the main technology aspects of prestressing, especially for thin flat slabs, are presented and the design aspects are described in a brief way.

This section illustrates some examples of post-tensioning systems currently used in building construction.

B.2 The monostrand post- tensioning system with unbounded greased and sheathed strand

For thin construction elements such as: slabs in buildings, construction methods. These light, flexible monostrands can be easily and rapidly installed and as there is no grouting can lead to economical solutions. Each end of the strand is anchored in an individual anchorage device.

The following types of anchorage are available:- stressing anchorage, dead end anchorage and coupler.

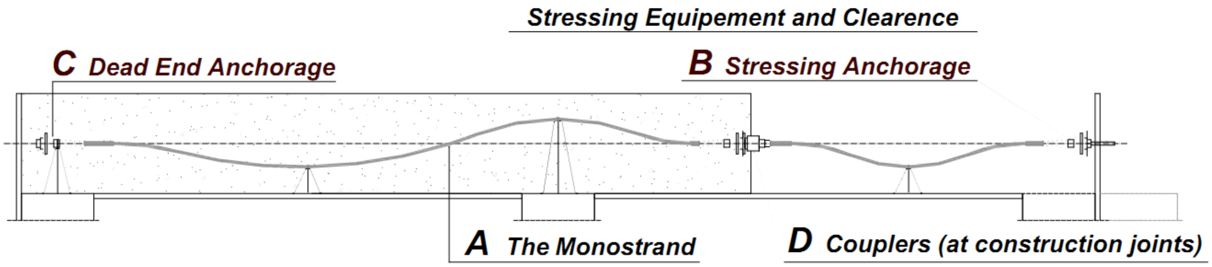


Fig.B.1:- Schematic layout of the mono-strand post-tensioning system

A. The monostrand

The monostrand is a 7-wire strand of patented cold-drawn twisted wires which have been stress-relieved or stabilized. In the factory or workshop, the strand is first given a continuous coating of permanent corrosion preventive grease and then a plastic sheath of either polyethylene or polypropylene is extruded or pushed over the greased strand.

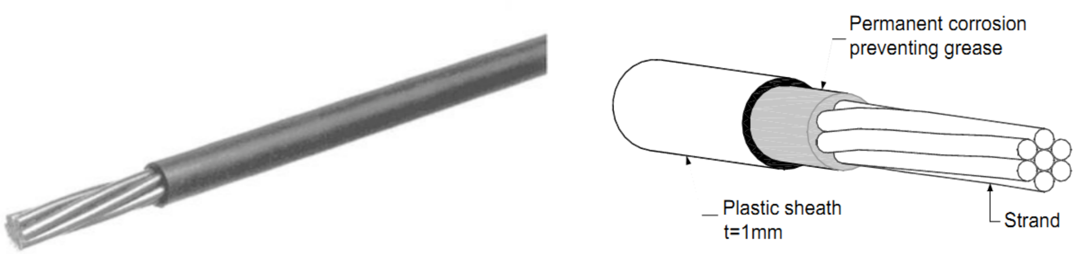


Fig.B.2:- Structure of the monostrand

The quality and dimensions of the materials vary from one country to another and therefore careful attention should be given to the criteria and codes.it is believed the EN standard, which combines the specifications of the materials certified in the member states of CEN (Comite Europeen de Normalisation), will soon become the common standard. The corrosion protection should be in accordance with ES-EN-1992:2015 recommendations.

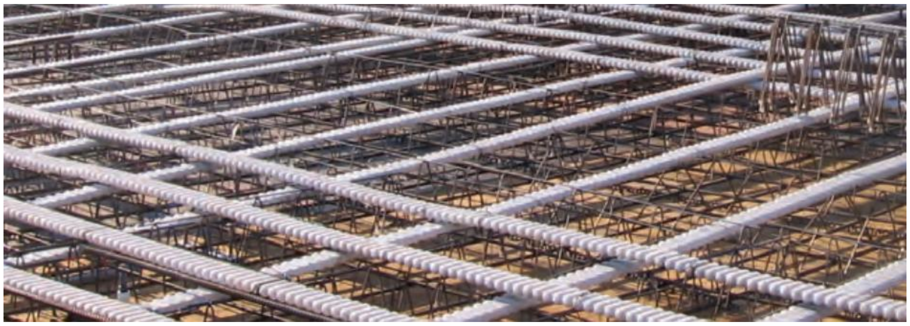


Fig.B.3:- Views of monostrand bundles

B. Stressing anchorage

The components of the stressing anchorages are the anchorage body of cast steel with wedges, a polyethylene sealing sleeve and the recess former. The fixation of the stressing anchorage is done by setting out and making of cable axes on stop-end formwork, drilling a hole diameter 30-35mm for passage of the recess former fastener, then fastening the recess former to the stop end with the lock nut.

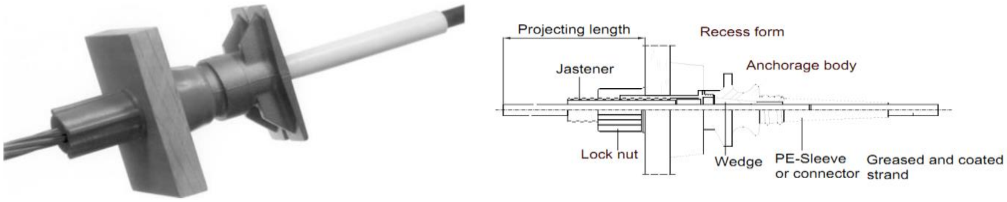


Fig.B.4:- Anchorage elements

The corrosion protection of the strand portion and wedges in the anchorage body is critical. The internal cavity of the anchorage body is therefore injected under pressure with permanent corrosion protective grease and closed by a grease-filled PE-protective cap.

Fig.B.5:-System with enhanced corrosion protective properties

C. Dead end anchorage

The dead end anchorage is identical in appearance to the stressing anchorage. It is usually fitted onto the tendon at the workshop. The wedges are pressed in and secured against backwards movement.

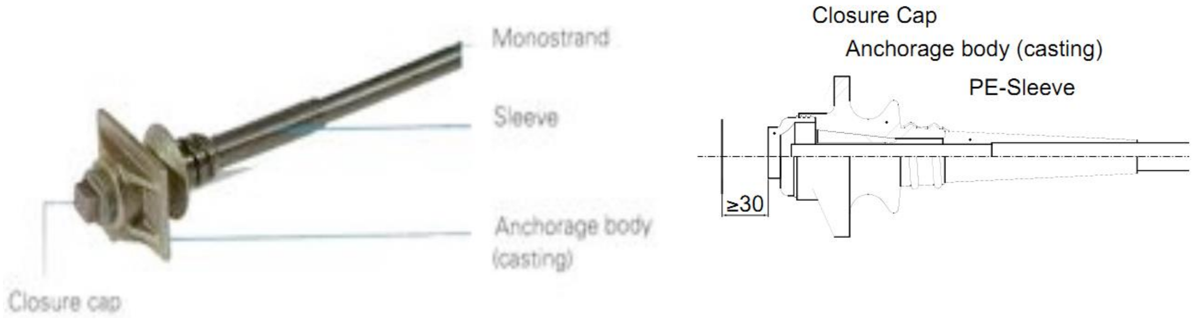


Fig.B.6:- Anchorage elements

D. Couplers

As extensive floor areas are subdivided into smaller manageable pouring stages and post-tensioned in sections, the cables at the construction point are connected with couplers to the cables that have been already stressed.

All the couplers are practically based on the same concept and consist of a coupling body with coupling head and threaded coupling. The coupling head is screwed in the coupling body of the stressed cable. The strand is then inserted in to the self-gripping locking device of the coupling head.

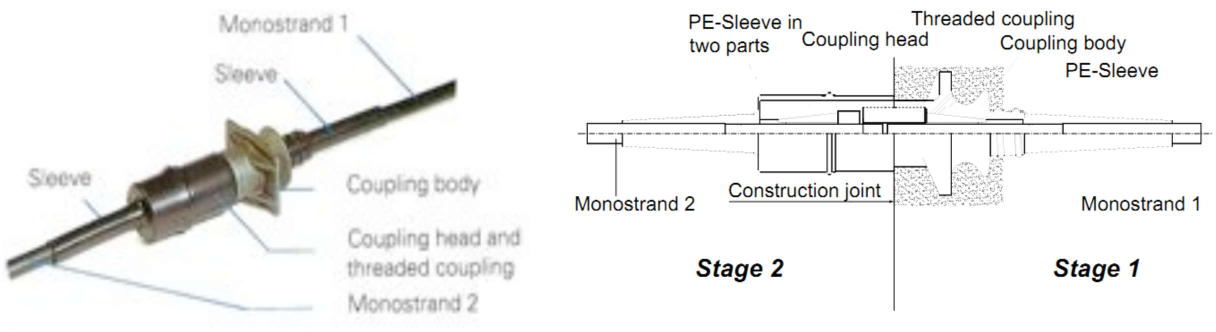


Fig.B.7:- Coupler elements

B.3 The bonded slab post-tensioning system

The multistrand system

The following types of anchorage are available: stressing anchorage, dead end anchorage and coupler.

A. Flat duct

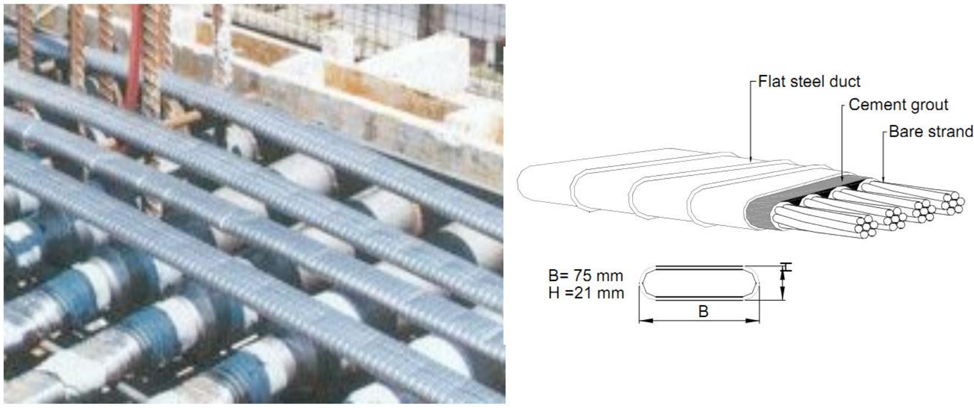


Fig.B.8:- Steel flat ducts

B. **Stressing anchorage**.

Anchorages for flat duct system can be differentiated in three groups:

- <u>One piece</u>: comprising of both bearing plate and anchor head, which is installed into a block-out after concreting; the four strands, which lie alongside one another in the flat tendon are individually threaded through the anchorage and stressed. After concreting, the reusable block-out form is removed with the end formwork.

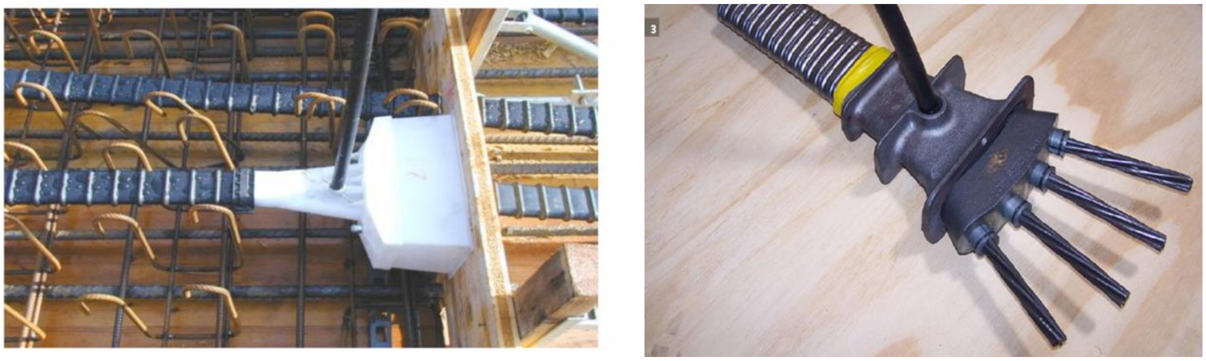


Fig.B.9:- Components of a stressing anchorage of group one.

- <u>One piece</u>: comprising of both bearing casting and anchor head where the casting is installed in the same as a casting for the multi-strand system.

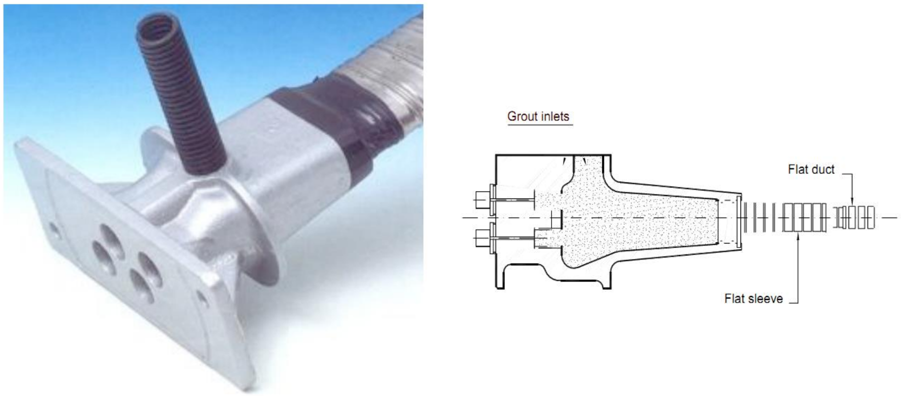


Fig.B.10:- Components of a stressing anchorage of group two.

- <u>Two pieces</u>: with a casting and an anchor block allowing the stressing of four or five strands.

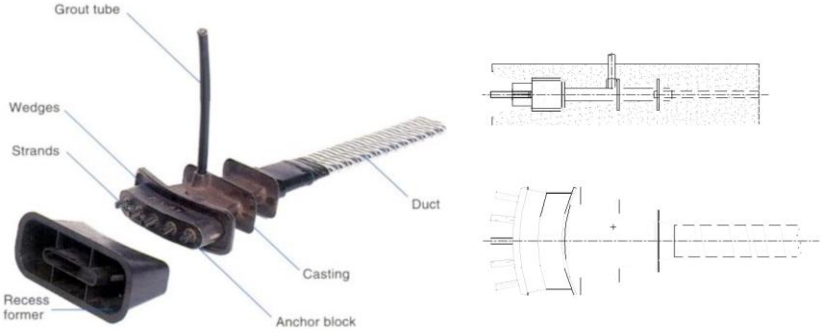


Fig.B.11:- Components of a stressing anchorage of group three.

C. **Dead-end anchorage**.

Where high level of corrosion protection are required and a need for the pre-stressing force to be transferred as near as possible to the end of a structure component, the stressing anchorage can be used as dead end anchorage.

Where the pre-stressing force must only be transferred as near as possible to the end, an anchorage with retainer plates and compression fitting can be used.

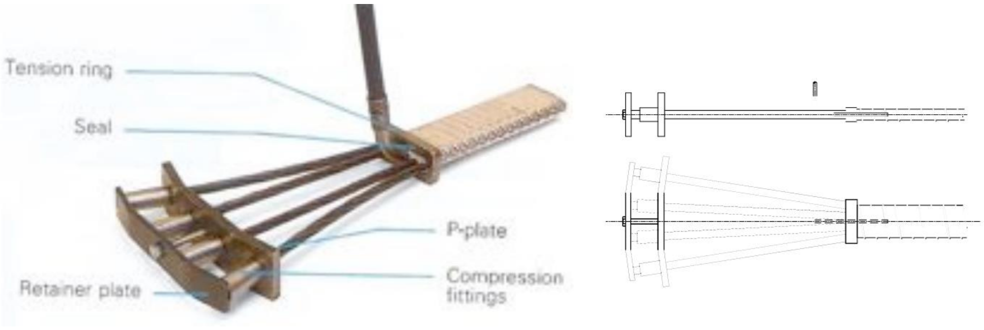


Fig.B.12:- Elements for dead end anchorage with bearing plate.

In other case, the pre-stressing force can be transferred by the bonding of the bare strand and partly by bearing of the bulb at the end of the strands.

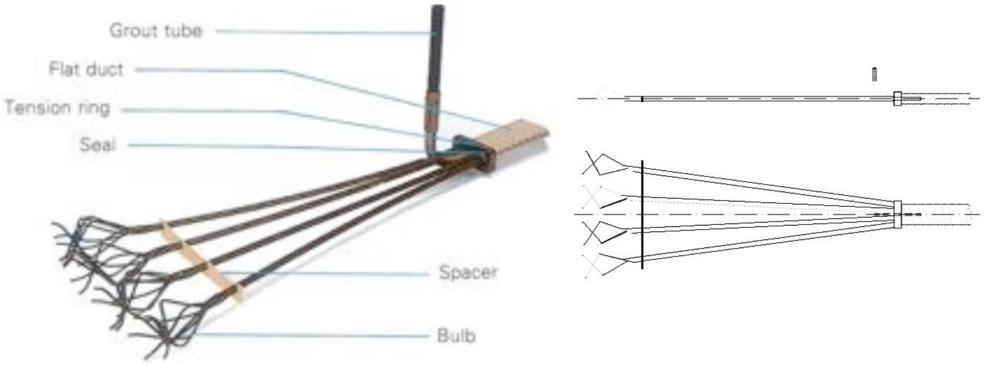


Fig.B.13:- Elements for dead end anchorage by bond.

D. **Couplers.**

The couplers enable a new cable to be connected onto a previously installed and stressed cable.

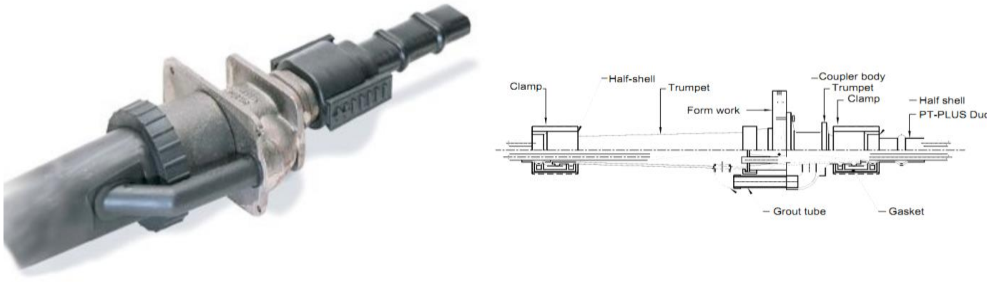
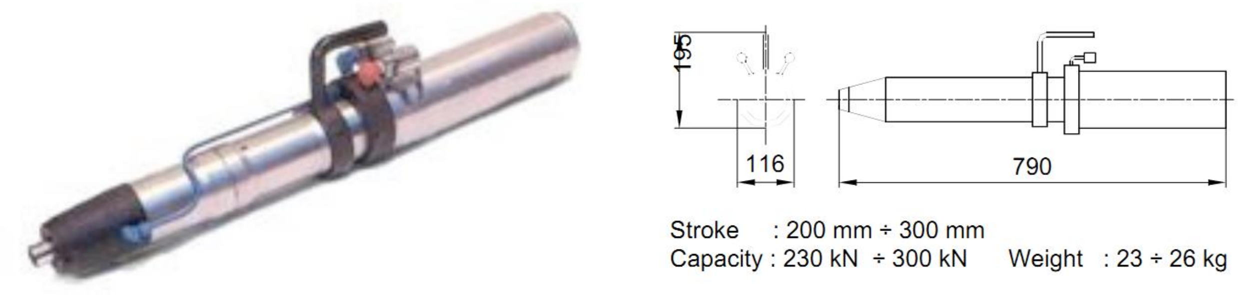


Fig.B.14:- Components of a coupler with high degree of corrosion protection

B.4 Stressing equipment and clearance

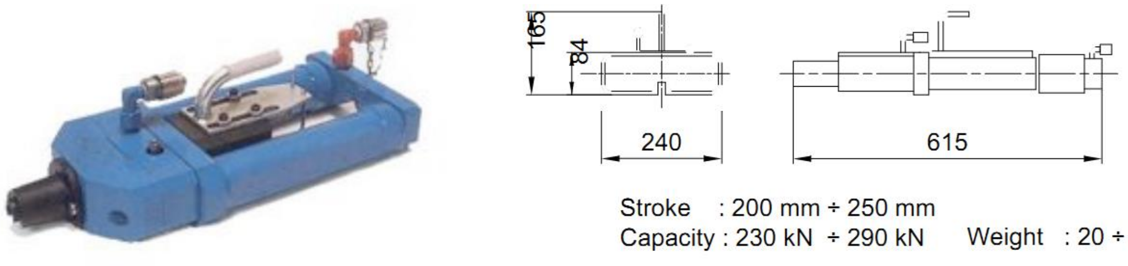
In normal case, the stressing of the strands for post-tensioning in buildings is done with a front-gripping hollow piston jack with a stroke of 200mm and a weight of approximately 20kg.



Stroke : 200 mm ÷ 300 mm
Capacity : 230 kN ÷ 300 kN Weight : 23 ÷ 26 kg

Fig.B.15:- View of the front-gripping hollow piston jack and some characteristics

In special cases such as short clearances (block-outs) or exceptionally for intermediate anchorage with continuous strands, the twin ram jack will be used with a special chair for stressing.



Stroke : 200 mm ÷ 250 mm
Capacity : 230 kN ÷ 290 kN Weight : 20 ÷ 30 kg

Fig.B.16:- View of the twin ram jack

Jacking systems with two to four strands have been developed. (See the Fig C.17).

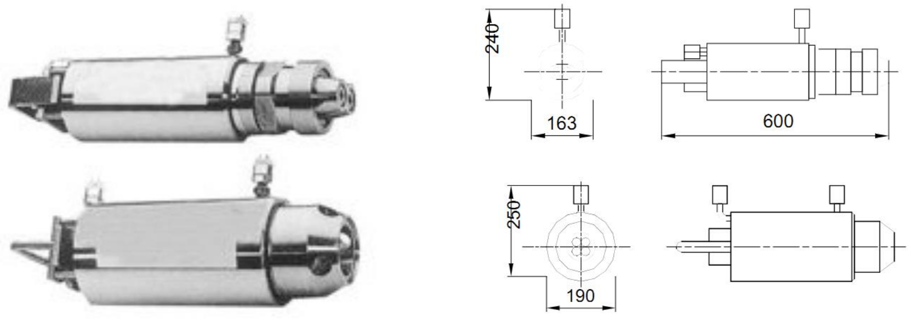


Fig.B.17:- Systems with two and four strands

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POST -TENSIONED FLAT SLAB DESIGN SPREADSHEET

1.DESIGN METHODS.

1.1 Divide the slab in to a series of design strips in the two principal directions of a structure, the width of each design strip is a function of both the center-to -center spans,see figure 1.1.

1.2 using the equivalent frame analysis method determine the fixed-end-moment due to the final prestressing after all losses. The equivalent frame is then analyzed to find the restraint moments. performing separate analysis of the equivalent frame subjected to slab dead loading, additional Divide the slab in to a series of design strips in the two principal directions of a structure, dead loading and live loading.

1.3 the moment for a particular strip are recovered from the analysis and flexural design is carried out based on the ultimate strength design method for prestressing concrete ES-EN :2015. the slab flexural design procedure for each load combination involves the follwing:

1 determine factored moments for each slab strip.
2 design flexural reinforcement for the strip.

1.4 the punching shear is checked at the face of the column and at a critical sections and design the required punching shear reinforcements when the punching shear capacity ratio exceeds unity.

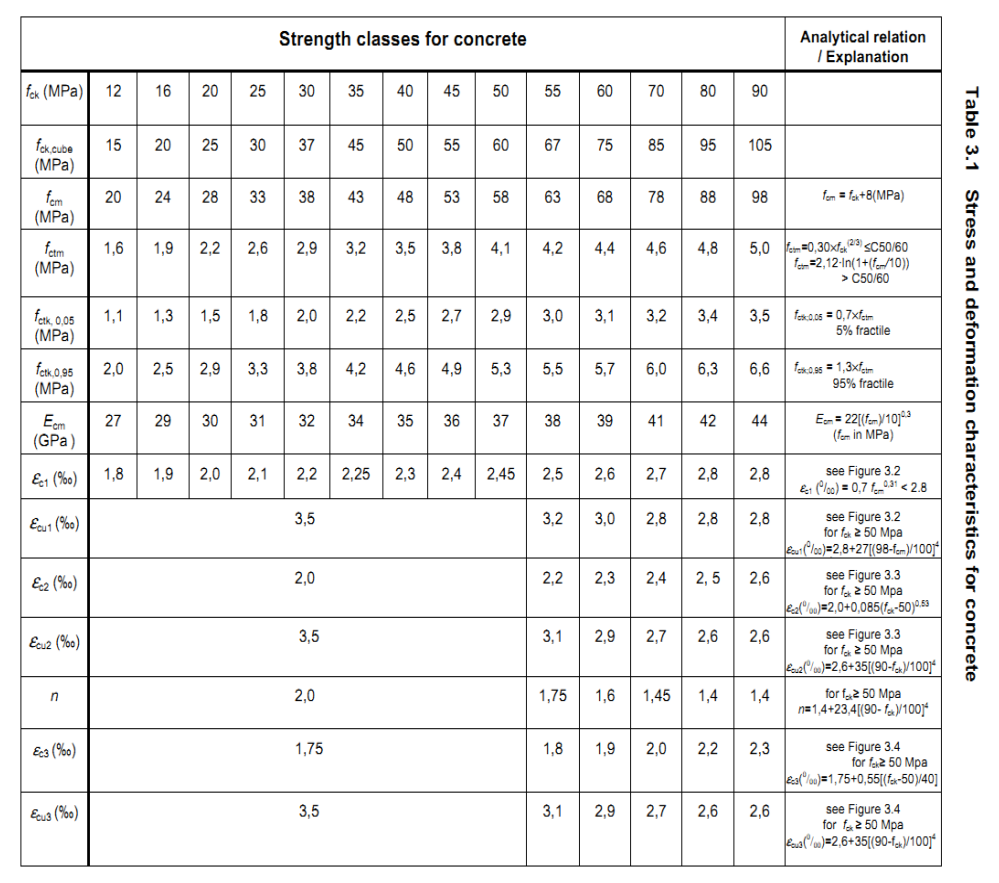
2. INPUT DATA.

2.1 Material properties.

Material.	symbol	Value	unit		unit weight					
Concrete	f_{ck}	35	Mpa			effective age of concrete at 1st loading	$t_{o,eff}$	8.671089795	days	OK
	f_{cm}	43	Mpa	C35/45	25	Cement factor	s	0.2		
	f_{ctm}	3.209962442	Mpa			Temperature adjusted concrete age	$t_{o,T}$	3.765851606	days	
	E_{ccm}	34.0771462	Gpa			Time of anchorage	Δt_i	2	days	
	γ_c	1.5				Final time of consideration	t_f	75	days	
	$f_{cm}(t,eff)$	36.66435395	Mpa	**OK,The anchorage can be releasd after ∆ti days**		Temperature	T	35	^{o}C	
	E_{cm}(t,eff)	31.46668612	Gpa			Humidity(%)	RH	70	%	
	E_{csm}	34.0771462	Gpa			Cement factor	α	1		
Test value	$f_{ctm}(t,eff)$	2.737004631	Mpa			Exposure class	X	XC		
Non-prestressing reinforcement.										
	f_{yk}	414	Mpa							
	E_s	200	Gpa							
	γ_s	1.15								

Tables and formulas

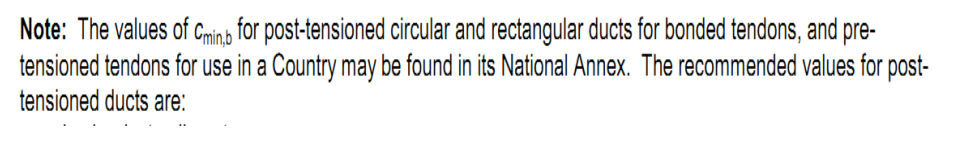
Table 3.1 stress and deformation characterisics for concrete



Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30\times f_{ck}^{(2/3)} \leq$C50/60 $f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0,7\times f_{ctm}$ 5% fractile
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1,3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	32	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 ε_{c1} (‰) = $0,7\ f_{cm}^{0,31} < 2.8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1}(‰)$=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2}(‰)$=2,0+0,085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2}(‰)$=2,6+35[(90-f_{ck})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n=1,4+23,4[(90-f_{ck})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{c3}(‰)$=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3}(‰)$=2,6+35[(90-f_{ck})/100]^4$

Table 3.1 Stress and deformation characteristics for concrete

Minimum concrete cover for bonding



Note: The values of $c_{min,b}$ for post-tensioned circular and rectangular ducts for bonded tendons, and pre-tensioned tendons for use in a Country may be found in its National Annex. The recommended values for post-tensioned ducts are:

Prestressing reinforcement.

			Tondon area	A_{ps}	112 mm2
f_{pk}	1861.58	Mpa	duct diameter	d_{duct}	19 mm
$f_{p0.1k}$	1689.9	Mpa	Allowable tendon stress before transfer	$\sigma_{po,max}$	1489.264 Mpa
E_p	196.5	Gpa	Allowable tendon stress after transfer	$\sigma_{p,max}$	1396.185 Mpa
γ_p	1.15				

2.2 Structrual data

2.2.1 Slab data

				Slab thickness	d	200	mm	
Designer's recommended depth	D	200	mm	Column equivalent stiffness	K_{ec}			
Span dimnsions in EW dir.	l_{1A}	7	m	Column stiffness	K_c			
	l_{2A}	7	m	slab stiffness	K_s			
Span dimnsions in NS dir.	l_{1B}	7	m	Torsional stiffness	K_t			
	l_{2B}	7	m					
Column deimentions	l_c	3	m					
	C_1	400	mm	% of self-weight balanced by prestressing	Φ_p	0.8		
	C_2	400	mm					
For EW direction strip								
For Interior design strip				Fire rating(hr)		2	hours	
width of column strip		3.5	m	Bar diameter for bonding and crack control	Φ	10	mm	
width of design strip		7	m	Cofficient for point of inflection from support	β	0.1		
For exterior design strip				interior span	λ	0.5		
width of column strip		1.75	m	end span	λ	0.45		
width of design strip		5.25	m	point of inflection from support	Lβ	0.7	m	
For NS direction strip				**2.2.2 Estimated slab thickness**				
For Interior design strip				Cofficient for structural system	K	1.2	ES-EN-1992: 2015	
width of column strip		3.5	m	Reference reinforcment ratio(%)	ρ_o	0.59160798		
width of design strip		7	m	The required tension reinforcment(%)	ρ	0.5		
For exterior design strip				The required compression reinforcment(%)	ρ'	0		
width of column strip		1.75	m	Span to depth ratio	$\frac{l}{d}$	27.5815999	Designer's recommended ratio	35
width of design strip		5.25	m	Minimum depth required	d_{min}	0.2	m	
Total length of design strip		30	m	Minimum depth required for given fire rating	$d_{min,fire}$	0	m	

	Dead load	D	2.4	KN/m2	Slab partition load	Dss	1	KN/m2
	live load	L	2.4	KN/m2	Beam partition load	DsB	1	KN/m2
	super Dead load	Ds	1.2	KN/m2				
	effective age of concrete at 1st loading	$t_{o,eff}$	8.671089795	days				
	Final time of consideration	t_f	75	days				
	Temperature	T	35	o_C				
	Factored load(1.35D+1.6L)	w_f	13.32	KN/m2				
For Interior design strip	self-weight(end-span,EW)	w_{end}		KN/m2	For edge design strip	self-weight(end-span,NS)	w_{end}	KN/m2
	self-weight(int-span,EW)	w_{int}		KN/m2		self-weight(int-span,NS)	w_{int}	KN/m2
	self-weight(end-span,NS)	w_{end}		KN/m2		self-weight(end-span,EW)	w_{end}	KN/m2
	self-weight(int-span,NS)	w_{int}	5	KN/m2		self-weight(int-span,EW)	w_{int}	KN/m2

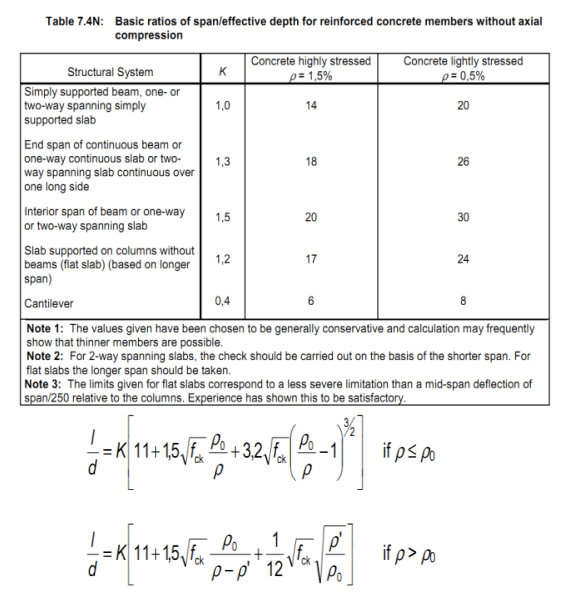


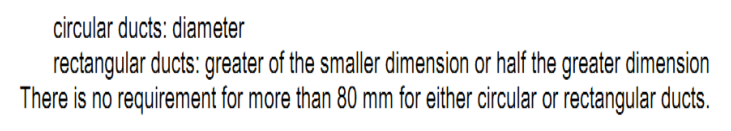
Table 7.4N: Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	K	Concrete highly stressed $\rho = 1,5\%$	Concrete lightly stressed $\rho = 0,5\%$
Simply supported beam, one- or two-way spanning simply supported slab	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory.

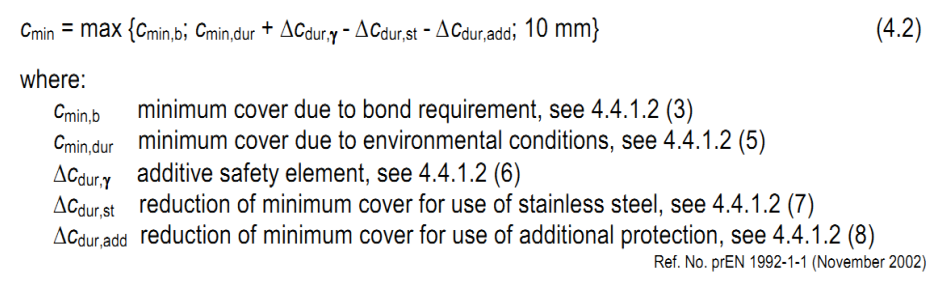
$$\frac{l}{d} = K\left[11 + 1,5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3,2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \quad \text{if } \rho \le \rho_0$$

$$\frac{l}{d} = K\left[11 + 1,5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{if } \rho > \rho_0$$



circular ducts: diameter
rectangular ducts: greater of the smaller dimension or half the greater dimension
There is no requirement for more than 80 mm for either circular or rectangular ducts.

Minimum Cover of concrete



$$c_{min} = \max\{c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10\text{ mm}\} \qquad (4.2)$$

where:
$c_{min,b}$ minimum cover due to bond requirement, see 4.4.1.2 (3)
$c_{min,dur}$ minimum cover due to environmental conditions, see 4.4.1.2 (5)
$\Delta c_{dur,\gamma}$ additive safety element, see 4.4.1.2 (6)
$\Delta c_{dur,st}$ reduction of minimum cover for use of stainless steel, see 4.4.1.2 (7)
$\Delta c_{dur,add}$ reduction of minimum cover for use of additional protection, see 4.4.1.2 (8)
Ref. No. prEN 1992-1-1 (November 2002)

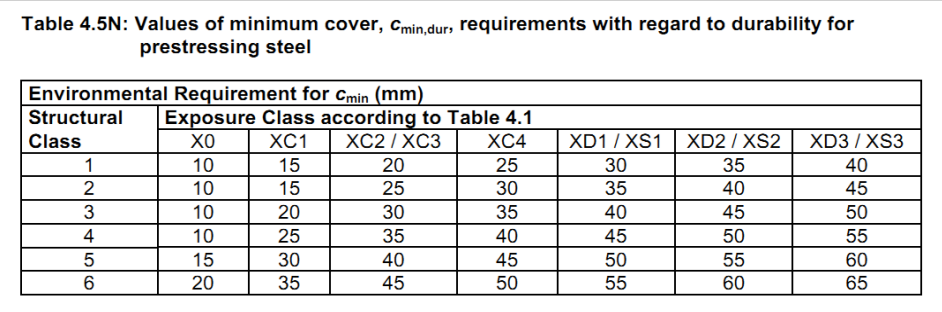


Table 4.5N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for prestressing steel

Environmental Requirement for c_{min} (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
1	10	15	20	25	30	35	40
2	10	15	25	30	35	40	45
3	10	20	30	35	40	45	50
4	10	25	35	40	45	50	55
5	15	30	40	45	50	55	60
6	20	35	45	50	55	60	65

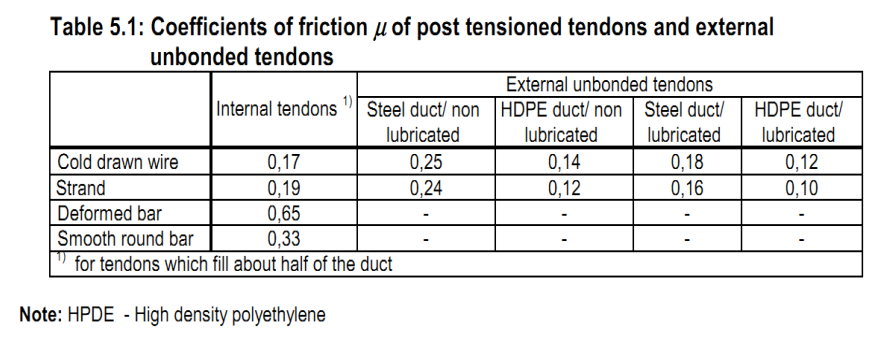


Table 5.1: Coefficients of friction μ of post tensioned tendons and external unbonded tendons

	Internal tendons [1]	External unbonded tendons			
		Steel duct/ non lubricated	HDPE duct/ non lubricated	Steel duct/ lubricated	HDPE duct/ lubricated
Cold drawn wire	0,17	0,25	0,14	0,18	0,12
Strand	0,19	0,24	0,12	0,16	0,10
Deformed bar	0,65	-	-	-	-
Smooth round bar	0,33	-	-	-	-

[1] for tendons which fill about half of the duct

Note: HPDE - High density polyethylene

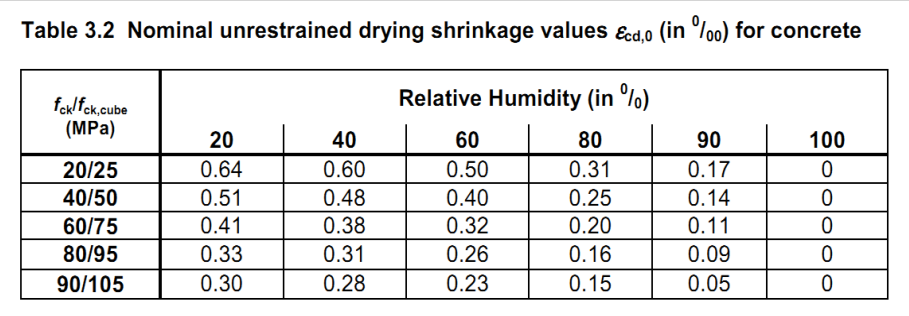


Table 3.2 Nominal unrestrained drying shrinkage values $\varepsilon_{cd,0}$ (in $^0/_{00}$) for concrete

$f_{ck}/f_{ck,cube}$ (MPa)	Relative Humidity (in $^0/_0$)					
	20	40	60	80	90	100
20/25	0.64	0.60	0.50	0.31	0.17	0
40/50	0.51	0.48	0.40	0.25	0.14	0
60/75	0.41	0.38	0.32	0.20	0.11	0
80/95	0.33	0.31	0.26	0.16	0.09	0
90/105	0.30	0.28	0.23	0.15	0.05	0

3.PRELIMINARY CHECK OF THE PUNCHING SHEAR.

3.1 Assume a trial depth.

Cofficient	K			2	≤	2		OK		ES-EN-1992: 2015	
Cofficient	$C_{Rd,c}$			0.12							
Minimum shear resistance	v_{min}			0.689910499	Mpa						
Allowable compressive stress in concret	$\sigma_{c,max}$			21	Mpa						
Minimum bonded tension rebar	ρ			0.05							
Allowable shear resistance	$v_{Rd,c}$			1.444366083	Mpa	≥	v_{min}	OK.		ES-EN-1992: 2015	
Shear stress on critical area due to factored load	v_{ED}			1.34643	Mpa	<	$v_{Rd,c}$	THE DEPTH IS ADEQUATE			
concrete sterss due to prestressing	σ_{cP}			1.0424848	Mpa						

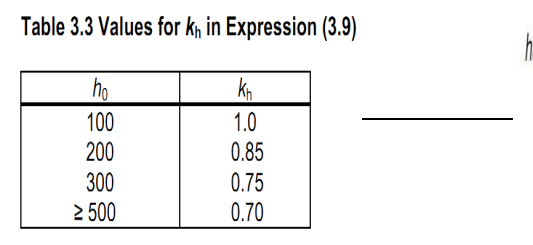
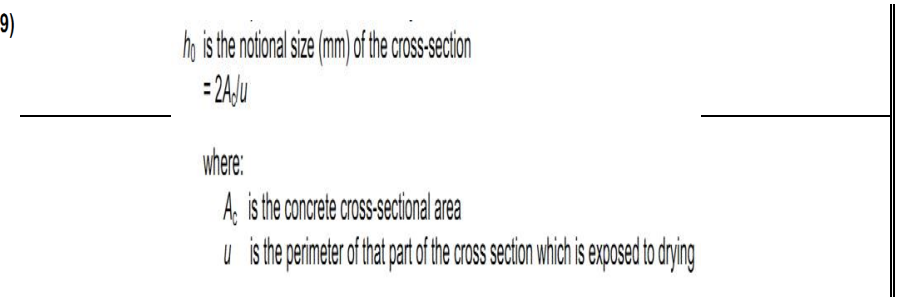


Table 3.3 Values for k_h in Expression (3.9)

h_0	k_h
100	1.0
200	0.85
300	0.75
≥ 500	0.70



h_0 is the notional size (mm) of the cross-section

$= 2A_c/u$

where:

A_c is the concrete cross-sectional area

u is the perimeter of that part of the cross section which is exposed to drying

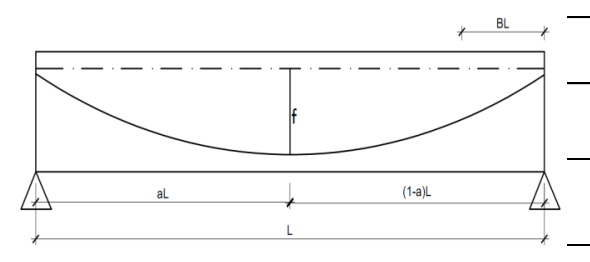
$$v_{Rd} = C_{Rd,c}k(100\,\rho f_{ck})^{1/3} \text{ x } 2d/a \geq v_{min} \text{ x } {}^{2d}\!/_{a}$$

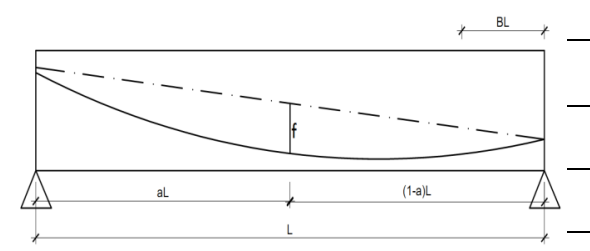
$C_{Rd,c}$ is $0,18/\gamma_c$

4. CHOOSING TENDON PROFILES AND PRESTRESSING FORCES.

4.1 Use maximum eccentricity approach with extera tendons in EW direction where we have diferent tendon forces in end and interior span

NS-Direction





Concrete cover for bonding	$C_{min,b}$			19	mm
Concrete cover for environmental	$C_{min,dur}$			10	mm
Required concrete cover at supoprt	C_{min}			19	mm
depth of tendon centroid from top at support	$d_{pl,sup}$			38.5	mm
depth of tendon centroid from bottom at mid-span	$d_{pl,mid}$			28.5	mm
Centroid of slab from top	Z_t			100	mm
Centroid of slab from bottom	Z_B			100	mm
Eccentricity of lower point of the profile	e_1			71.5	mm
Eccentricity of upper point of the profile	e_2			61.5	mm
Point of inflection near to support for interior-span	$h_{2,int}$			26.6	mm
Point of inflection near to support for end-span	$h_{2,end}$			29.55555556	mm
Tendon drape in mid span	f_{mid}			159.6	mm
Tendon drape in end-span	f_{end}			117.0277778	mm
Tendon force end-span	P_{end}		interior design strip	209.3520057	KN/m
Tendon force interior-span	P_{int}		Interior design strip	250.2528	KN/m
Concrete stress due to Prestress	$\Delta\sigma_c(t)$			1.251264	Mpa

6. TENDON AND CONCRETE STRESS CHECK.

The sum of angular displecment over the distance x	θ	1.325777778	rad							
coefficient of friction	μ	0.2								
an unitentional angular displacement	k	0.005	0.005<k<0.01/m							
The distance along the tendon from active to end	x	7	m							
Prestress force at transfer or anchorage	$P_{mo}(x)$	77.1705242	KN	<	156.37272	Good		ES-EN-1992: 2015 (5.10.3.(2))		
Mean Prestress force after time t	$P_{mt}(x)$	66.96933323	KN							
Allowable maximum tendon force at transfer.	P_{max}	166.797568	KN		Force on concret due to tendon	$P_{c,p}$	160.484456	KN	Good	
Maximum tendon stress capacity at transfer	$\sigma_{p,max}$	1489.264	Mpa		Moment due to prestressing	$M_{c,p}$	9.86979404	KNm		
Allowable Tendon stress during tensioning or anchoring	$\sigma_{pmo}(x)$	1396.185	Mpa		Moment due to quasi-permanent action	$M_{c,q,p}$	571.095	KNm		
Allowable concrete stress during tensioning or anchoring at time t	$\sigma_{ck,all}(t)$	21.99861237								
Concrete stress during tensioning or transfer	$\sigma_c(to)$	7.28151471	Mpa	<	21.99861237	Good		ES-EN-1992:2015 (5.10.2.2.(5))		
Stress varation at the center of tendon of gravity applied at time t	$\Delta\sigma_c(t)$	0.11914112	Mpa							

$$k = 1+\sqrt{\frac{200}{d}} \leq 2,0 \text{ with } d \text{ in mm}$$

$$v_{min} = 0,035\, k^{3/2} \cdot f_{ck}^{1/2}$$

$$f_{cm}(t) = \beta_{cc}(t)\, f_{cm}$$

$$E_{cm}(t) = (f_{cm}(t) / f_{cm})^{0,3}\, E_{cm}$$

with

$$\beta_{cc}(t) = \exp\left\{ s\left[1-\left(\frac{28}{t}\right)^{1/2}\right]\right\}$$

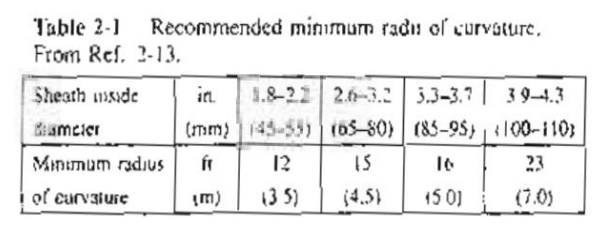


Table 2-1 Recommended minimum radii of curvature. From Ref. 2-13.

Sheath inside diameter	in. (mm)	1.8–2.2 (45–55)	2.6–3.2 (65–80)	3.3–3.7 (85–95)	3.9–4.3 (100–110)
Minimum radius of curvature	ft (m)	12 (3.5)	15 (4.5)	16 (5.0)	23 (7.0)

5.PRESTRESS LOSSES OF DESIGN STRIP NS - DIRECTION.

5.1 Immediate Losses in Prestressing

Friction stress losses	$\Delta\sigma_{p,\mu}$	3.4904625	Mpa/m	ES-EN-1992: 2015 (5.10.5.2(1))
Deformation of concrete losses	$\Delta\sigma_{p,el}$	3.607599277	Mpa	ES-EN-1992: 2015 (5.10.5.1(2))
Loss at anchorage	$\Delta\sigma_{p,set}$	64.15025555	Mpa	Assume anchorage set is=6mm In (European Technical Approval)
The length affected by anchorage set	L_{set}	18.37872647	m	
Total immediate losses		89.6270438	Mpa	

5.2 Long term losses

Creep,shrinkage and relaxation stress	$\Delta\sigma_{p,c+s+r}$	91.08206218	Mpa	
Total Shrinkage strain	ε_{cs}	-0.000181597		ES-EN-1992: 2015 (3.1.4(6))
Drying shrinkage strain	ε_{cd}	-0.00023304		
autogenous shrinkage strain	ε_{ca}	5.14424E-05		
Time dependent coefficient for dry shrinkage	$\beta_{ds}(t,ts)$	0.379488921		
Time dependent coefficient for auyogenous shrinkage	$\beta_{as}(t)$	0.823078794		
Coefficient	$\varepsilon_{ca\infty}$	0.0000625		

Relaxation tendon class(ES-EN-1992: 2015 (3.3.2(4-6))

Class	σ_{1000}(%)	
1	8%	0.08
2	2.50%	0.025
3	4%	0.04

Cement type coefficient(ES-EN-1992: 2015 ANNEX,B.2)

Cement type	α_{ds1}	α_{ds2}
S	3	0.13
N	4	0.12
R	6	0.11

A power depend on type of cement(ES-EN-1992:2015 ANNEX,B.2)

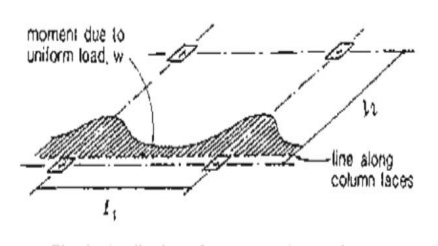
Type	α
S	-1

7. SUMMERY OF TENDON STRESSES.

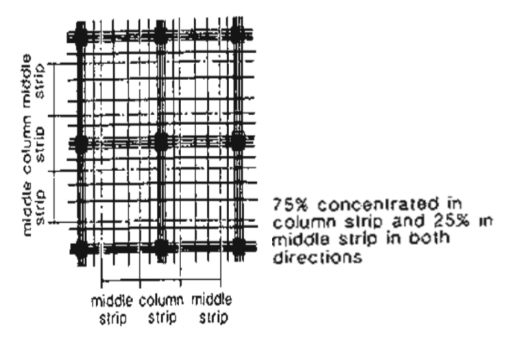
Tendon	Span length(m)	Stress at active end(Mpa)	Stress at the point where anchring set end(Mpa)	Stress at passive end(Mpa)	Average Stress at anchoring(Mpa)	Stress after All losses(Mpa)	Tendon forces After all losses per tendon(KN)		Design tensile stress limit of tendon		ES-EN-1992:2015 (5.10.2.2.(5))
NS Strip end-span	7	1392.577401	1332.034744	1291.471125	1338.694423	1247.612361	139.7325845	<	$P_{max/}\gamma_p$	145.041363	Good
NS Strip interior-span	7	1392.577401	1332.034744	1291.471125	1338.694423	1247.612361	139.7325845	<	$P_{max/}\gamma_p$	145.041363	Good
EW Strip end-span	7							<	$P_{max/}\gamma_p$	145.041363	Good
EW Strip end-span	7							<	$P_{max/}\gamma_p$	145.041363	Good

8. NUMBER AND DISTRIBUTION OF TENDON.

Span	Total force required(KN)	Number of Tendons	Column strip (75%)		Number of strand in Mid strip(25%)	Total Tendon forces(KN)	Stress in concrete cross-section(Mpa)				
			Column(60%) `	out side column(40%)					Concrete tensile stress limit		ES-EN-1992:2015 (3.1.2(9))
NS Strip end-span	1465.46404	11	5	4	2	1537.058429	1.097898878	<	$f_{ctm}(t,eff)$	2.73700463	Good
NS Strip interior-span	1751.7696	13	6	4	3	1816.523598	1.297516856	<	$f_{ctm}(t,eff)$	2.73700463	Good
EW Strip end-span								<	$f_{ctm}(t,eff)$	2.73700463	Good
EW Strip end-span								<	$f_{ctm}(t,eff)$	2.73700463	Good



Elastic distribution of moments along column line.



NOTE:-

The advantage of using tendons concentrated or balanced in the column strip region rather than using uniform distribution includes:-

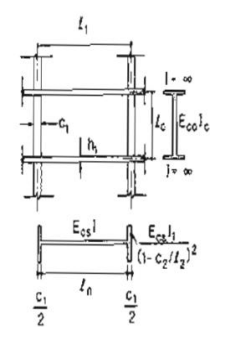
1. A more uniform load-balancing effect is achieved.
2. The distribution more closely resembles the distribution of moment in the slab.
3. Concentration of tendons in the immediate column region increases the punching shear strength of the slab.
4. Result in an increase in moment-transfer strength of the slab-column connection.
5. Banding tendons in one or more directions facilitates placement of tendons.

Coefficient	$\varepsilon_{cd,o}$	-0.000722457		3.2.ES-EN-1992: 2015 (3.1.4(6))	N	0
Coefficient depending on the notional size	k_h	0.85		interpolation from table 3.3.EBCS-EN 2015 (3.1.4(6))	R	1
The notional size	h_o	194.4444444	mm			
Perimeter of cross-section	u	14400	mm			
End of curing time	t_s	8.671089795	days			
Age of concrete at the moment considerd	t	75	days			
Modifying age of loading	$t_{o,T}$		days			
Area of concrete design strip	A_c	1400000	mm2			
Varation of stress due to relaxation	$\Delta\sigma_{pr}$	56.36707233	Mpa			
Absolute value of the initial prestress	σ_{pi}	1396.185	Mpa			
The relaxation loss(%) at 1000 hours after tensioning	σ_{1000}	0.08	hours			
coefficient	μ	0.75				
coefficient	β_{RH}	-1.55				
coefficient depending on type of cement	α_{ds1}	6				
coefficient depending on type of cement	α_{ds2}	0.11				
constant	f_{cmo}	10	Mpa			
Creep coefficient	φ(t,to)	1.947288496				
Notional creep coefficient	φ_o	3.738879969				
Factor allow for humidity effect	φ_{RH}	2.39385919				
Factor allow for the effect of concrete strength	$\beta(f_{cm})$	2.561975982				
coefficient to describe the development of creep after loading	$\beta_c(t,to)$	0.520821346				

9. ANALYSIS BY EQUIVALENT FRAME METHOD.

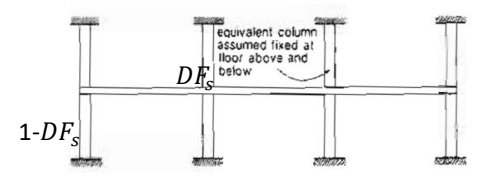
9.1 Equivalent frame properties

Column deimentions	l_c	3000	mm
	C_1	400	mm
	C_2	400	mm
Column equivalent stiffness	K_{ec}	49785242.33	MN-m
Column stiffness	K_c	223685882.7	MN-m
slab stiffness	K_s	93545107.21	MN-m
Torsional stiffness	K_t	64038038.33	MN-m
Constant	C	730666666.7	mm4
Column Moment of inertia	I_c	2133333333	mm4
Slab moment of inertia	I_s	4666666667	mm4
Distribution Factor	DF	0.652653869	
Caryover factor	COF	0.5	



$$K_t = \frac{\sum 9E_{cs}C}{l_2(1-c_2/l_2)^3}$$

$$C = \frac{(1-0.63x/y)x^3y}{3}$$



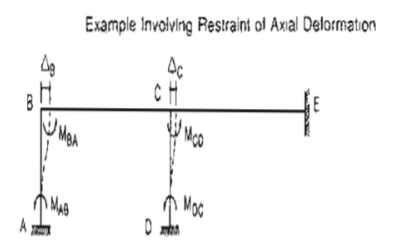
	Factor allow for the effect of concrete age	$\beta(t_o)$		0.609632158				
	coefficients influence of concrete strength	α_1		0.865804248				
		α_2		0.959665579				
		α_3		0.902193709				
		t_i		2	days			
		$t_{o,T}$		3.765851606	days			
		t_o		8.671089795	days	Good		
	Cofficient depend on the relative humidity	β_H		517.2150939				
	A power depend on cement type	α		-1				
	The second moment of the area of the concrete section	I_c		4666666667	mm4			
	The stress of concrete due to self-weight, initial prestress and quasi-permanet actions.	$\sigma_{c,Qp}$		7.28151471	Mpa			
	The distance b/n the center of gravity of the concrete section and the tendon	Z_{cp}		61.5	mm			
	Total long-term losses	$\Delta P_{c,s+c+r}$		10.20119096	KN			

9.2 Fixed end moments due to prestressing.

Fixed end moment at end support	M_A	119.9190215	KN-m
Fixed end moment at mid-suport to the right	M_B	40.88575421	KN-m
Fixed end moment at mid-suport to the left	M_c	40.88575421	KN-m
Moment arm for end-span	Z_e	78.01851852	mm
Moment arm for mid-span	Z_m	26.6	mm

9.3 Slab restraint moments due to prestressing.

Fixed end moment due to restraint at end support	$M_{R,A}$	0.023074852	KN-m
Fixed end moment due to restraint at mid support	$M_{R,B}$	0.024543252	KN-m
Deformation at end support	Δ_A	0.000476111	mm
Deformation at mid- support	Δ_B	0.000506409	mm

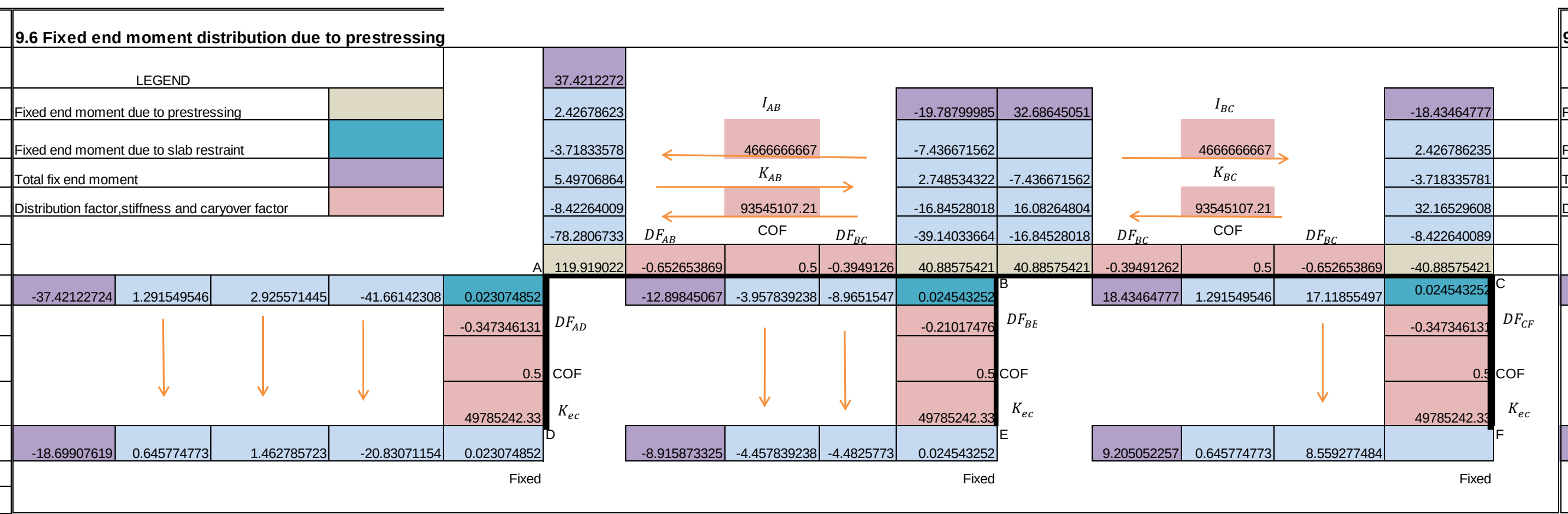


9.4 Moments due to Service loads.

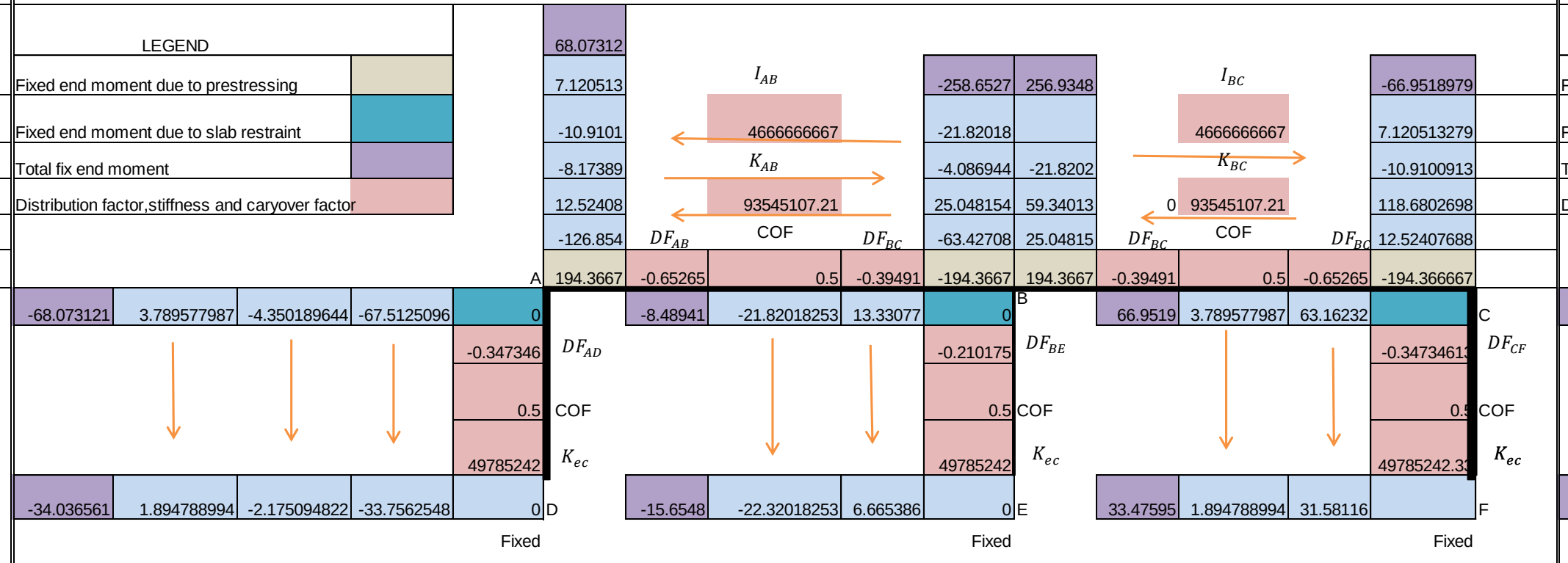
moment at top face column end span		$M_{c,end}$	-7.16	KN-m
moment at bottom face of end span		$M_{B,end}$	63.41	KN-m
moment at bottom face of interior span		$M_{B,int}$	49.38	KN-m
moment at top face column interior span		$M_{c,int}$	-138.59	KN-m

9.4.1 Moments due to Service loads from software result

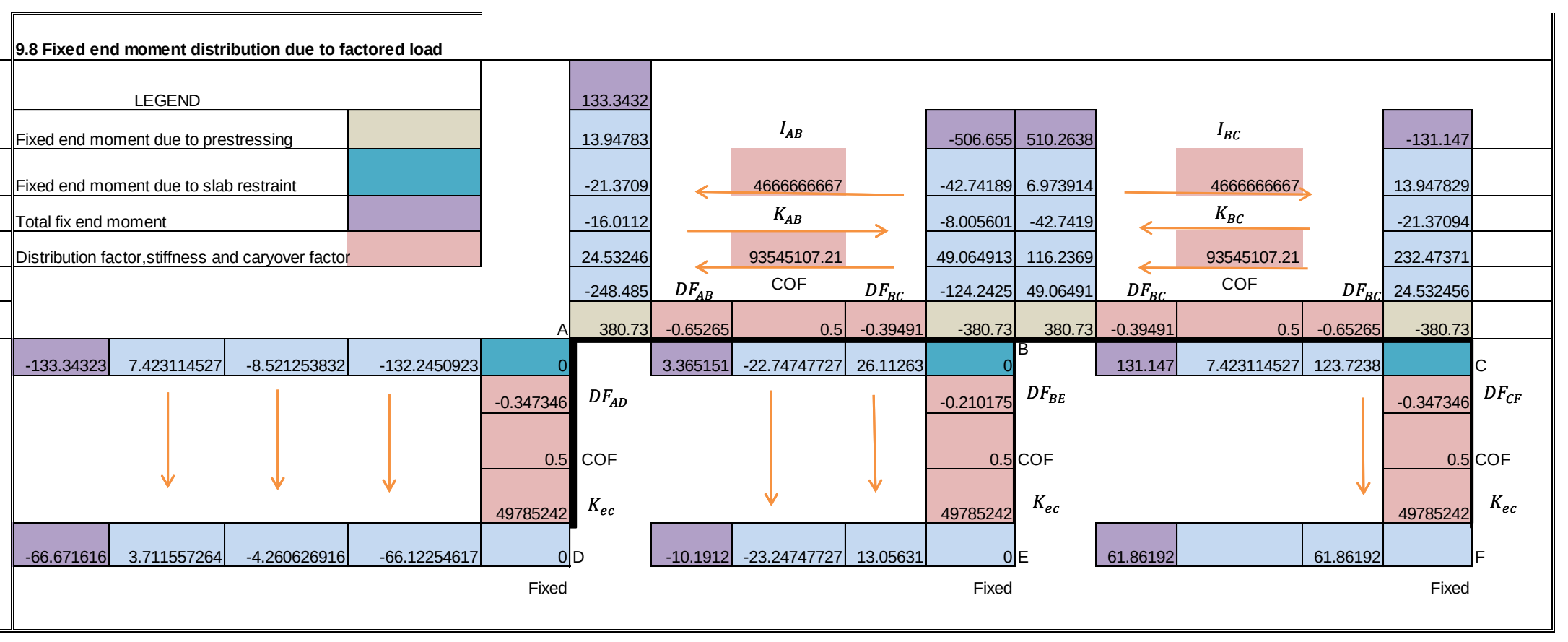
9.6 Fixed end moment distribution due to prestressing



9.7 Fixed end moment distribution due to service load



9.8 Fixed end moment distribution due to factored load



10. CHECK CONCRETE TENSILE STRESS UNDER SERVICE LOADS.

Design strip	Location	Stress calculations				Section Calculation			cement coefficient	Coefficenet depend on concrete age	Coefficient	Tensile strength		ES-EN-1992: 2015 (3.1.2(9))	
		Stress due to Prestressing(Mpa)	Stress due to Prestress moment(Mpa)	Stress due to service load(Mpa)	Total stress(Mpa)	e	Z	I_s	S	$\beta_{cc}(t)$	α	$f_{ctm}(t)$	Remark	Cement type	s-value
NS- Direction	Bot-face end-span	-1.097898878	-1.946156463	0.080286981	-2.96376836	59.08730159	46666.66667	4666666.67	0.2	1.080904274	1	2.737004631	Good	R	0.2
	Top-face at column	-1.097898878	-1.946156463	-0.00906568	-3.05312102	59.08730159	46666.66667	4666666.67	0.2	1.080904274	1	2.737004631	Good	N	0.25
	Bot-face interior-span	-1.297516856	-2.6986497	0.073359533	-3.92280702	69.32857143	46666.66667	4666666.67	0.2	1.080904274	1	2.737004631	Good	S	0.38
EW- Direction	Bot-face end-span												Not Ok, aditional bond reinforcement required	$\alpha = 1$ for $t < 28$ $\alpha = 2/3$ for $t \geq 28.$	
	Top-face at column												Not Ok, aditional bond reinforcement required	$f_{ctm}(t) = (\beta_{cc}(t))^{\alpha} \cdot f_{ctm}$	
	Bot-face interior-span												Not Ok, aditional bond reinforcement required		

moment at top face column end span	M_c	-7.16	KN-m
moment at bottom face of end span	$M_{B,end}$	63.41	KN-m
moment at bottom face of interior span	$M_{B,int}$	49.38	KN-m
moment at top face column interior span	$M_{c,int}$	-138.59	KN-m

9.5 Moments due to Factored loads.

moment at top face column of end span	$M_{c,end}$	-10.13	KN-m
moment at bottom face of end span	$M_{B,end}$	86.58	KN-m
moment at bottom face of interior span	$M_{B,int}$	68.94	KN-m
moment at top face column interior span	$M_{c,int}$	-196	KN-m

9.5.1 Moments due to Factored loads from software result

moment at top face column end span	M_c	-10.13	KN-m
moment at bottom face of end span	$M_{B,end}$	86.58	KN-m
moment at bottom face of interior span	$M_{B,int}$	68.94	KN-m
moment at top face column interior span	$M_{c,int}$	-196	KN-m

11. MINIMUM REINFORCEMENT TO CONTROL CRACK.

Cofficient accoun for the nature of the stress distribution	Bot-face end-span	K_c	0.646214167		OK	ES-EN-1992: 2015 (7.3.2(2))
	Top-face at column	K_c	0.653637113		OK	$A_{s,min}\sigma_s = k_c\, k\, f_{ct,eff}\, A_{ct}$
	Bot-face interior-span	K_c	0.725886016		OK	$k_c = 0,4 \cdot \left[1 - \frac{\sigma_c}{k_1(h/h^*)f_{ct,eff}}\right] \leq 1$
Cofficient accoun for the effects of axial forces	Bot-face end-span	K_1	1.5			
	Top-face at column	K_1	1.5			$k_1 = 1,5$ if N_{Ed} is a compressive force
	Bot-face interior-span	K_1	1.5			$k_1 = \frac{2h^*}{3h}$ if N_{Ed} is a tensile force
Mean stress of concrete (EW-direction)	Bot-face end-span	σ_c	-2.96376836	Mpa		NOTE:-Provide bottom rebar perpendicular to all free-edge along width of design strips,since there are edge zones that are not prestressed
	Top-face at column	σ_c	-3.053121021	Mpa		
	Bot-face interior-span	σ_c	-3.922807023	Mpa		$\rho_s = 0.0015\text{-}\rho_p > 0.0005$
Minimum reinforcement area	Bot-face end-span	$A_{s,min}$	854.4401777	mm2/m		$A_s = \rho_s{*}A_c$
	Top-face at column	$A_{s,min}$	1037.105973	mm2/m		See ES-EN-1992:2015 section 9.4
	Bot-face interior-span	$A_{s,min}$	959.7842454	mm2/m		

12. CHECK FLEXURAL STRENGTH.

Design strip	Location	Tendon area total in the section	Min.rebar area	Tendon depth	Depth of rebar	Individual tendon stress	Stress block depth	Value of ϕ	Design flexural strength	Factored moment	Remark			
		A_{ps}	$A_{s,min}$	d_p	d_s	f_{ps}	a	ϕ	ϕM_n(KNm	M_u(KNm)				
NS- Direction	End-span column-strip	924	854.4401777	162	181	1247.612361	6.905853711	0.85	208.7406832	-10.13	OK			
	End-span middle-strip	308	854.4401777	152	181	1247.612361	3.382961311	0.85	103.0085446	86.58	OK			
	Interior-span column-strip	1092	959.7842454	117.1	181	1247.612361	8.066559408	0.85	190.7052523	-196	OK			
	Interior-span middle-strip	364	959.7842454	120.0555556	181	1247.612361	3.903141117	0.85	106.0627311	68.94	OK			

13. CHECK SHEAR AND MOMENT TRANSFER STRENGTH.

Column location	Critical perimeter	Critical area	Tributary area	Critical section properties						Unbalanced moment EW-direction	Unbalanced moment NE-direction	Factored shear force in control perimeter	Support load eccentricity factor	Factored shear stress	Maximum factored shear stress	Punching shear stress capacity with out shear rebar	Punching shear stress capacity with shear rebar	Perimeter	Remark
NS- Direction	U_1(mm)	A_c(mm2)	A_t(mm2)	e_1(mm)	e_2(mm)	J_1(mm4)	J_2(mm4)	γ_{v1}	γ_{v2}	M_{u1}(KNm)	M_{u2}(KNm)	F_{vf}(KN)	β	V_f(Mpa)	$V_{max,f}$(Mpa)	$V_{Rd,c}$(Mpa)	$V_{Rd,cs}$(Mpa)	$U_{out,ef}$(mm	
Interior column	2400	480000	49000000	300	300	29600000000	29600000000	0.4	0.4			646.2864	1.15	1.34643	1.5483945	0.799810487	0.599857865	4646.284163	Not Ok, Provide shear rebar
Edge column	2000	320000	22050000	156.25	300	40770833333	4090000000	0.378333925	0.42206445			289.4436	1.4	0.723609	1.0130526	0.779848689	0.584886517	2598.074765	Not Ok,Provide shear rebar
Corner column	1600	200000	11025000	125	125	18041666667	18041666667	0.4	0.4			144.189	1.5	0.450590625	0.675885938	0.779848689	0.584886517	1386.701696	Not Ok,provide shear rebar

ES-EN-1992: 2015 (6.4.4 & 6.4.5)

$$v_{Rd,c} = C_{Rd,c}\, k\, (100 \rho_l f_{ck})^{1/3} + 0{,}10\sigma_{cp} \geq (v_{min} + 0{,}10\sigma_{cp})$$

where:

f_{ck} is in MPa

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 \quad d \text{ in mm}$$

$$\rho_l = \sqrt{\rho_{ly} \cdot \rho_{lz}} \leq 0{,}02$$

$$v_{Ed} = \beta \frac{V_{Ed}}{u_i d}$$

$$v_{Ed} = \frac{\beta V_{Ed}}{u_0 d} \leq v_{Rd,max} = 0{,}5 \nu f_{cd}$$

where

u_0 for an interior column u_0 = length of column periphery

for an edge column $u_0 = c_2 + 3d \leq c_2 + 2c_1$

for a corner column $u_0 = 3d \leq c_1 + c_2$

c_1, c_2 are the column dimensions as shown in Figure 6.20

ν see Expression (6.6)

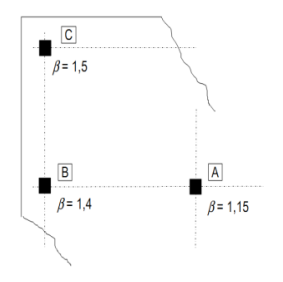
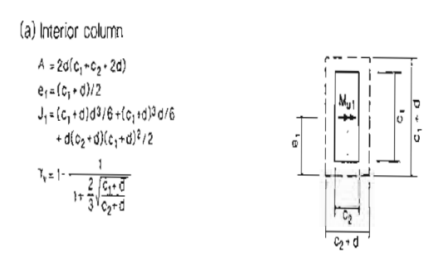
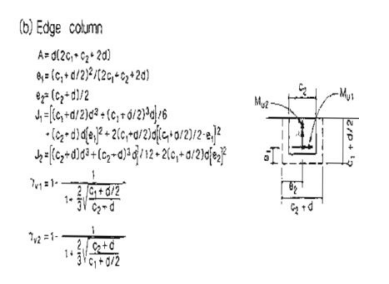
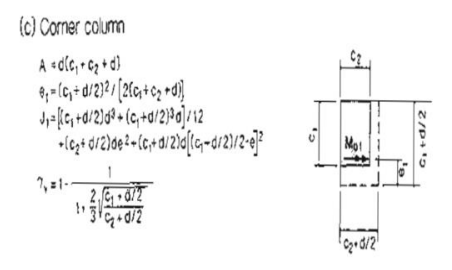


Figure 6.21N: Recommended values for β







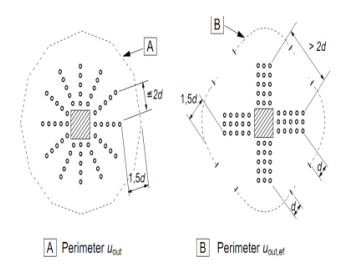


Figure 6.22: Control perimeters at internal columns

14. CHECK FOR DEFLECTION

14.1 INSTANTANEOUS LIVE LOAD DEFLECTION

See ES-EN-1992:2015 section 7.4

	Live load deflection	Δ_{LL}	5.020116679	mm	Code limit of deflection	L/250	28	mm	SATISFACTORY

14.2 CREEP DEFLECTION

See ES-EN-1992:2015 section 7.4

Total dead load(KN/m2)	Load balanced by prestressing(KN/m2)	% of live load sustained	% of live load sustained(KN/m2)	Total load cause deflection(KN/m2)	Deflection due to total load(mm)	Creep coefficent	Creep deflection expected(mm)	Deflection due to non-sustained live load(mm)	Sum of creep and non-sustained LL deflection(mm)	
						φ(t,to)	Δ_{creep}	$\Delta_{Non,s,LL}$	$\Delta_{creep+non,s,LL}$	
7.2	4	0.3	0.72	3.92	5.718089042	1.947288496	11.13476901	3.514081675	14.64885069	SATISFACTORY

POST -TENSIONED FLAT SLAB DESIGN SPREADSHEET

1.DESIGN METHODS.

1.1 Divide the slab in to a series of design strips in the two principal directions of a structure, the width of each design strip is a function of both the center-to -center spans,see figure 1.1.

1.2 using the equivalent frame analysis method determine the fixed-end-moment due to the final prestressing after all losses. The equivalent frame is then analyzed to find the restraint moments. performing separate analysis of the equivalent frame subjected to slab dead loading, additional Divide the slab in to a series of design strips in the two principal directions of a structure, dead loading and live loading.

1.3 the moment for a particular strip are recovered from the analysis and flexural design is carried out based on the ultimate strength design method for prestressing concrete ES-EN :2015. the slab flexural design procedure for each load combination involves the follwing:

1 determine factored moments for each slab strip.
2 design flexural reinforcement for the strip.

1.4 the punching shear is checked at the face of the column and at a critical sections and design the required punching shear reinforcements when the punching shear capacity ratio exceeds unity.

2.INPUT DATA

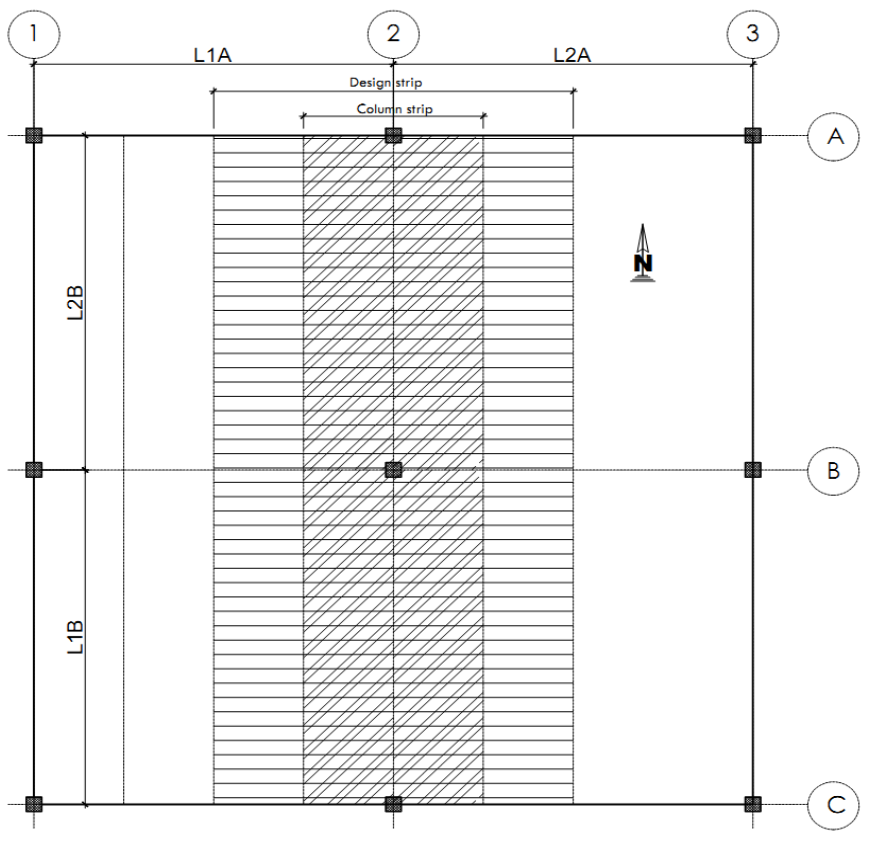
2.1 Material properties.

Material.	symbol	Value	unit	Description		symbol	Value	unit	Description
Concrete	f_{ck}	35	Mpa		effective age of concrete at 1st loading	$t_{o,eff}$	8.67109	days	**OK**
	f_{cm}	43	Mpa		Cement factor	s	0.2		
	f_{ctm}	3.209962442	Mpa		Temperature adjusted concrete age	$t_{o,T}$	3.76585	days	
	E_{ccm}	34.0771462	Gpa		Time of anchorage	Δt_i	2	days	
	unit weight	25							
	γ_c	1.5			Final time of consideration	t_f	75	days	
	$f_{cm}(t,eff)$	36.66435395	Mpa	**OK,The anchorage can be releasd after Δti days**	Temperature	T	35	o_C	
	E_{cm}(t,eff)	31.46668612	Gpa		Humidity(%)	RH	70	%	
	E_{csm}	`	Gpa		Cement factor	α	1		
	$f_{ctm}(t,eff)$	2.737004631	Mpa	Test value	Exposure class	χ	XC		
Non-prestressing reinforcement.									
	symbol	Value	unit	Description					
	f_{yk}	414	Mpa						
	E_s	200	Gpa						
	γ_s	1.15							
Prestressing reinforcement.									
	symbol	Value	unit	Description		symbol	Value	unit	
					Tondon area	A_{ps}	112	mm2	
	f_{pk}	1861.58	Mpa		duct diameter	d_{duct}	19	mm	
	$f_{p0.1k}$	1689.9	Mpa		Allowable tendon stress before transfer	$\sigma_{po,max}$	1489.26	Mpa	
	E_p	196.5	Gpa		Allowable tendon stress after transfer	$\sigma_{p,max}$	1396.19	Mpa	
	γ_p	1.15							

2.2 Structural properties.

2.2.1 Slab and Tendon data

	symbol	Value	unit			symbol	Value	unit	Description
Designer's recommended depth	D	200	mm		% of self-weight balanced by prestressing	Φ_p	0.8		
Span dimnsions in EW dir.	l_{1A}	7	m		Fire rating(hr)		2	hours	
	l_{2A}	7	m		Bar diameter for bonding and crack control	Φ	10	mm	
Span dimnsions in NS dir.	l_{1B}	7	m		Cofficient for point of inflection from support	β	0.1		
	$l_{2`B}$	7	m		interior span	λ	0.5		
Column deimentions	l_c	3	m		end span	λ	0.45		
	C_1	400	mm		point of inflection from support	Lβ	0	m	
	C_2	400	mm		Concrete cover for environmental	$C_{min,dur}$	10	mm	
Total length of design strip	L	30	m						



2.2.2 Estimated slab thickness

	symbol	Value	unit	Description		symbol	Value	unit	Description
Cofficient for structural system	K	1.2		ES-EN-1992: 2015	Allowable shear resistance		1.44437	Mpa	
Reference reinforcment ratio(%)	ρ_o	0.591607978			Shear stress on critical area due to factored load		1.34643	Mpa	
The required tension reinforcment(%)	ρ	0.5			Minimum shear resistance v_{min}		0.68991		**OK.**
The required compression reinforcment(%)	ρ′	0			v_{ED}	<	$v_{Rd,c}$		**THE DEPTH IS ADEQUATE**
Span to depth ratio	$\frac{l}{d}$	27.58159994	**Designer's recommended ratio**	35	The sum of angular displecment over the distance x	θ	1.32578	rad	
Minimum depth required	d_{min}	0.2	m		coefficient of friction	μ	0.2		
Minimum depth required for given fire rating	$d_{min,fire}$		m		an unitentional angular displacement	k	0.005	0.005<k<0.01/m	

Minimum bonded tension rebar	ρ	0.05			The distance along the tendon from active to end	x	7	m	

2.2.3 Tendon profil data

	symbol	Value	unit	Description		symbol	Value	unit	Description
Concrete cover for bonding	$C_{min,b}$	19	mm		Eccentricity of upper point of the profile	e_2	61.5	mm	
Concrete cover for environmental	$C_{min,dur}$	10	mm		Point of inflection near to support for interior-span	$h_{2,int}$	26.6	mm	
Required concrete cover at supoprt	C_{min}	19	mm		near to support for end-span	$h_{2,end}$	29.5556	mm	
depth of tendon centroid from top at support	$d_{pl,sup}$	38.5	mm		Tendon drape in mid span	f_{mid}	159.6	mm	
depth of tendon centroid from bottom at mid-span	$d_{pl,mid}$	28.5	mm		Tendon drape in end-span	f_{end}	117.028	mm	
Centroid of slab from top	Z_t	100	mm		Tendon force end-span	P_{end}	209.352	KN/m	
Centroid of slab from bottom	Z_B	100	mm		Tendon force interior-span	P_{int}	250.253	KN/m	
Eccentricity of lower point of the profile	e_1	71.5	mm		Concrete stress due to Prestress	$\Delta\sigma_c(t)$	1.25126	Mpa	

3. NUMBER AND DISTRIBUTION OF TENDON.

Span	Total force required(KN)	Number of Tendons	Column strip (75%)		Number of strand in Mid strip(25%)
			Column(60%)	out side column(40%)	
NS Strip end-span	1465.46404	11	5	4	2
NS Strip interior-span	1751.7696	13	6	4	3

4. MINIMUM REINFORCEMENT TO CONTROL CRACK.

Minimum reinforcement area		Bot-face end-span	$A_{s,min}$	854.4401777	mm2/m		See ES-EN-1992:2015 section 9.4
		Top-face at column	$A_{s,min}$	1037.105973	mm2/m		
		Bot-face interior-span	$A_{s,min}$	959.7842454	mm2/m		

5.TENDON STRESSES CHCKE.

			Tendon forces After all losses per tendon(KN)		Design tensile stress limit of tendon P_{max}/γ_p		
			139.7325845	<	145.0413635	**Good**	ES-EN-1992:2015 (5.10.2.2.(5))

6.CONCRETE STRESS CHECK.

	stress during tensioning or transfer	$\sigma_c(to)$	7.28151471	<	21.99861237	**Good**	ES-EN-1992:2015 (5.10.2.2.(5))

7. CHECK CONCRETE TENSILE STRESS UNDER SERVICE LOADS.

			Total stress(Mpa)		Tensile strength	
			-2.96376836	<	2.737004631	**Good**

8.CHECK FLEXURAL STRENGTH.

			Design flexural strength		Factored moment	
			208.7406832	<	-10.13	**OK**

9.CHECK SHEAR AND MOMENT TRANSFER STRENGTH.

			Maximum factored shear stress		Punching shear stress capacity with shear rebar		ES-EN-1992: 2015 (6.4.4 & 6.4.5)
			1.5483945	>	0.599857865	**Not Ok, Provide shear rebar**	

10.CHECK FOR DEFLECTION

			Live load deflection		Code limit of deflection		See ES-EN-1992:2015 section 7.4
			5.020116679		28	**SATISFACTORY**	
			Sum of creep and non-sustained LL deflection(mm)		Code limit of deflection		See ES-EN-1992:2015 section 7.4
			14.64885069		28	**SATISFACTORY**	