
PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

BY: EDOM ADANE

ADVISOR: DR. ASNAKE ADAMU



**A Thesis submitted to School of Civil and Environmental Engineering,
Addis Ababa University in partial fulfillment of the requirement for the
Degree of Master of Science in Civil Engineering**

DECEMBER, 2015



ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIROMENTAL ENGINEERING

**PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE
HOUSING PROJECTS**

BY
EDOM ADANE

APPROVED BY BOARD OF EXAMINERS:

DR. ASNAKE ADAMU

ADVISOR

DR. GIRMA ZERAYOHANNES

INTERNAL EXAMINER

DR. ABRHAM GEBRE

INTERNAL EXAMINER

ATO NEBYU SIRAJ

CHAIRMAN

ACKNOWLEDGMENT

To my father in heaven, I live by your grace and your mercies endure forever. You are author of my life, my salvation and I give you all the praise and glory. Your word brought me to into existence. I surrender to You; You are the way, the truth and the life.

To Dr. Abeje Gedele, the fact of introducing prestressed unit for building project was your idea and I appreciate the support, enlightenment and encouragement. For my Thesis Advisor Dr. Asnake Adamu, thank you for your priceless comments, encouragement and tolerance. For helping me from the beginning, till the end of the project. I would like to thank Dr. Esayas G.youhannes for being a great advisor, inspiration and guidance.

I would also like to confirm my deep gratitude for Dr. Girma Zerayohannes, Ato Melaku, Ato Asmerom, Ato Yishak Adane and Ato Birhanemeskel for their kind treatment, valuable comments, assistance, enthusiasm and patience. Blen Tesfaluel, I will never forget the great assistance towards the completion of this project. To Natnael Tadele, thank you for being a good cheer and wonderful assistance.

Dear Families and Friends, though I was hard to be with due to tight schedule, thank you for believing in me and helping me all the way. Your unconditional love, guidance and moral support is all what kept me going forward.

I wish to express our sincere thanks to the Governmental and non-governmental organizations have generously pledge their time, money and knowledge for the successful completion of this research work. Special Thanks in favor of CREC and ERC, for both the site visit and educational support. I am grateful for what you did for me.

Last but not least I would like to thank the department of civil and environmental engineering for their support in the provision of materials and letters whenever needed.

ABSTRACT

This research is a first step towards advancement of construction industry, specially the housing project. The glimpse of the effect of introducing pre-cast prestressed system for housing project is dealt here. Further researches on how to apply the system and detailing regarding the column to beam should be studied specifically. In addition to detailing the pre-cast prestressed industry installation cost and operation cost should be assessed and detail cost should be considered.

The research mainly focus on the design of 40/60 housing project using a pre-cast prestressed concrete and noting the significance of adopting such a system in quality, time and cost. To compare the outcome of adopting such a system, the design values of the prestressed concrete system is compared with the design of conventional reinforced concrete.

The research finally concludes by discussing the effect of the newly developed system and by noting areas of further research that are needed to confidently apply pre-cast prestressed system for the 40/60 housing project.

TABLE OF CONTENT

ACKNOWLEDGMENT	I
ABSTRACT.....	II
LIST OF FIGURES	VI
LIST OF TABLES	VII
LIST OF ANNEXES.....	VIII
LIST OF ABBREVIATIONS	IX
CHAPTER 1	1
INTRODUCTION.....	1
1.1 BACKGROUND TO THE HOUSING PROJECT PROBLEM.....	1
1.2 OBJECTIVES	2
1.2.1 GENERAL OBJECTIVE.....	2
1.2.2 SPECIFIC OBJECTIVES	2
1.3 APPLICATION OF RESULTS	3
1.4 LIMITATION / SCOPE OF THE RESEARCH.....	3
1.5 STRUCTURE OF THE RESEARCH.....	4
CHAPTER 2	5
PRESTRESSED CONCRETE	5
2.1 INTRODUCTION.....	5
2.1.1 DEVELOPMENT OF PRESTRESSED CONCRETE	6
2.1.2 ADVANTAGES AND LIMITATIONS OF PRESTRESSED CONCRETE	7
2.2 TYPES OF PRESTRESSING.....	10
2.2.1 SOURCE OF PRESTRESSING FORCE	11
2.2.2 LOCATION OF THE PRESTRESSING TENDON WITH RESPECT TO THE CONCRETE SECTION	11
2.2.3 SEQUENCE OF CASTING THE CONCRETE AND APPLYING TENSION TO THE TENDONS	12
2.2.4 SHAPE OF THE PRESTRESSED MEMBER	14
2.2.5 AMOUNT OF PRESTRESSING FORCE.....	15

2.2.6	BASED ON DIRECTIONS OF THE PRESTRESSING MEMBER.....	15
2.2.7	CONCRETE CASTING MECHANISM.....	15
2.3	PRESTRESSING SYSTEMS AND DEVICES.....	16
2.3.1	PRE-TENSIONING SYSTEMS AND DEVICES.....	16
2.3.2	POST-TENSIONING SYSTEMS AND DEVICES.....	17
2.4	MATERIALS FOR PRESTRESSED CONCRETE.....	18
2.4.1	CONCRETE.....	18
2.4.2	PRESTRESSING STEEL.....	20
2.5	ANALYSIS AND DESIGN OF STRUCTURES USING PRESTRESSED CONCRETE.....	21
2.5.1	ANALYSIS FOR FLEXURE.....	21
2.5.2	DESIGN OF FLEXURE REINFORCEMENTS.....	27
2.5.3	ANALYSIS AND DESIGN FOR SHEAR.....	32
2.5.4	DESIGN OF SHEAR REINFORCEMENTS.....	34
2.5.5	PRESTRESSED CONCRETE MEMBERS IN TORSION.....	35
2.5.6	DESIGN OF REINFORCEMENTS FOR TORSION SHEAR AND BENDING.....	36
2.5.7	ANALYSIS AND DESIGN REQUIREMENTS FOR DEFLECTION.....	38
2.6	LOSSES IN PRESTRESSING TENDONS.....	39
2.6.1	IMMEDIATE LOSSES.....	39
2.6.2	DIFFERED LOSSES.....	41
2.6.3	TOTAL LOSSES.....	43
CHAPTER 3	44
ANALYSIS AND DESIGN OF THE HOUSING PROJECTS USING CONVENTIONAL REINFORCED CONCRETE.....		44
3.1	INTRODUCTION.....	44
3.2	MATERIAL PROPERTY AND LOAD COMBINATIONS.....	44
3.3	MODELLING AND DESIGN ASPECTS.....	47
CHAPTER 4	49
ANALYSIS AND DESIGN OF THE HOUSING PROJECTS USING PRECAST PRESTRESSED CONCRETE UNITS.....		49
4.1	DIMENSIONING AND SETTING UP STRUCTURAL ARRANGEMENTS.....	49
4.2	SETTING MATERIAL PROPERTY AND LOADING.....	49
4.3	ASSUMPTIONS.....	49
4.4	ANALYSIS AND DESIGN FOR THE SLAB PANELS.....	50

4.5 ANALYSIS AND DESIGN OF THE GIRDER BEAM	64
4.5.1 ARRANGEMENTS AND INITIAL DESIGN FOR THE GIRDER BEAM	64
4.5.2 ANALYSIS OUTPUTS OF GIRDER BEAM USING SOFTWARE.....	66
4.5.3 DESIGN OF THE PRE-CAST PRESTRESSED GIRDER BEAM	68
4.6 CONNECTIONS AND DETAILINGS	70
CHAPTER 5	72
ECONOMIC ANALYSIS	72
5.1 BACKGROUND OF REINFORCED CONCRETE.....	72
5.2 CURRENT USAGE OF PRESTRESSED CONCRETE	72
5.3 QUALITY	73
5.4 TIME	74
5.5 MATERIAL COST	75
5.5.1 MATERIAL COST FOR SLAB.....	75
5.5.2 MATERIAL COST FOR COLUMN.....	79
5.5.3 MATERIAL COST FOR GIRDER BEAM.....	80
5.6 DISCUSSION	81
5.7 COMPARISON OF HOUSING PROJECTS USING REINFORCED CONCRETE AND PRE-CAST PRESTRESSED CONCRETE	82
CHAPTER 6	83
CONCLUSIONS AND RECOMMENDATIONS	83
6.1 CONCLUSIONS.....	83
6.2 RECOMMENDATIONS	84
7. REFERENCES.....	85

LIST OF FIGURES

Figure 2.1: Methods of pretensioning	12
Figure 2.2: Hoyer's Long line system of pretensioning	13
Figure 2.3: parabola rectangle diagram for concrete under compression	19
Figure 2.4: rectangular stress distribution	20
Figure 2.5: concentric prestressing	22
Figure 2.6: Eccentric prestressing	22
Figure 2.7: Stress Distribution due to Eccentric prestressing, dead and live loads	23
Figure 2.8: Beam with eccentric tendons	24
Figure 2.9: Distribution of stresses at various sections along the span	24
Figure 2.10: Location of pressure line in the prestressed beam	24
Figure 2.11: Load carrying mechanism of reinforced concrete and prestressed concrete beam sections	25
Figure 2.12: Stress strain diagram for a prestressed beam section	27
Figure 2.13: principal tensile stress on a prestressed member	33
Figure 4.1: Layout of the T-section and the main girder beam	50
Figure 4.2: Magnel diagram for type 1	55
Figure 4.3: cross section for Type 1	61
Figure 4.4: modified cross section for Type 1	62
Figure 4.5: Distribution of reinforcement in the rib (T-section)	63
Figure 4.6: Connection of the ribbed slab to girder beam and girder beam to column	65
Figure 4.7 a: Frame beam to column connection detail elevation	70
Figure 4.7 b: Frame beam to column connection detail plan	71
Figure 4.8: Frame beam to column connection	71
Figure 5.1: usual problem in the current construction slabs	73
Figure 5.2: problems in stair connection of condominium site	74
Figure 5.3: cost summary	81

LIST OF TABLES

Table 2.1: Total losses in prestress	43
Table 3.1: Material properties for concrete and steel	45
Table 3.2: Load cases	45
Table 3.3: Seismic Data	45
Table 3.4: Load combinations	46
Table 3.5: Live load reduction factors	47
Table 3.6: Stiffness modification factors	47
Table 3.7: Stability index values	48
Table 4.1: T-sections dimensions for initial stage design	50
Table 4.2: Material properties used for initial design	51
Table 4.3: Modified geometric property	52
Table 4.4: Percentage losses calculation results for type 1	59
Table 4.5: Modified geometric property after increased web width	62
Table 4.6: Design summary for the rib (T-sections)	64
Table 4.7: Initial dimensions of the girder beam	65
Table 4.8: stiffness modification factors	66
Table 4.9: analysis result for the girder beams	67
Table 4.10: Design summary for the girder beams	69
Table 5.1: Cost summary for the reinforced concrete system slab	75
Table 5.2: Cost summary for the concrete used in Pre-cast prestressed concrete system slab	78
Table 5.3: Cost summary for the re-bar used in Pre-cast prestressed concrete system slab	78
Table 5.4: Cost summary for the prestressing strand used in Pre-cast prestressed concrete system slab...	79
Table 5.5: Cost summary for the steel used in columns	79
Table 5.6: Cost summary for the concrete used in columns	80
Table 5.7: Cost summary for the steel used in girder beams	80
Table 5.8: Cost summary for the concrete used in girder beams	81

LIST OF ANNEXES

ANNEX-A

DESIGN FIGURES AND TABLES88

ANNEX-B

MANUAL CALCULATION.....92

ANNEX-C

STRAND SPECIFICATION MANUAL148

ANNEX-D

DRAWINGS151

ANNEX-E

PRESTRESSED OR PRECAST SITE VIST DATA157

ANNEX-F

PICTURES.....161

LIST OF ABBREVIATIONS

- ϵ_{c2} strain reaching to the maximum strength
- ϵ_{cu2} ultimate strain
- σ_c compressive stress
- f_{ck} characteristics compressive strength
- f_{cd} design compressive strength
- λ effective height of the compression zone
- η effective strength
- P prestressing force (positive when producing direct compression)
- e eccentricity of the prestressing force
- A_C cross-sectional area of the concrete member
- I second moment of area of section about its centroid
- Z_t section modulus of the top fibers
- Z_b section modulus of the bottom fibers
- f_{sup} prestress in concrete developed at the top fibers
- f_{inf} prestress in concrete developed at the bottom fibers
- y_t distance of the top and bottom fibers from the centroid of the section
- y_b distance bottom fibers from the centroid of the section
- i radius of gyration
- A_p area of the prestressing wire
- σ_{pu} stress at the prestressing wire at the ultimate condition
- f_{sy} stress at the reinforcing bars
- σ_t stress at the prestressing wire at top transfer
- σ_b stress at the prestressing wire at bottom transfer
- $\sigma_{t,service}$ stress at the prestressing wire at top service
- $\sigma_{b,service}$ stress at the prestressing wire at bottom service
- f_{max} maximum allowable compressive stress in concrete at transfer
- $(f_{max})_{serv}$ maximum allowable compressive stress in concrete at service

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND TO THE HOUSING PROJECT PROBLEM

For years the practice of using prestressed concrete is applicable in developed countries such as Europe, United States of America, Russia and considerable nations of Asia. It is known that pre-stressed concrete will makes a remarkable contribution for the future development of the construction industry in developing nations including Ethiopia. As the rate of construction is increasing rapidly, architectural design limitation, construction time, quality, cost and serviceability should be major concerns.

Ethiopia is undergoing a development phase which includes construction of large number of housing projects and infrastructures. Rapid infrastructure development requires a large quantity and construction materials, where the quality of concrete should be taken into consideration. Currently, the undergoing housing project is one of the major infrastructural developments.

The aim of this housing project was to enable low-income urban dwellers acquire homes of their own, changing the image of the city so as to meet international standards, transfer of knowledge and skill to the construction industry and promoting cost efficient housing construction technology [21]. At present, the scope of this housing project has been widened to include middle income dwellers and fugitive Ethiopians. The G+4 building initially designed couldn't meet the space constraint so the project has been extended to G+12 living condos. In addition, to meet the ever increasing housing demand 40,000 houses per year and 200,000 houses in the first five years was intended to be completed [16].

Designing such condos requires a tight serviceability, safety, time and high quality requirements. Evaluating the handed over projects and progress of the construction, modification regarding the design, quality, construction technology and constructing time might come to play in favor of future improvements. Application of pre-stressed units for this housing project will enable the designer to amend the structural arrangement in such a manner where span of the members will not serve as a constraint. key advantages of using prestressed concrete over reinforced concrete are; a considerable saving in concrete and steel thus making the entire concrete cross-section more slender, smaller deflections, good crack control and therefore, permanent protection of the steel against corrosion, almost unchanged serviceability even after considerable overload, high fatigue strength, high punching shear strength and considerable reduction in construction time as a result of earlier striking of formwork [10]. Such a technology can also

be applicable in commercial buildings and residential houses. Thus, introducing the concept of prestressed in the housing project will further facilitate the application of prestressed by making such types of construction achievable.

1.2 OBJECTIVES

1.2.1 GENERAL OBJECTIVE

- To study the current housing projects in Ethiopia in view of cost effective design, timely delivery and use of high tech construction.
- Perform Structural analysis and design of Prestressed concrete
- Economically compare the newly developed prestressed concrete system with the current construction systems used.

1.2.2 SPECIFIC OBJECTIVES

- To assess if the time of the construction of the housing projects can be reduced.
- To help the project achieve its aim and meet the ever increasing housing demand in Ethiopia.
- To indicate the future Ethiopians construction technology in advance.
- To achieve better control of concrete quality.
- To check if improved system can be developed that is safe and serviceable.
- To conduct a cost comparison of the current project and the newly developed systems.
- To advance the type of buildings built to a more sophisticated level and give the architects more freedom while designing.
- To improve the fire safety
- Achieve the same structure with much more reduced load.
- To introduce a new design trend for building designs.
- To draw conclusions and give recommendations based on the research findings and indicate areas for further study.

1.3 APPLICATION OF RESULTS

The successful completion of this research will play a major role in the construction industry by means of:

- Introducing a solution to the delayed housing projects.
- Improving the traditional construction method to a more technologically advanced system.
- Creating awareness about pre-stressed concrete, its structural advantage, strength and availability
- Produce environmental friendly concrete products.
- Assist the current fast growing construction industry by providing alternatives.
- Enabling advancement in architectural designs which in turn helps improve the global image of Ethiopia.

1.4 LIMITATION / SCOPE OF THE RESEARCH

The scope and limitations of the research are:

- Scope of the research
 - Comparison mainly focuses on the girder beams and the slabs.
 - Architectural modification regarding the layout of frames is made.
 - Effect of large column spacing on the column section and reinforcement shown.
 - Design comparison is made regarding base reaction and column output for the two systems i.e reinforced concrete and pre-cast prestressed systems
 - Comparison is made regarding quality, timely deliverance and quantity of the two systems
- Limitations
 - Precise price of prestressing strands was not available
 - Prestressing and production cost was not available and price comparison focus on the material cost.
 - Detail design and Price of connection used for beam to column is not included
 - Transportation and installation cost are not included in the comparison.
 - Effect of the prestressing on the cantilever slabs is not studied.
 - Design of the stair is not covered in this research.

1.5 STRUCTURE OF THE RESEARCH

This research is structured with seven chapters and further break down in to different sections and sub sections. An introduction is provided with the objectives and limitation and scope of the study in the first chapter. In addition, statement of the problem is clearly stated in this chapter.

The second chapter consists of the fundamentals of prestressed concrete and its constituents by referring and reviewing different literatures. Designing property of prestressed concrete and its design process will be treated in this chapter. Also the different types of prestressing techniques and losses regarding of prestressed concretes are also constituents of the chapter.

Chapter three, deals with the details regarding modeling of the housing project by using a new structural arrangement. It addresses material properties used for the new structure, the loading conditions and design assumptions.

Chapter four will be focus on the analysis of the housing projects. It discusses the type of materials used for the production of prestressed concrete; software used for analysis, analysis type and analysis procedures. In the analysis procedure resulting design outputs were determined and used to economically analyze the two systems.

Chapter five highlights the discussion and economic analysis based on the quality, time and material cost comparison of the two systems regarding the two systems. It will illustrate and explain in detail the significance and contribution of each system. This will be followed by the last chapter, chapter six, which states the conclusion and recommendations derived from the research. Finally lists of reference material used to assist this research are listed together with annexes showing detailed design procedures and results. Photographic presentation is also attached in the Annex F.

CHAPTER 2

PRESTRESSED CONCRETE

2.1 INTRODUCTION

Prestressed concrete is basically concrete in which internal stresses of a suitable magnitude and distribution are introduced so that the stresses resulting from external loads are counteracted to a desired degree. In reinforced concrete members, the prestress is commonly introduced by tensioning the steel reinforcements. The earliest examples of wooden barrel construction by force fitting of metal bands and shrink fitting of metal tires on wooden wheels indicate that the art of prestressing has been practiced from ancient times. The tensile strength of plain concrete is only a fraction of its compressive strength and the problem of it being deficient in tensile strength appears to have been the driving factor in the development of the composite materials known reinforced concrete [23].

The development of early cracks in reinforced concrete due to incompatibility in the strains of steel and concrete was perhaps that starting point in the development of new material like "prestressed concrete". The application of permanent compressive stress to a material like concrete, which is strong in compression but weak in tension, increases the apparent tensile strength of that material, because the subsequent application of tensile stress must first nullify the compressive prestress [23].

The main difference between reinforced and prestressed concrete is the fact that reinforced concrete combines concrete and steel bars by simply putting them together and letting them act together as they may wish. Prestressed concrete, on the other hand combines high strength steel in an "active" manner. This is achieved by tensioning the steel and holding it against the concrete, thus putting the concrete into compression. This active combination results in a much better behavior of the two materials. Steel is ductile and now made to act in high tension by prestressing. Concrete is a brittle material with its tensile capacity now improved by being compressed, while its compressive capacity is not really harmed. Thus prestressed concrete is an ideal combination of modern high strength materials [32].

2.1.1 DEVELOPMENT OF PRESTRESSED CONCRETE

The historical development of prestressed concrete actually started in a different manner when prestressing was only intended to create permanent compression in concrete to improve its tensile strength. Later, it became clear that prestressing steel was also an essential to efficient utilization of high-tensile steel. As stated above the basic principles of prestressing were applied to construction perhaps centuries ago, when ropes or metal bands were wound around wooden staves to form barrels. When the bands were tightened, they were under tensile prestress which in turn created compressive prestress between the staves and thus enabled them to resist hoop tension produced by the internal liquid pressure. In other words, the band and the stave were both prestressed before they were subjected to any service load [32].

In 1886 P.H. Jackson, an engineer of San Francisco, California, obtained patents for tightening steel tie rods in artificial stones and concrete arches to serve as floor slabs. Around 1888, C.E.W. Doehring of Germany independently secured a patent for concrete reinforced with metal that had tensile stress applied to it before the slab was loaded. These applications were based on the conception that concrete, though strong in compression, was quite weak in tension, and prestressing steel against the concrete would put the concrete under compressive stress which would be utilized to counterbalance any tensile stress produced by dead or live-loads. These first patented methods were not successful because the low tensile prestress then produced in the steel was soon lost as a result of the shrinkage and creep of concrete. In 1908, C.R. Steiner of the United States suggested the possibility of retightening the reinforcing rods after some shrinkage and creep of concrete had taken place in order to recover some of the losses. In 1925, R.E. Dill of Nebraska tried high-strength steel bars coated to prevent bond with concrete. After the concrete had set, the steel rods were tensioned and anchored to the concrete by means of nuts. But these methods were not applied chiefly for economic reasons [32].

Modern development of prestressed concrete is credited to E. Freyssinet of France, who in 1928 started using high-strength steel wires for prestressing. Such wires, with ultimate strength as high as 250,000 psi (1,725N/mm²) and a yield point over 180,000 psi (1,240N/mm²), are prestressed to about 145,000psi (1,000N/mm²), creating a unit strain of 0.0050 [32].

$$\delta = \frac{f}{E} = \frac{145,000}{29,000,000} = 0.0050$$

Assuming a total loss of 0.0008 due to shrinkage and creep of concrete and other causes, a net strain of 0.0050-0.0008=0.0042 would still be left in the wires which is equivalent stress of;

$$f = E \delta = 29,000,000 * 0.0042 = 121,800 \text{psi (840N/mm}^2\text{)}$$

Although Freyssient also tried the scheme of pretensioning where the steel was bonded to the concrete without end anchorage, practical application of this method was first made by E.Hoyer of Germany. The Hoyer system consist of stretching wires between two buttresses several hundred feet apart, putting shutters between the units, placing the concrete and cutting the wires after the concrete has hardened. This method enables several units to be cast between two buttresses [32].

Wide application of prestressed concrete was not possible until reliable and economical methods of tensioning and end anchorages were devised. In 1939, Freyssient developed conical wedges for end anchorages and designed double acting jacks which tensioned the wires and then thrust the male cones onto the female cones for anchoring them. In 1940, professor G.Magel of Belgium developed a Magnel system, wherein two wires were stretched at a time and anchored with simple metal wedge at each end. About that time, prestressed concrete began to acquire importance, though it did not actually come to the fore until about 1945. Perhaps the shortage of steel in Europe during the war had given it some impetus, since much less steel is required for prestressed concrete than for conventional types of construction. But it must also be realized that time was needed to prove and improve the serviceability, economy, and safety of prestressed concrete as well as to acquaint engineers and builders with a new method of design and construction [32].

Although France and Belgium led the development of prestressed concrete, England, Germany, Switzerland, Holland and soviet Russia and Italy quickly followed. Since 1965, about 47% of all bridges built in Germany were of prestressed concrete. Soviet Russia annually produced 25,000,000m³ of prestressed concrete in 1978, most of which was pretensioned products for buildings. In United States, while there was only one precast pretensioning plant in 1950, there were 229 in 1961. The total volume of precast prestressed products was estimated over 2,000,000yd³ (1,530,000m³) in 1962, of which it was roughly estimated that 50% went to bridges and the remaining to buildings and other construction projects. A survey by prestressed concrete institute in 1975 indicated that 500 pre-casting and prestressing plants were operating in the United States [32].

2.1.2 ADVANTAGES AND LIMITATIONS OF PRESTRESSED CONCRETE

The prestressing of concrete has several advantages as compared to traditional reinforced concrete without prestressing. A fully prestressed concrete member is usually subjected to compression during service life. This rectifies several deficiencies of concrete. A main deficiency of reinforced concrete members is the cracking of concrete, but the prestressing overcomes the cracking. For a fully prestressed member, the concrete remains un-cracked under the service conditions, and this leads to several advantages of prestressed concrete. If the concrete is un-cracked, the primary advantage that we get is the

reduction of deterioration and corrosion. The reduction of steel corrosion leads to increased durability of precast and prestressed member. The next advantage is that the full section is utilized. That means the section will have a higher moment of inertia, which will give higher stiffness, this will lead to less deformation, which in turn will have improved serviceability. Since the section is un-cracked, there is also an increase in the shear capacity. An un-cracked member is suitable to be used in pressure vessels and in liquid storage tanks. Because in these types of structures, we do not want cracks through which the liquid can seep out or, which can create a hazard. Hence, prestressing is applied in these types of structures. And also, we can see that there is an improved performance due to resilience under dynamic and fatigue loading. Big structures can undergo several cycles of loading which can induce fatigue in a member; but when the member is prestressed; its behavior gets better [4].

The second advantage is the high span-to-depth ratios for prestressed concrete members. Since the span-to-depth ratio can be high, prestressing is applied for large spans like bridges, buildings with large column free spaces. Typical values of span-to-depth ratios in slabs are as follows: for a non-prestressed slab, the span-to-depth ratio can be 28:1; for a prestressed slab, the ratio can increase to 45:1. That means, as we are prestressing the slab, the clear span can be much larger compared to an equivalent reinforced concrete (RC) slab, and this helps in the future use of the building. For the same span, the depth of a prestressed member is less than the corresponding RC member. It leads to reduction in self weight, more aesthetic appeal due to slender sections, and finally it gives a more economical section because the amount of concrete used is much less [4].

The next advantage of prestressing is that, it is very much suitable for precast construction. The advantages of precast construction are as follows: it is a rapid construction; there is better quality control, reduced maintenance; it is suitable for repetitive construction; multiple use of formwork, which leads to reduction of formwork, and availability of standard shapes. Thus, prestressing goes very much hand-in-hand with precast construction. The advantages of precast construction can be fully utilized, and this leads to a much better and faster construction [4].

The advantages and disadvantages of prestressed concrete as compared with reinforced concrete with respect to their serviceability, safety and economy are discussed below.

Serviceability: Prestressed-concrete design is more suitable for structures of long spans and those carrying heavy loads, principally because of the higher strengths of materials employed. Prestressed structures are more slender and hence more adaptable to artistic treatment. They yield more clearance where it is needed. They do not crack under working loads, and whatever cracks may be developed under overloads will be closed up as soon as the load is removed, unless the load is excessive. Under dead loads,

the deflection is reduced, owing to the cambering effect of prestress. This becomes an important consideration for such structures as long cantilevers. Under live loads, the deflection is also smaller because of the effectiveness of the entire un-cracked concrete section, which has a moment of inertia two to three times that of the cracked section. Prestressed elements are more adaptable to precasting because of lighter weight. So as far as serviceability is concerned, the only shortcoming of prestressed concrete is its lack of weight. Although seldom encountered in practice, there are situations where weights and mass are desired instead of strength. For these situations, plain or reinforced concrete could often serve just as well and at lower cost [32].

Safety: it is difficult to say that one type of structure is safer than another. The safety of structures depend more on its design and construction than on its type. However, certain inherent safety features in prestressed concrete may be mentioned. There is partial testing of both the steel and concrete during prestressing operations. For many structures, during prestressing both the steel and the concrete are subjected to the highest stress that will exist in them during their life of service. Hence, if the materials can stand prestressing, they are likely to possess sufficient strength for the service loads. When properly designed by present conventional methods, prestressed concrete structures have overload capacities similar to and perhaps slightly higher than those of reinforced concrete. For the usual design, they deflect appreciably before ultimate failure, thus giving ample warning before impending collapse. The ability to resist shock and impact loads and repeated working loads has been shown to be as good in prestressed as in reinforced concrete. The resistance to corrosion is better than that of reinforced concrete for the same amount of cover, owing to the nonexistence of cracks and high quality of concrete used for prestressed members. If cracks should occur, corrosion can be more serious in prestressed concrete. Regarding fire resistance, high-tensile steel is more sensitive to high temperatures, but for the same amount of minimum cover, prestressed tendons can have a greater average cover because of the spread and curvature of the individual tendons [32]

Economy: from economics point of view, it is at once evident that smaller quantities of materials, both steel and concrete are required to carry the same loads, since the materials are of high strength. There is also a definite saving in stirrups, since shear in prestressed concrete is reduced by the presence of prestress. The reduced weight of the member will help in economizing the sections; the smaller dead load and depth of members will result in saving materials from other portions of the structure. In precast members, a reduction of weight saves handling and transportation costs. In spite of the above economics possible with prestressed concrete, its use cannot be advocated for all conditions. First of all, the stronger materials will have a higher unit cost. More auxiliary materials are required for prestressing, such as end anchorages, conduits, and grouts. More complicated formwork is also needed, since nonrectangular

shapes are often necessary for prestressed concrete. More labor is required to place 1lb of steel in prestressed concrete, especially when the amount of work involved is small. More attention to design is involved and more supervision is necessary; the amount of additional work will depend on the experience of the engineer and the construction crew, but it will not be serious if the same typical design is repeated many times. It can be concluded that prestressed concrete design is more likely to be economical when the same unit is repeated many times or when heavy dead loads on long spans are encountered. It should find suitable application when combined with pre-casting or semi pre-casting such as composite or lift-slab construction. Each structure must be considered individually. The availability of good designers, of experienced crews, of pretensioning factories, and of competitive bidding often helps to tip the balance in favor of prestressed concrete [32].

Thus, limitations of prestressed concrete can be summarized as follows; first, it needs skilled technology. Hence, it is not as common as reinforced concrete. Next, use of high strength materials is costly. Then, there is an additional cost in auxiliary equipment. Finally, there is need for better quality control and inspection. Even though there are many advantages of prestressing, it should be noted that the additional advantages bring in responsibilities. Good technology is required to implement the prestressing. There should be good quality control, so that the advantages that we are expecting will be materialized. Hence, care should be taken in manufacturing prestressed concrete products [4].

2.2 TYPES OF PRESTRESSING

Pre-stressing of concrete can be classified in several ways based on;

- The source of prestressing force
- The location of the prestressing tendon with respect to the concrete section
- The sequence of casting the concrete and applying tension to the tendons
- The shape of the member prestressed
- The amount of prestressing force
- The directions of the prestressing member
- The concrete casting mechanism
- The types of anchorages used

2.2.1 SOURCE OF PRESTRESSING FORCE

The first classification is based on the source of prestressing force. This classification is based on the method by which the prestressing force is generated. There are four sources of prestressing force: mechanical, hydraulic, electrical and chemical [4].

Hydraulic prestressing: This is the simplest type of prestressing producing large prestressing forces. The hydraulic jack used for the tensioning of tendons comprises of calibrated pressure gauges, which directly indicate the magnitude of force developed during the tensioning. This is the most common form of applying the prestress to the steel, which is then transferred to the concrete. Hydraulic jacks which operate based on oil pressure are used to apply the prestress [4].

Mechanical prestressing: In this type of prestressing, the devices include weights with or without lever transmission, geared transmission in conjunction with pulley blocks, screw jacks with or without gear drives, and wire-winding machines. This type of prestressing is adopted for mass scale production. Thus, the mechanical prestressing is based on equipments with mechanical components, and these equipments are more popular when mass scale production of prestressed members is adopted [4].

Electrical prestressing: In this type of prestressing, the steel wires are electrically heated and anchored, before placing concrete in the moulds. This type of prestressing is also known as thermo-electric prestressing. That means, when the wires are heated they expand, and then they are allowed to cool down, which transfers the prestress to the concrete [4].

Chemical prestressing: In this type of prestressing, expansive cements are used, and the degree of expansion is controlled by varying the curing conditions. The expansive action of cement is restrained while setting. This generates tensile forces in the tendons and compressive stresses in concrete. This chemical prestressing is relatively rare, but it can be used in order to transfer prestress to the concrete [4].

2.2.2 LOCATION OF THE PRESTRESSING TENDON WITH RESPECT TO THE CONCRETE SECTION

External prestressing: When the prestressing is achieved by elements located outside the concrete member, for example, by cables lying outside a beam, it is called external prestressing. This technique is adopted in repair and strengthening works, such as retrofitting of bridges [4].

Internal prestressing: When the prestressing is achieved by elements located inside the concrete member commonly by embedded tendons, it is called internal prestressing [4].

2.2.3 SEQUENCE OF CASTING THE CONCRETE AND APPLYING TENSION TO THE TENDONS

2.2.3.1 PRETENSIONING SYSTEMS

In pretensioning systems, the tendons are first tensioned between rigid anchor blocks cast on the ground or in column or unit-mold type pretensioning bed, prior to the casting of concrete in the moulds. High early-strength concrete is often used in a factory to facilitate early stripping and reuse of moulds. When the concrete attains sufficient strength, the jacking pressure is released. A typical column type pretensioning bed is shown in the Figure 2.1. The tendons comprising individual wires or strands are stretched with constant eccentricity as shown in (a) or variable eccentricity as shown in (b) with tendon anchorage at one end and jack at other. The high-tensile wires tends to shorten but are checked by bond between concrete and steel. In this way the prestress is transferred to the concrete by bond, mostly near the ends of the beam and no special anchorages are required in pretensioned members [23].

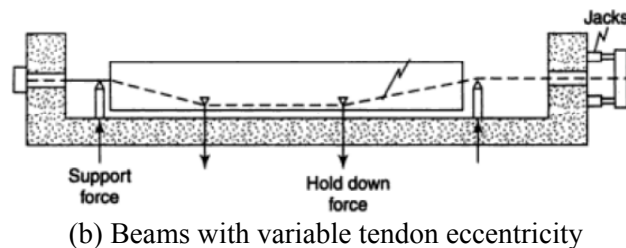
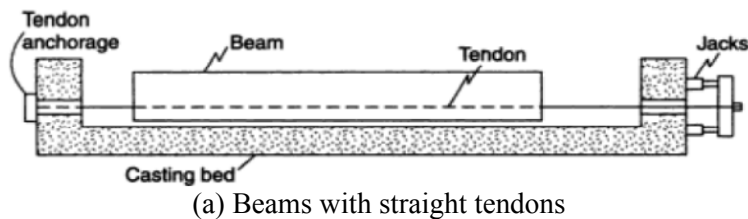


Figure 2.1: Methods of pretensioning [23]

For mass production of pretensioned elements, the long-line process developed by Hoyer is generally used in factory. In this method the tendons are stretched between two bulk heads several hundred meters apart so that a number of similar units may cast along the same group of tensioned wires as shown in Figure 2.2. The tension is applied by hydraulic jacks or by a moveable stressing machine. The wires or strands when tensioned singly or in group are generally anchored to the abutments by steel wedges [23].

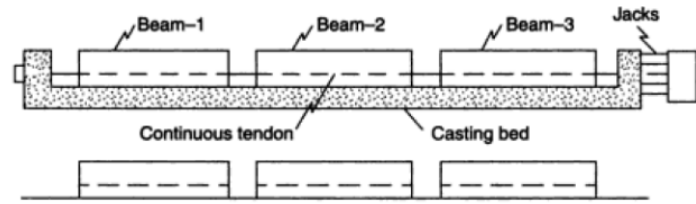


Figure 2.2: Hoyer's Long line system of pretensioning [23]

The transfer of prestress to the concrete is usually achieved by large hydraulic or screw jacks by which all the wires are simultaneously released after concrete attains the requisite compressive strength. Generally, strands of up to 18mm diameter and high tensile wires up to 7mm diameter anchor themselves satisfactorily with the help of the surface bond and the interlocking of the surrounding matrix in the micro indentations on the wires. The bond of prestressing wires may be considerably improved by forming surface indentations and by helical crimping of the wires. Strands have considerably better bond characteristics than plain wires. Strands have considerably better bond characteristics than plain wires of equal cross-sectional area. Supplementary anchoring devices are required when single wires of larger diameter (exceeding 7mm) are used in pretensioned units. The most commonly used devices are 'weinberg clips' developed in France and the 'Dorland clips' developed in the United States. These clips are clamped on to the tensioned wires close to the end diaphragms of the units before concreting operations [23].

The advantages of pre-tensioning as compared to post-tensioning are [4];

- Suitable for precast members produced in bulk
- Absence of large anchorage devices

The disadvantages of pre-tensioning are [4];

- Requirements of pre-stressing beds
- Waiting period in pre-stressing beds before concrete attains sufficient strength
- Requirements good bond between concrete and steel over the transmission length

2.2.3.2 POST-TENSIONING SYSTEMS

In the post-tensioning, the concrete units are first cast by incorporating ducts or groove to the house the tendons. When the concrete attains sufficient strength, the high -tensile wires are tensioned by means of jack bearing on the end face of the members and anchored by wedges or nuts. The force is transmitted to the concrete by means of the end anchorages and, when the cable is curved, through the radial pressure

between the cable and the duct. The space between the tendons and the duct is generally grouted after the tensioning operation [23].

Most commercially patented prestressing systems are based on the following principles of anchoring the tendons [23]:

- Wedge action producing a frictional grip on the wires.
- Direct bearing from rivet or bolt heads formed at the end of the wires.
- Looping the wires around the concrete.

Post-tensioning is ideally suited for medium to long span in situ work where the tensioning cost is only a small proportion of the cost of the whole job. Hence it is more economical to use a few cables or bars with large forces in each than a large number of small ones. Post-tensioning may be used with advantage to fabricate large members, such as long span bridge decks of the box-girder type by prestressing together a number of small pre-cast units. Apart from these advantages, the chief merit of post-tensioning is that it allows the use of curved and stopped-off cables which helps the designer to vary the prestress distribution at will from section to section so as to counter the external loads more efficiently. Post-tensioning is invariably used for strengthening concrete dams, circular prestressing of large concrete tanks and biological shields of nuclear reactors. Post-tensioning is ideally suited in concrete construction work involving stage prestressing. Most of the long-span bridge structures are constructed using post-tensioning systems [23].

Advantages of post-tensioning as compared to pre-tensioning are [4];

- Suitable for heavy cast in place members
- Less waiting period in the casting bed
- Transfer of prestress independent of transmission length

Disadvantages of post-tensioning as compared to pre-tensioning [4];

- The requirement of anchorage device and grouting equipment

2.2.4 SHAPE OF THE PRESTRESSED MEMBER

This classification is based on the direction. When the prestressed members are straight or flat in the direction of prestressing, the prestressing is called linear prestressing. For example, the prestressing of beams, piles, poles and slabs. The prestressing cable profile may be curved. That is, if the prestressing tendons are along the line of the member then it is called linear prestressing. The next type is the circular

prestressing. When the prestressed members are curved, the direction of prestressing also changes and this prestressing is called the circular prestressing. For example, circumferential prestressing of tanks, silos, pipes and similar structures [4].

2.2.5 AMOUNT OF PRESTRESSING FORCE

This classification is related with the amount of prestressing.

Full prestressing: When the level of prestressing is such that no tensile stress is allowed in the concrete under service loads, it is called full prestressing. Thus, in a fully prestressed member, there cannot be any tension in the concrete during the service life [4].

Limited prestressing: When the level of prestressing is such that the tensile stress under service loads is within the cracking stress of concrete, it is called limited prestressing. Thus in a limited prestressed member, tension is allowed in concrete but it is made sure that it is lower than the tensile strength of the concrete. Hence, cracking will not develop within the concrete [4].

Partial prestressing: The partial prestressing means the level of prestressing is such that under tensile stresses due to service loads, there can be cracking and the crack width is limited within some allowable values. Thus, in partial prestressing we allow not only tensile stresses but also cracking of the concrete. The crack width will be limited within allowable values [4].

2.2.6 BASED ON DIRECTIONS OF THE PRESTRESSING MEMBER

Based on the different directions of prestressing prestressed members could be uniaxial, biaxial or multiaxial. When the prestressing cables are parallel to one axis, it is called uniaxial prestressing, for example, longitudinal prestressing of beams. When there are prestressing cables parallel to two axes, it is called biaxial prestressing; for example biaxial prestressing of slabs. When the prestressing cables are parallel to more than two axes, it is called multiaxial prestressing; for example, prestressing of domes. In domes of some big structures, prestressing can be done in more than two directions and in that case, it is called a multiaxial prestressing.

2.2.7 CONCRETE CASTING MECHANISM

Depending on the concrete casting mechanisms prestressed members can be categorized as cast in-situ, pre-cast and composite structural members.

Precast members: precasting involves the placing of concrete away from its final position, the members being cast either in permanent plant or somewhere near the site of the structure, and eventually erected at the final location. Precasting permits better control in mass production and is often economical [32].

Cast in-situ members: cast in place concrete requires more form and false-work per unit of product but saves the cost of transportation and erection and it is a necessity for large and heavy members [32].

Composite structural members: In between pre-cast and cast in-situ there are tilt-up wall panels and lift slabs which are constructed at place near or within the structure and then erected to their final portions; no transportation is involved for these. Oftentimes, it is economical to pre-cast part of members, erect it, then cast the remaining portion in place. This procedure is called composite construction. The pre-cast elements in a structure of composite construction can be more easily joined together than those in a totally pre-cast structure. By composite construction, it is possible to save much of the form and false-work required for total cast-in-place construction. However, the suitability of each type must be studied with respect to particular condition of given structure [32].

2.3 PRESTRESSING SYSTEMS AND DEVICES

2.3.1 PRE-TENSIONING SYSTEMS AND DEVICES

Stages of the pre-tensioning operation is summarized as follows

- Anchoring of tendons against the end abutments
- Placing of the jacks
- Applying tension to the tendons
- Casting of concrete
- Curing of concrete
- Cutting of the tendons

Essential devices for pre-tensioning are as follows;

- Prestressing beds with end abutments: extension of this system is the Hoyer system which is generally used in mass production. The system is also called the long line method. The abutments should be sufficiently stiff and should have good foundations. This is usually an expensive proposition particularly when large prestressing force are required but
- Moulds/shuttering
- Jacks

- Anchoring devices
- Harping devices (optional)

Anchoring devices are often made of wedge and friction principle. In pre-tensioned members, the tendons are to be held in tension during the setting and hardening of concrete. Here simple and cheap quick release grip are generally adopted.

The tendons are frequently bent, except in the case of slabs-on grade, poles, piles etc... the tendons are bent (harped) in between the supports with shallow sag.

2.3.2 POST-TENSIONING SYSTEMS AND DEVICES

The various stages of post-tensioning can be summarized as follows [4]

- Casting of concrete
- Placements of tendons
- Placement of the anchorage bocks and jack
- Applying tension to the tendons
- Seating of wedges
- Cutting of the tendons

The economical devices for post-tensioning are as follows

- Casting beds
- Shuttering
- Ducts
- Anchoring devices
- Jacks
- Couplers (optional devices used to connect the strands or bars) they are located at the junction of members and they are tested to transmit the full capacity of the strands or bars.
- Grouting equipments (optional) it is used for filling of the duct with a material that provides an anti-corrosive alkaline environment to the prestressing steel also a strong bond between the tendons and the surrounding concrete.

Anchoring devices for post-tensioning beams transfer the prestress to the concrete. The devices are based on the following principles of anchoring principles [4];

- Wedge action: it produces a frictional grip on the wires. The anchorage device based on wedge action consisting of an anchorage blocks and wedges. The strands are held by the wedge in the anchorage block. Some examples of the of the systems based on the wedge-action are Freyssinet, Gifford-Udall, Anderson an Manel Halton anchorages.
- Direct bearing from rivet or bolt-heads or buttons-heads formed at the end of the wires. The B.B.R.V post-tensioning systems and the Prescon systems are based on this principle .
- Looping wires around the concrete: it is used for a single wire at time. The Baur-leonhardt systems, Leoba-systems and also the Dwidag-single bar anchorage system, work on this principle. The anchorage devices are tested to calculate their strength and fatigue characteristics.

2.4 MATERIALS FOR PRESTRESSED CONCRETE

2.4.1 CONCRETE

Prestressed concrete requires concrete which has a high compressive strength at a reasonably early age, with comparatively higher tensile strength than ordinarily concrete. Low shrinkage, minimum creep characteristics and high value of young modulus are generally deemed necessary for concrete used for prestressed members. Many desirable properties, such as durability, impermeability and abrasion resistance, are highly influenced by the strength of concrete. The minimum 28-cube day cube strength recommended in euro code manual for design of reinforced concrete [31] is C30/37for post-tensioned members and C40/50 for pretensioned members.

The stress - strain behavior of concrete under uniaxial compression is initially linear and elastic. With generation of micro cracks the behavior becomes nonlinear and inelastic. After the specimen reaches the pick stress the resisting stress decreases with increase in strain.

Euro code 2 part 1-1 1992 states the stress -strain relationship for design of cross section as;

$$\sigma_c = f_{cd} \left[1 - \left(1 - \frac{\epsilon_c}{\epsilon_{c2}} \right)^n \right], \text{ For } 0 \leq \epsilon_c \leq \epsilon_{c2}$$

$$\sigma_c = f_{cd}, \text{ For } \epsilon_{c2} \leq \epsilon_c \leq \epsilon_{cu2}$$

Where:

n is the exponent according to Table 3.1 of Euro code 2 part 1-1 1992

ϵ_{c2} is the strain reaching to the maximum strength according to Table 3.1 of Euro code 2 part 1-1 1992

ϵ_{cu2} is the ultimate strain according to Table 3.1 of Euro code 2 part 1-1 1992

σ_c is the compressive stress

f_{ck} is the characteristics compressive strength

f_{cd} is the design compressive strength

Parabola- rectangular diagram can be plotted using the above relation and it's shown in Figure 2.3 below.

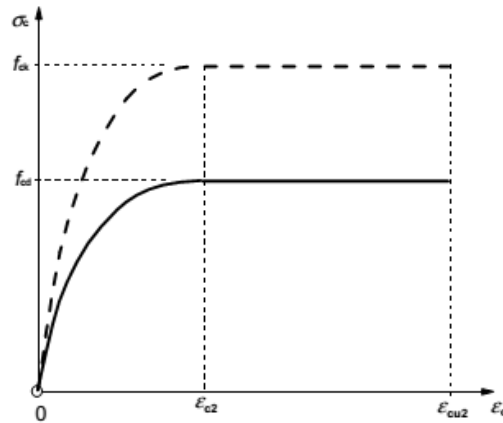


Figure 2.3: parabola rectangle diagram for concrete under compression [12].

The rectangular stress distribution as shown in Figure 2.4 may be assumed. The factor λ , defining the effective height of the compression zone and the factor η , defining the effective strength, follow from [12]:

$$\lambda = 0.8 \quad \text{for } f_{ck} \leq 50 \text{ MPa}$$

$$\lambda = 0.8 - (f_{ck} - 50) / 400 \quad \text{for } 50 < f_{ck} \leq 90 \text{ MPa}$$

And

$$\eta = 1.0 \quad \text{for } f_{ck} \leq 50 \text{ MPa}$$

$$\eta = 1.0 - (f_{ck} - 50) / 200 \quad \text{for } 50 < f_{ck} \leq 90 \text{ MPa}$$

If the width of the compression zone decreases in the direction of the extreme compression fiber, the value ηf_{cd} should be reduced by 10%.

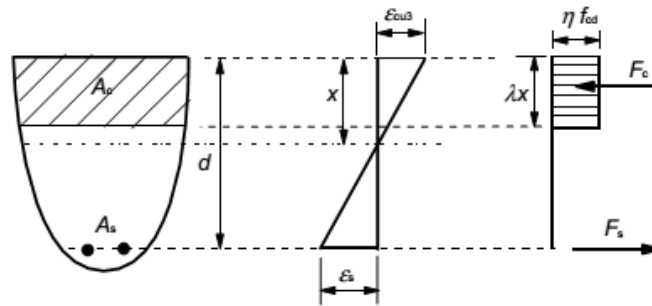


Figure 2.4: Rectangular stress distribution [12].

2.4.2 PRESTRESSING STEEL

Forms of prestressing steel can be listed as follows:

- Wires: single unit made of steel. Nominal diameter of wires are 2.5mm, 3.0mm, 4.0mm, 5.0mm, 7.0mm, and 8.0mm. Wires can be plain or indented wires.
- Strands: few wires spun together in a helical form to form a prestressing strand. Strands can be two-wire strand, three-wire strands or seven-wire strand.
- Tendon: a group of strands or wires wound to form a prestressing tendon.
- Cable: a group of tendons.
- Bars: a tendon can be made up of a single steel bar. The diameter of the bar is much larger than that of a wire.

Properties of prestressing steel:

- High strength
- Adequate ductility
- High bond: require in pre-tensioned members
- Low relaxation to reduce loss
- Minimum corrosion

Strength of prestressing steel is important property of prestressing steel. The tensile strength of the prestressing steel is given in terms of characteristic tensile strength, denoted by f_{pk} . The characteristic strength is defined as the ultimate tensile strength of coupon specimens below which not more than 5% of the test results are expected to fall. The stiffness of prestressing steel is given by the initial modulus of elasticity.

2.5 ANALYSIS AND DESIGN OF STRUCTURES USING PRESTRESSED CONCRETE

2.5.1 ANALYSIS FOR FLEXURE

2.5.1.1 STRESSES IN PRESTRESSED CONCRETE BEAMS

Concrete is visualized as being subjected to two systems of forces: internal prestress and external load, with tensile stress due to external load counteracted by the compressive stress due to the prestress. Similarly, the cracking of concrete due to load is prevented or delayed by pre-compression produced by the tendons. So long as there are no cracks, the stresses, strains and deflections of the concrete due to the two systems of forces can be considered separately and superimposed if necessary [32].

The following notations and sign conventions are used for the analysis of prestress

P = prestressing force (positive when producing direct compression)

e = eccentricity of the prestressing force

$M = P.e$ = moment

A = cross-sectional area of the concrete member

I = second moment of area of section about its centroid

Z_t and Z_b = section modulus of the top and bottom fibers

f_{sup} and f_{inf} = prestress in concrete developed at the top and bottom fibers (positive when compressive and negative when tensile in nature)

y_t and y_b = distance of the top and bottom fibers from the centroid of the section

i = radius of gyration

Concentric tendons: consider a concrete beam with concentric tendons as shown in the Figure 2.5 below, uniform prestress in concrete = P/A , which is compressive across the depth of the beam. Generally, the applied loads of the beam induce tensile stress towards the soffit and are counterbalanced more effectively by eccentric tendons [23].

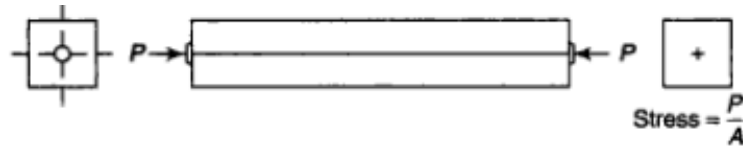


Figure 2.5: concentric prestressing [23]

Eccentric tendons: Figure 2.6 shows a beam subjected to an eccentric prestressing force of magnitude P located at eccentricity e. the stress developed at the top and bottom fibers of the beam are obtained by the relations:

$$f_{ins} = \left(\frac{P}{A} + \frac{Pe}{Z_b} \right) = \frac{P}{A} \left(1 + \frac{ey_b}{i^2} \right), \quad (2.1)$$

$$f_{sup} = \left(\frac{P}{A} - \frac{Pe}{Z_t} \right) = \frac{P}{A} \left(\frac{ey_t}{i^2} - 1 \right), \quad (2.2)$$

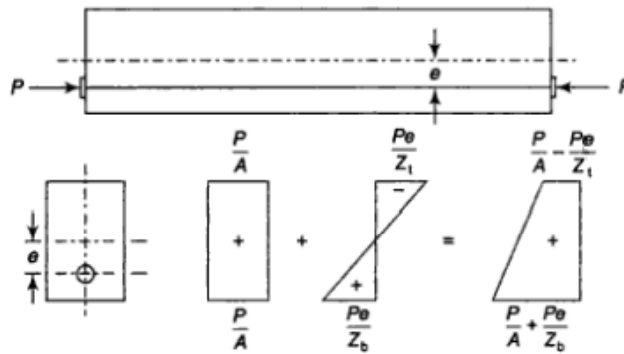


Figure 2.6: Eccentric prestressing [23]

2.5.1.2 RESULTANT STRESSES AT A SECTION

The concrete beam supports uniformly distributed live and dead loads of intensity q and g. The beam is prestressed by a straight tendon carrying a prestressing force P at an eccentricity e as shown in Figure2.7. The resultant stresses in the concrete at any section are obtained by superimposing the effect of prestress and flexural stresses developed due to loads. If M_q and M_g is the live and dead load moments at the central span section,

$$M_q = \frac{qL^2}{8}, \quad (2.3)$$

$$M_g = \frac{gL^2}{8}, \quad (2.4)$$

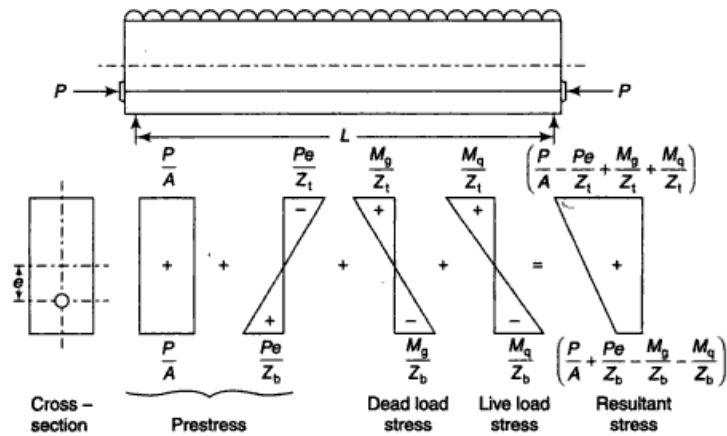


Figure 2.7: Stress Distribution due to Eccentric prestressing, dead and live loads [23]

The resultant stresses at the top and bottom fibers of concrete at any section are obtained as:

$$f_{\text{ins}} = \left(\frac{P}{A} + \frac{Pe}{Z_b}\right) - \left(\frac{M_g}{Z_b}\right) - \left(\frac{M_l}{Z_t}\right), \quad (2.5)$$

$$f_{\text{sup}} = \left(\frac{P}{A} - \frac{Pe}{Z_t}\right) + \left(\frac{M_g}{Z_t}\right) + \left(\frac{M_l}{Z_t}\right), \quad (2.6)$$

In case of prestressed members, the cross-sectional area of the high-tensile steel being very small percentage of the total concrete, the stress computation are generally based on the nominal concrete cross sectional properties. The use of equivalent concrete section, although important in interpreting test results of the experimental investigation, generally does not significantly influence the stress resulting from the use of nominal concrete section.

At any given section of prestressed concrete beam, the combined effect of the prestressing force and externally applied load will result distribution of concrete stresses that can be resolved into a single force. The locus of the points of application of this resultant force in any structure is termed as 'pressure trust line'. The concept of pressure trust line is very useful in understanding the load-carrying mechanism of a prestressed concrete section. In the case of prestressed concrete beam the location of the pressure line depends upon the magnitude and distribution of stress due to the prestressing force.

Consider a concrete beam shown in Figure 2.8, which is prestressed by force \$P\$ acting at eccentricity \$e\$. the beam supports uniformly distributed load (including self-weight) of intensity \$q\$ per unit length.

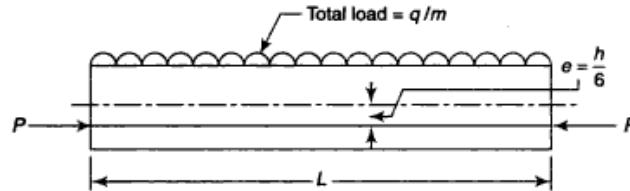


Figure 2.8: Beam with eccentric tendons [23]

The load is of such magnitude that the bottom-fiber stress at the central span section of the beam is zero. Figure 2.9 shows the resultant stress distribution at the support, center and quarter span sections of the beam. At the support section, since there are no flexural stresses resulting from the external loads, the pressure line coincides with that of the centroid of steel, located at an eccentricity of $h/6$. At the center of the span section, the external loading is such that the resultant stress developed is maximum at the top fiber and zero at the bottom fiber.

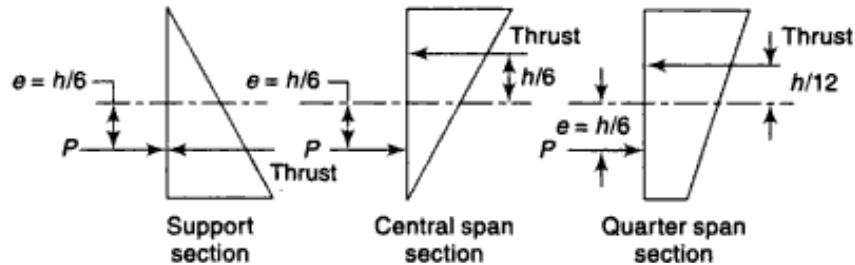


Figure 2.9: Distribution of stresses at various sections along the span [23]

It can easily be seen that for this section the pressure line has shifted towards the top fiber by an amount equal to $h/3$ from its initial position. The external moment at the quarter span section being smaller in magnitude, the shift in pressure line also is correspondingly smaller, being equal to $h/4$ from the initial position. In a similar manner, it can be shown that a larger uniformly distributed load on the beam would result in the pressure line being shifted even higher at the center and quarter span sections. The pressure line location in the beam is shown in Figure 2.10. These observations lead to the following important principles: "A change in the external moments in the elastic range of a prestressed concrete beam results in a shift of the pressure line rather than in an increase in the resultant force in the beam."

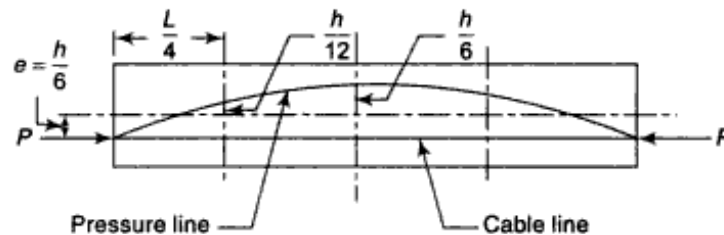


Figure 2.10: Location of pressure line in the prestressed beam [23]

This is in contrast to a reinforced concrete beam section, where an increase in the external moments results in corresponding increase in the tensile force and the compressive force. The increase in the resultant force are due to a more or less constant lever arm, as in the case of prestressed concrete sections, and a changing force with a constant lever arm prevailing in reinforced concrete sections as shown in Figure 2.11. However, if prestressed concrete member is cracked, it behaves in a manner similar to that of a reinforced concrete section. The pressure or trust line concept can be use to evaluate the stresses. In these methods, generally referred to as the internal resisting couple method or the c-line method, the prestressed beam is analyzed as a plain concrete elastic beam using the basic principles in statics. The prestressing force is considered as an external compressive force with a constant tensile force T in the tendon throughout the span. Consequently, at any section of a loaded pressured beam, equilibrium is maintained satisfying the equations, $H=0$ and $M=0$.

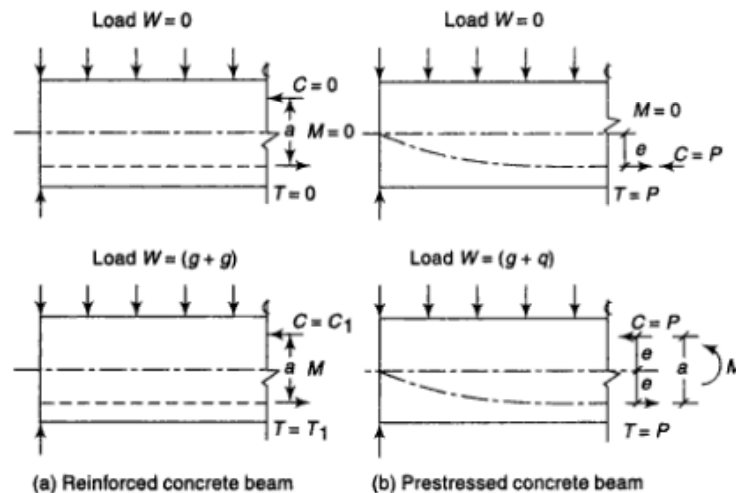


Figure 2.11: Load carrying mechanism of reinforced concrete and prestressed concrete beam sections[23]

When the gravity loads are zero, the C and T lines coincide since there is no moment at the section. Under transverse loads, the C line, or the center of pressure or trust line is at varying distance from the T -line.

If M = bending moment at the section due to dead and live loads

e = eccentricity of the tendon

$T=P$ = prestressing force in the tendon

Moment equilibrium yields the relation,

$$M = Ca = Ta = Pa \text{ and } a = (M/P)$$

The shift of pressure line e measured from the centroidal axis is obtained as; $e' = (a - e) = (M/P) - e$

The resultant stress at the top and bottom fibers of the section are expressed as,

$$f_{ins} = \left(\frac{P}{A}\right) - \left(\frac{pe'}{Z_b}\right), \quad (2.7)$$

$$f_{sup} = \left(\frac{P}{A}\right) + \left(\frac{pe'}{Z_t}\right), \quad (2.8)$$

Where, Z_t and Z_b are the section moduli of the top and bottom fibers respectively.

- Decompression moment

Decompression moment (M_{dec}) is the total moment at which the concrete stress in the bottom fiber is zero

$$M_{dec} = Z_b \left[\left(\frac{P}{Ag}\right) + \left(\frac{pe}{Z_b}\right) \right], \quad (2.9)$$

- Cracking moment

The total moment which is just enough to cause tensile cracking in the bottom fiber is called cracking moment (M_{cr}). $\sigma_b = -f_t$

$$M_{cr} = Z_b \left[\left(\frac{P}{Ag}\right) + \left(\frac{pe}{Z_b}\right) + f_t \right], \quad (2.10)$$

- Ultimate moment

In a post-cracking range, the prestressing steel acts more or like reinforcing steel. Hence, prestressing steel in the tension region contributes significantly to the moment capacity of the section. To calculate the ultimate moment capacity (M_u) with beams with prestressing we will need to know the stress conditions at different levels. Figure 2.12 shows the strain and stress diagram .Just after the prestressing is applied to the beam the concrete will have a compression strain (ϵ_{ce}) due to the prestressing wire and there will be a strain in the prestressing wire (ϵ_{pe}) which is very large as compared to the strain in the concrete. After the application of external moment there will be a compressive strain in the top fiber (ϵ_o), a tensile strain in the reinforcing steel and concrete at the level of reinforcing steel (ϵ_s) and tensile strain in concrete adjacent to prestressing tendon (ϵ_{cp}). The change in strain ($\Delta\epsilon$) in concrete will be the sum of ϵ_{ce} and ϵ_{cp} . And the total strain in the prestressing wire at point of failure will be the sum of ϵ_{pe} , ϵ_{ce} and ϵ_{cp} . That is, if the strains are known it is easy to calculate the ultimate moment capacity.

$$\epsilon_p = \epsilon_{pe} + \epsilon_{ce} + \epsilon_{cp} = \epsilon_{pe} + \epsilon_{ce} + \epsilon_o \frac{d_p - d_n}{d_n}, \quad (2.11)$$

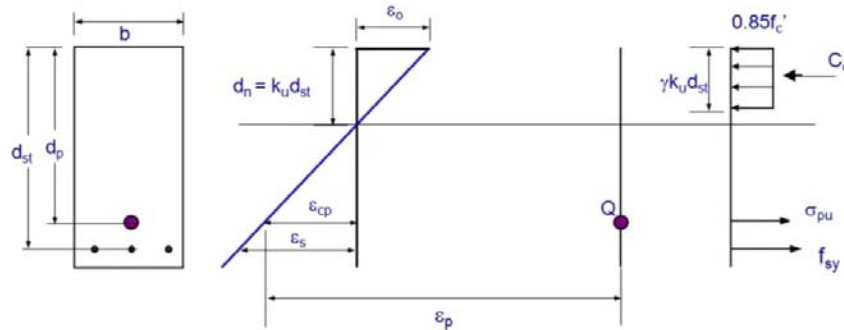


Figure 2.12: Stress strain diagram for a prestressed beam section

Taking moment about C_c :

$$M_u = \sigma_{pu} A_p \left(d_p - \frac{\gamma k_u d_{st}}{2} \right) + f_{sy} A_{st} \left(d_p - \frac{\gamma k_u d_{st}}{2} \right), \quad (2.12)$$

Where A_p = area of the prestressing wire

A_{st} = area of the reinforcing wires

σ_{pu} = stress at the prestressing wire at the ultimate condition

f_{sy} = stress at the reinforcing bars

There are three different ways to calculate ultimate moment capacity (M_u) of a prestressed concrete section.

- Approximate analysis for M_u , using yielding strength of prestressing steel.
 $\sigma_{pu} = f_{py}$
- Rigorous method using strain compatibility
- Semi-empirical expressions for σ_{pu}

2.5.2 DESIGN OF FLEXURE REINFORCEMENTS

In the assessment of the likely behavior of a prestressed concrete structure or element, the amount of flexural tensile stress allowed under service load defines its class as follows [12]:

- Class 1: no flexural tensile stresses;
- Class 2: flexural tensile stresses but no visible cracking;

- Class 3: flexural tensile stresses but surface width of cracks not exceeding 0.1 mm for members in very severe environments and not exceeding 0.2 mm for all other members.

In general, the design of class 1 and 2 members is controlled by the concrete tension limitations for service load conditions, but the design ultimate strength in flexure, shear and torsion should be checked. The design of class 3 members is usually controlled by ultimate limit state conditions or by deflection.

Figure 2.6 shows a simply supported beam carrying a uniform load. The mid span stress at top and bottom fibers of the beam at transfer and service are shown in the equations below.

At transfer:

$$\sigma_t = \frac{\alpha P_o}{A_c} - \frac{\alpha P_o e}{Z_t} + \frac{M_o}{Z_t} \quad (2.13 \text{ a})$$

$$\sigma_b = \frac{\alpha P_o}{A_c} + \frac{\alpha P_o e}{Z_b} - \frac{M_o}{Z_b} \quad (2.13 \text{ b})$$

At service:

$$\sigma_{t,service} = \frac{\beta P_o}{A_c} - \frac{\beta P_o e}{Z_t} + \frac{M_{serv}}{Z_t} \quad (2.13 \text{ c})$$

$$\sigma_{b,service} = \frac{\beta P_o}{A_c} + \frac{\beta P_o e}{Z_b} - \frac{M_{serv}}{Z_b} \quad (2.13 \text{ d})$$

The maximum allowable compressive stress in concrete are f_{max} , $(f_{max})_{serv}$ at transfer and service. The minimum stress at transfer and service is f_{min} . If f_{min} is negative it ought to indicate a permissible tensile stresses. Then the equations can be written as inequalities:

$$\frac{\alpha P_o}{A_c} - \frac{\alpha P_o e}{Z_t} + \frac{M_o}{Z_t} \geq f'_{min} \quad (2.14 \text{ a})$$

$$\frac{\alpha P_o}{A_c} + \frac{\alpha P_o e}{Z_b} - \frac{M_o}{Z_b} \leq f'_{max} \quad (2.14 \text{ b})$$

$$\frac{\beta P_o}{A_c} - \frac{\beta P_o e}{Z_t} + \frac{M_s}{Z_t} \leq (f_{max})_{serv} \quad (2.14 \text{ c})$$

$$\frac{\beta P_o}{A_c} + \frac{\beta P_o e}{Z_b} - \frac{M_s}{Z_b} \geq f_{min} \quad (2.14 \text{ d})$$

By combining the inequalities (2.14 a) and (2.14 c) the expression for Z_t can be derived as:

$$Z_t \geq \frac{\alpha M_s - \beta M_t}{\alpha (f_{max})_s - \beta f_{min}} \quad (2.15)$$

And combining inequalities (2.14 b) and (2.14 d) the expression for Z_b can be derived as:

$$Z_b \geq \frac{\alpha M_s - \beta M_t}{\beta f'_{max} - \alpha f_{min}} \quad (2.16)$$

The design process is to find the prestress force based on the maximum eccentricity determined from section properties. After rearranging the inequalities:

$$P_o \geq \frac{(Z_t f'_{min} - M_t)}{\alpha \left(\frac{Z_t}{A_c} - e \right)} \quad (2.17 a)$$

$$P_o \leq \frac{(Z_b f'_{max} + M_t)}{\alpha \left(\frac{Z_b}{A_c} + e \right)} \quad (2.17 b)$$

$$P_o \leq \frac{(Z_t (f_{max})_{serv} - M_s)}{\beta \left(\frac{Z_t}{A_c} - e \right)} \quad (2.17 c)$$

$$P_o \geq \frac{Z_b f_{min} - M_s}{\left(\beta \left(\frac{Z_b}{A_c} - e \right) \right)} \quad (2.17 d)$$

The above inequalities have two upper bounds and two lower bounds to the value of the prestress force. In general the minimum value is required since the cost of the prestressing steel is a significant proportion of the total cost of the prestressed concrete structure.

The Magnel diagram has been introduced Belgian engineer Magnel. The relationship between $1/P_o$ and e are linear and if plotted graphically, they offer very useful meanings to find appropriate values of P_o and e . The Magnel diagram is designed with $1/P_o$ and not with P_o . The inequalities for prestress force are just rearranged. Nevertheless the value e may not be such a range, since the innermost of the bounds could overlap. In this case another value of e must be chosen and the limits for P_o found again, the process being repeated until satisfactory combination of P_o and e is found. The inequalities can be represented in the following form [5].

$$\frac{1}{P_o} \leq \frac{\alpha \left(\frac{Z_t}{A_c} - e \right)}{(Z_t f'_{min} - M_t)} \quad (2.18 a)$$

$$\frac{1}{P_o} \geq \frac{\alpha \left(\frac{Z_b}{A_c} + e \right)}{(Z_b f'_{max} + M_t)} \quad (2.18 b)$$

$$\frac{1}{P_o} \leq \frac{\beta \left(\frac{Z_b - e}{A_c} \right)}{(Z_b f_{min} - M_s)} \quad (2.18 \text{ c})$$

$$\frac{1}{P_o} \geq \frac{\beta \left(\frac{Z_t - e}{A_c} \right)}{(Z_t (f_{max})_s - M_s)} \quad (2.18 \text{ d})$$

Following the choices of the value of the prestress force, the limit of the eccentricity e now may be found. At this step the term cable is used to indicate the resultant of all the individual tendons. At the same time as the cable lies in the zone the stresses at the different loading stages will not exceed the allowed values, even though some tendons physically outside the cable zone.

$$e \leq \frac{Z_t}{A_c} + \frac{1}{P_o} (M_o - Z_t f'_{min}) \quad (2.19 \text{ a})$$

$$e \leq \frac{1}{\alpha P_o} (M_o + Z_b f'_{max}) - \frac{Z_b}{A_c} \quad (2.19 \text{ b})$$

$$e \geq \frac{Z_t}{A_c} + \frac{1}{\beta P_o} (M_{serv} - Z_t (f_{max})_{serv}) \quad (2.19 \text{ c})$$

$$e \geq \frac{1}{\beta P_o} (M_{serv} - Z_t (f_{max})_{serv}) - \frac{Z_b}{A_c} \quad (2.19 \text{ d})$$

2.5.2.1 DESIGN PROCEDURE FOR RECTANGULAR BEAM

The procedure for the design of rectangular beam with bonded tendons is as follows [10]:

- Calculate the ratio of the stress in the prestressing tendon after all losses to the characteristic strength of the tendons, f_{pe}/f_{pk}
- Calculate $K=M/bd^2f_{ck}$, where f_{ck} is the concrete cylinder strength.
- Determine the limiting value of x/d from Table A1 of Annex A for appropriate value of f_{ck} and percentage of redistribution (generally zero for prestressed concrete members). Hence determine K_{lim} corresponding to the appropriate value of f_{pe}/f_{pk} from Figure A1 of Annex A.
- If $K=K_{lim}$, determine $A_p f_{pk}/bdf_{ck}$ corresponding to appropriate value of f_{pe}/f_{pk} from Figure A2 for Annex A
- Hence determine the area of prestressing tendons required, A_p .
- If this is less than the area being provided for serviceability the section is satisfactory at the ultimate limit state. Otherwise ordinary reinforcement must be added in the tension zone of the section. The area of ordinary reinforcement, A_s can be taken into account by replacing it with an equivalent are of prestressing tendons $A_s f_{yk}/f_{pk}$.

- If $K > K_{lim}$ then compression reinforcement is required. The area of ordinary compression reinforcement, A'_s , is calculated from:

$$A'_s = \frac{M - K_{lim} b d^2 f_{ck}}{0.87 f_{yk} (d - d')} \quad (2.20)$$

Where d' is the depth to the center of the compression reinforcement from the compression face.

If $d' > \left(1 - \frac{f_{ck}}{800}\right) x$, use $700 \left(1 - \frac{d'}{x}\right)$ in lieu of $0.87 f_{yk}$

- Determine A_p corresponding to K_{lim} from Figure A of Annex A and z/d corresponding to the limiting value x/d from Figure A1 of Annex A. the required area of prestressing tendon in the tension zone is given by:

$$A_p \left(1 + \frac{0.87 A'_s f_{yk} z}{K_{lim} b d^2 f_{ck}}\right) \quad (2.21)$$

- If this is less than the area being provided for serviceability the section is satisfactory at the ultimate limit state. Otherwise ordinary reinforcement must be added in the tension zone of the section. The area of ordinary reinforcement, A_s , can be taken in to account by replacing it with n equivalent area of prestressing tendons $A_s f_{yk} / f_{pk}$.

2.5.2.2 DESIGN PROCEDURE FOR FLANGED BEAM

The procedure for the design of flanged beam with bonded tendons is as follows [10]:

- Calculate the ratio of the stress in the prestressing tendon after all losses to the characteristic strength of the tendon, f_{pe} / f_{pk} .
- Check the position of the neural axis by determining $K = M / b d^2 f_{ck}$ using the flange width b , where f_{ck} is the cylinder strength, and selecting x/d from figure A1 of annex A. check that x/d is less than the limiting value obtained from table A1 of annex A, otherwise redesign the section and calculate x .
- If $0.8x \leq h_f$, the depth of the flange, then A_p is determined as for a rectangular beam of breadth b .
- If $0.8x > h_f$, the stress block lies outside the flange. Calculate the resistance moment of the flange, M_{uf} , from

$$M_{uf} = 0.567 f_{ck} (b - b_w) h_f (d - 0.5 h_f) \quad (2.22)$$

- Calculate

$$K_w = \frac{(M - M_{uf})}{f_{ck} b_w d^2} \quad (2.23)$$

If $K_w \leq K_{lim}$, obtained as a rectangular beam of width b_w , then determine z/d from Figure A1 of Annex A and $A_p f_{pk} / b d f_{ck}$ from figure A2 of Annex A corresponding to K_w and f_{pe} / f_{pk} ; otherwise redesign the section

- Calculate the stress in the prestressing tendon, f_p at the ultimate limit state from

$$f_p = \frac{K_w}{\left(\frac{z}{d}\right)} \left(\frac{bd f_{ck}}{A_p f_{pk}}\right) f_{pk} \quad (2.24)$$

And determine A_p from;

$$A_p = \frac{M_{uf}}{f_p(d-0.5h_f)} + \frac{(M-M_{uf})}{f_p z} \quad (2.25)$$

- If this is less than the area being provided for serviceability the section is satisfactory at the ultimate limit state. Otherwise ordinary reinforcement must be added in the tension zone of the section. The area of ordinary reinforcement, A_s can be taken into account replacing it with an equivalent area of prestressing tendon $A_s f_{yk}/f_{pk}$.

2.5.3 ANALYSIS AND DESIGN FOR SHEAR

It may be state that prestressed concrete beams possess greater reliability in shear resistance than reinforced concrete beams, because prestressing will usually prevent the occurrence of shrinkage cracks which could conceivably destroy the shear resistance of the reinforced concrete beams, especially near the point of contra-flexure [32].

The shear distribution in un-cracked structural concrete member for which the deformation is assumed to be linear is a function of the shear force and the properties of the cross section of the member. The shear stress at a point is expressed as [23],

$$\tau_v = \left(\frac{VS}{Ib}\right), \quad (2.26)$$

Where τ_v = shearing stress due to transverse loads

V= shearing force

S= statical moment (first moment of area)

I= second moment of area of section about its centroid

b = breadth of section t the given point

The effect of this shear stress is to induce principal tensile stresses on diagonal planes. The strength of concrete subjected to pure shear being nearly twice that in tension, local failures first appear in the form of diagonal tension cracks on region of high shear stresses. In prestressed concrete members, the shear stress is generally accompanied by a direct stress in the axial direction of the member, and if transverse, vertical prestressing is adopted, the compressive stresses in the direction perpendicular to the axis of the

member will be present in addition to the axial prestress [23]. The most general case of an element subjected to a two-dimensional stress system is shown in Figure 2.13.

The maximum and minimum principal stresses developed are given by;

$$f_{\max/\min} = \left[\left(\frac{f_x + f_y}{2} \right) \pm \frac{1}{2} \sqrt{(f_x - f_y)^2 + 4\tau_v^2} \right], \quad (2.27)$$

Where f_x and f_y are the direct stresses and τ_v , is the shear stress acting at the point.

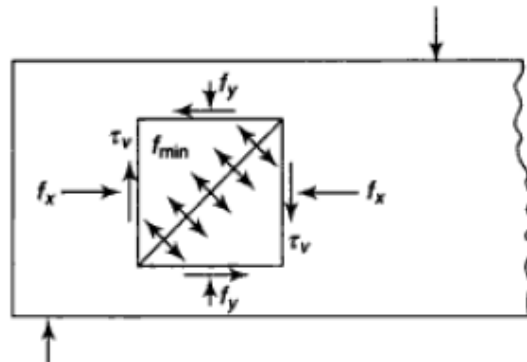


Figure 2.13: principal tensile stress on a prestressed member [23]

In prestressed concrete members, the direct stresses f_x and f_y being compressive, the magnitude of the principal tensile stress is considerably reduced, and in some cases even eliminated, so that under working loads, both major and minor principal stresses are compressive, thereby eliminating the risk of diagonal tension cracks in concrete. In general there are three ways of improving shear resistance of structural concrete members by prestressing techniques:

- Horizontal or axial prestressing
- Prestressing by inclined or sloping cables; and
- Vertical or transverse prestressing.

By transverse prestressing, the principal tension is completely eliminated resulting in a compressive state of stress at the support. Vertical prestressing is not generally adopted because the length of the cables being short, the loss of prestress due to anchorage slip is excessively large. Post-tensioning is generally un-economical for vertical prestressing due to the large number of anchorages required and the losses of prestress encountered. A viable alternative is to see pre-tensioned vertical wires closely spaced to achieve the desired prestress [23].

2.5.4 DESIGN OF SHEAR REINFORCEMENTS

V_{sd} is the ultimate shear force. And for the design of the shear reinforcement, V_{sd} may be taken as that acting at a distance d from the face of support. At any section, V_{sd} should not exceed the maximum shear capacity, $V_{Rd2,red}$, given by [31]:

$$V_{Rd2,red} = 1.67 V_{Rd2} \left(\frac{1-1.5\sigma_{cp,eff}}{f_{ck}} \right) \leq V_{Rd2} \quad (2.28)$$

Where:

$$V_{Rd2} = 0.15 f_{ck} b_{w,nom} d$$

$$b_{w,nom} = b_w - 0.5 \sum \emptyset \text{ (for grouted ducts) or } b_w - \sum \emptyset \text{ (for non-grouted ducts)}$$

b_w is the width of the beam web

\emptyset is the diameter of grouted ducts which may be taken as zero when $\emptyset = b_w/8$, or the diameter of non-grouted ducts.

$$\sigma_{cp,eff} = 1.2 P_o / A_c$$

Where V_{sd} exceeds V_{Rd1} , the shear capacity of the section without shear reinforcement, shear reinforcement will be required. V_{Rd1} is calculated from [31]:

$$V_{Rd1} = v_{Rd1} b_w d \quad (2.29)$$

Where: v_{Rd1} is obtained from figure A3 in annex A

Standard method for providing shear reinforcement in the form of vertical links should be provided in accordance with [10]:

$$A_{sw} = \frac{1.28 s (V_{sd} - V_{Rd1})}{f_{yk} d} \quad (2.30)$$

Where:

A_{sw} is the total cross section of the links

s is the longitudinal spacing of the links and

f_{yk} is the characteristic strength of the link

Minimum reinforcement requirement: $A_{sw} = \rho_w s b_w$ where;

ρ_w is obtained from Table A3 in annex A

s is the longitudinal spacing of the links and

b_w is the width of web

The maximum spacing of shear reinforcement is given in Table A4 in Annex A

2.5.5 PRESTRESSED CONCRETE MEMBERS IN TORSION

In case of structural concrete members subject to torsion, shear stresses develop depending upon the type of cross section and the magnitude of the torque. The shear stresses in association with the flexural stresses may give rise to principal tensile stress, the value of which when exceeds the tensile strength of the concrete results in the development of cracks on the surface of the member. The distribution of torsional shear-stress is uniform in circular section where the magnitude of the shear stress is proportional to the distance from the center. In the case of non-circular sections involving warping of the cross section, approximate formulae have been proposed based on the elastic analysis, due to St.Venant and Bach, to estimate the maximum torsional shear stress for un-cracked elements [23].

2.5.5.1 PURE TORSION

The failure of prestressed concrete member without additional un-tensioned reinforcement, under pure torsion, is more or less similar to that of plain concrete where sudden failure is imminent almost simultaneously with the formation of the first crack. For members subjected to pure torsion, concentric prestress is more advantageous than eccentric prestress. The use of longitudinal steel or spirals independent of each other does not increase the ultimate torsional resistance. But when longitudinal steel and spirals are provided in prestressed members, the ultimate torsional resistance is enhanced and according to Zia can be expressed as, $T_t = T_{tp} + T_{ts}$. Where T_{tp} is the torsional resistance moment of the prestressed concrete section and T_{ts} is the additional torsional resistance moment of the non-prestressed reinforcement, which must consist of transverse and longitudinal steel.

2.5.5.2 COMBINED BENDING MOMENT AND TORSION

Prestressed concrete members under combined bending moment and torsion exhibit a progressive failure pattern with extensive cracking. The interaction curves of combined bending and torsion for members is somewhat similar to that of reinforced concrete members. Based on the analysis of several experimental investigations, Ananthanarayana et al. have proposed circular interaction curves for concentric and eccentrically prestressed sections involving various parameters, such as pure torsional strength and flexural strength of the members.

2.5.5.3 COMBINED BENDING MOMENT, SHEAR AND TORSION

In most practical situations, prestressed members are subjected to torsion and bending together with transverse shear forces. Investigations by Gausel have indicated a circular interaction diagram between that moment causing flexural shear and torque, both expressed in a non-dimensional form against their individual capacities. Based on experimental investigations, Bishara has suggested parabolic interaction curves in non-dimensional form relating bending and twisting moments as well as shear and torque. The behavior of a prestressed concrete member is affected by the relative magnitude of the internal actions, such as torque, bending moment and shear force, in critical regions. If torsion is small, it has little effect on the overall behavior and the failures are controlled by either flexure or shear. Members subjected to torque, bending and shear are generally reinforced with longitudinal and transversal reinforcement. In order to study the contribution of the longitudinal and transverse reinforcement in resisting flexure, torsion and shear forces, it becomes necessary to analyze the system of forces acting on the warped cross-sections of the structural element at the limit state of failure.

2.5.6 DESIGN OF REINFORCEMENTS FOR TORSION SHEAR AND BENDING

The design procedure according to Euro code can be summarized as follows:

- The shear stress in a wall of a section subject to a pure torsional moment may be calculated from:

$$\tau_{t,i} t_{ef,i} = \frac{T_{Ed}}{2A_k} \quad (2.31)$$

The shear force $V_{Ed,i}$ in a wall i due to torsion is given by:

$$V_{Ed,i} = \tau_{t,i} t_{ef,i} z_i \quad (2.32)$$

Where

T_{Ed} applied design torsion (see Figure 6.11 Euro-code 2-1992-1-1, 2004)

A_k area enclosed by the centre-lines of the connecting walls, including inner hollow areas.

$\tau_{t,i}$ torsional shear stress in wall i

$t_{ef,i}$ is the effective wall thickness. It may be taken as A/u , but should not be taken as less than twice the distance between edge and centre of the longitudinal reinforcement. For hollow sections the real thickness is an upper limit

A is the total area of the cross-section within the outer circumference, including inner hollow areas

U is the outer circumference of the cross-section

z_i is the side length of wall i defined by the distance between the intersection points with the adjacent walls

- The required transverse reinforcement for the effects of torsion (see Equation 2.32) and shear for both hollow and solid members may be superimposed, assuming the same value for the strut inclination θ . The limits for θ given in 6.2.3 (2) (Euro-code 2-1992-1-1, 2004) are also fully applicable for the case of combined shear and torsion. The maximum bearing capacity of a member loaded in shear and torsion follows from 6.3.2 (4) (Euro-code 2-1992-1-1, 2004).
- The required cross-sectional area of the longitudinal reinforcement for torsion ΣA_{si} may be calculated from Equation 2.33

$$\frac{\Sigma A_{si} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot\theta \quad (2.33)$$

Where; u_k is the perimeter of the area A_k

f_{yd} is the design yield stress of the longitudinal reinforcement A_{si}

θ is the angle of compression struts (see Figure 6.5 of Euro-code 2-1992-1-1, 2004).

In compressive chords, the longitudinal reinforcement may be reduced in proportion to the available compressive force. In tensile chords the longitudinal reinforcement for torsion should be added to the other reinforcement. The longitudinal reinforcement should generally be distributed over the length of side, z_i , but for smaller sections it may be concentrated at the ends of this length.

- The maximum resistance of a member subjected to torsion and shear is limited by the capacity of the concrete struts. In order not to exceed this resistance the following condition should be satisfied:

For solid cross-sections:

$$T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max} \leq 1,0 \quad (2.34)$$

Where: T_{Ed} is the design torsional moment

V_{Ed} is the design transverse force

$T_{Rd,max}$ is the design torsional resistance moment according to

$$T_{Rd,max} = 2v \alpha \sin\theta \cos\theta T_{Rd,max} = c f_{cd} A_k t_{ef,i}$$

Where v follows from Expression 6.6 (Euro-code 2-1992-1-1, 2004) and αc from Expression 6.9 (Euro-code 2-1992-1-1, 2004)

$VR_{d,max}$ is the maximum design shear resistance according to Expressions (6.9) or (6.14) (Euro-code 2-1992-1-1, 2004). In solid cross sections the full width of the web may be used to determine $VR_{d,max}$

- For approximately rectangular solid sections only minimum reinforcement is required (see 9.2.1.1 Euro-code 2-1992-1-1, 2004) provided that both the following conditions are satisfied:

$$T_{Ed} / T_{Rd,c} + V_{Ed} / VR_{d,c} \leq 1,0 \quad (2.35)$$

Where: $T_{Rd,c}$ is the torsional cracking moment, which can be determined by setting $\tau_{t,i} = f_{ctd}$
 $VR_{d,c}$ follows from Expression 6.2 (Euro-code 2-1992-1-1, 2004)

2.5.7 ANALYSIS AND DESIGN REQUIREMENTS FOR DEFLECTION

The deflections of prestressed concrete members are influenced by the following salient factors:

- Imposed loads and self weight
- Magnitude of prestressing force
- Cable profile
- Second moment of inertia of cross section
- Modulus of elasticity of concrete
- Shrinkage, creep and relaxation of steel stress
- Span of the member
- Fixity condition

In pre-cracking stage, the whole cross section is effective and the deflections in this stage are computed by using the second moment of area of the gross concrete section. The computation of short term or instantaneous deflections, which occur immediately after transfer of prestress and on application of loads, is conveniently done by using Mohr's theorems. In post cracking stage, a prestressed concrete beam behaves in a manner similar to that of a reinforced concrete beam and the computation of deflections in this stage is made by considering moment curvature relationship which involves the section properties of cracked beam. In both cases, the effect of creep and shrinkage of concrete is to increase the long term

deflections under sustained loads, which is estimated by using empirical methods that involve the use of effective (long term) modulus of elasticity or by multiplying short-term deflections by a suitable factor.

2.5.7.1 DESIGN REQUIREMENTS OF DEFLECTION

It is general practice in the most of the codes to safeguard against excessive deflections under serviceability limit states, either indirectly by prescribing a minimum span to depth ratio for the member, or directly by specifying a maximum permissible deflection expressed as fraction of the span.

- The British code (BS: 8110-1985) specifies a maximum deflection limit of $\text{span}/250$, beyond which the sag in member will usually become noticeable. To prevent damage of non-structural elements, the code recommends that the deflection after installation of finishes and partitions should not exceed the following values:
 - $\text{Span}/500$ or 20mm, whichever is less. For brittle materials
 - $\text{Span}/350$ or 20mm, whichever is less, for non-brittle partitions or finishes.
- According to Euro code 2-1992-1-1, 2004, the appearance and general utility of the structure may be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds $\text{span}/250$. The sag is assessed relative to the supports. Pre-camber may be used to compensate for some or all of the deflection but any upward deflection incorporated in the formwork should not generally exceed $\text{span}/250$.

2.6. LOSSES IN PRESTRESSING TENDONS

The initial tensile force in prestressing tendon undergoes a gradual reduction with time from the instant when the steel is first tensioned due to various causes. These are referred to as losses of prestress. Several factors contribute to the loss of prestress and can be broadly divided into the following two main categories.

- Immediate losses
- Differed losses

2.6.1 IMMEDIATE LOSSES

Immediate losses take place prior to or during the transfer of prestressing force to the concrete. They can be due to elastic shortening losses, friction losses and anchorage slip losses.

2.6.1.1 ELASTIC SHORTENING LOSSES

In pretensioned member, the tensile strain in the prestressing steel decreases by an amount equal to the compressive strain induced in the concrete at the level of steel, due to the transfer of prestressing force. If σ_{ci} is the concrete compressive stress at the steel level just after the transfer, the loss in stress in prestressing steel due to elastic shortening of concrete is given by;

$$\Delta\sigma_p = \sigma_{ci} \frac{E_p}{E_c}, \quad (2.36)$$

Where: E_p = young modulus of the tendon

E_c = young modulus of concrete

The stress in the prestressing steel due to elastic shortening is given by [15]:

$$\sigma_{ci} = \frac{f_{po}}{m + \frac{A_c}{A_p(1 + \frac{e^2}{r^2})}} - \frac{M_o e}{I_c}, \quad (2.37)$$

Where:

f_{po} is P_o/A_p

m is the modular ratio E_s/E_{cm} (where E_{cm} is based on the long term value of concrete strength)

r is $(I_o/A_c)^{0.5}$

e is the eccentricity of the tendon.

The prestress force loss is then computed as; $\Delta P_{el} = m * \sigma_{ci} * A_p$

2.6.1.2 FRICTION LOSSES

In post tensioned members, friction between the cable and the ducts results in losses of prestress. The friction loss is mainly caused due to:

- Curvature effect caused by the curved profile of the cable along the length of the beam
- Wobble effect caused by wave like formation as cable and ducts cannot be perfectly aligned to follow a predetermined profile throughout the length of the beam

2.6.1.2 ANCHORAGE SLIP LOSSES

In post-tensioned members, when the cable is tensioned and the jack is released to transfer prestress to concrete the friction wedge employed to grip the wires slip over a small distance before the wires are firmly housed causing anchorage slip loss.

2.6.2 DIFFERED LOSSES

With time the prestressing force in tendons decreases due to stress relation in the prestressing steel, creep and shrinkage of concrete.

2.6.2.1 STRESS RELAXATION

Gradual decrease in stress in the steel when constant strain is applied over a period of time. This phenomenon can be removed completely, but can be limited by re-stressing the tendons 3-5 times. The delay will reduce and become almost negligible after few prestressed operations.

According to Euro code 2 EN 1991-1-1, the relaxation loss may be obtained from the manufacturers test certificates.

2.6.2.2 LOSSES DUE TO SHRINKAGE

Drying of concrete accompanied by reduction in volume is referred to as drying shrinkage. This reduction in volume reduces the length of the element and reduces the magnitude of prestressing force.

$$\Delta\sigma_p = -E_p\Delta\epsilon_r, \quad (2.38)$$

$$\Delta P_p = -A_p E_p \Delta\epsilon_r, \quad (2.39)$$

Where ϵ_r = Loss in prestress in steel due to shrinkage strain

When the reinforcement is distributed throughout the member, it reduces shrinkage. Hence the design shrinkage strain ϵ_r is reduced by dividing by $(1+15A_s/A_g)$. That is,

$$\Delta\sigma_p = -E_p \left(\frac{\Delta\epsilon_r}{1+15\frac{A_s}{A_g}} \right), \quad (2.40)$$

Use of high strength concrete with low water to cement ratio results in a reduction in drying shrinkage and consequently reduction in loss of prestress, but use of ultra -high strength concrete can lead to autogenous shrinkage. In case of pre-tensioned members, the total drying shrinkage strain will be large

after the transfer of prestress in comparison to post-tensioned members, where a portion of shrinkage will have already have taken place by the time of transfer of stress.

$$\epsilon_r = \epsilon_{cd,\infty} * \frac{j}{j+0.04*h_o^{1.5}} + \epsilon_{ca,\infty} * 1 - \exp(-0.2 * \sqrt{j}) \quad (2.41)$$

Where:

- ϵ_{cd} is the dry and shrinkage strain,
- $\epsilon_{ca,\infty}$ is the dry and shrinkage strain, $\epsilon_{ca,\infty} = 2.5 * (f_{ck} - 10) * 10^{-4}$
- $j = \text{days}$
- h_o is a notational size (mm) of the cross section, $h_o = \frac{2A_c}{U}$
- U is the perimeter of that part of the cross section which is exposed to drying

2.6.2.3 LOSSES DUE TO CREEP

If concrete stress is maintained over a period of time, the strain increases and the concrete element shortens, this phenomenon is called creep. Creep in concrete reduces the stress in high strength steel.

$$\Delta\sigma_p = E_p \epsilon_{cc}, \quad (2.42)$$

According to euro code 2- part 1-1, 1992, $\epsilon_{cc} = \text{creep coefficient}/E_c$ *stress in the concrete at transfer at the level of the tendon.

Where; $\epsilon_{cc} = \text{creep strain}$

The loss of force in the tendon due to creep is calculated by multiplying together the strain due to creep effect, the elastic modulus of the tendon and the total area of the tendon.

The creep coefficient, $\varphi(t,t_0)$ is related to E_c , the tangent modulus, which may be taken as $1.05 E_{cm}$. Where great accuracy is not required, the value found from Figure 3.1 of Euro code 2 EN 1991-1-1 may be considered as the creep coefficient, provided that the concrete is not subjected to a compressive stress greater than $0.45f_{ck}(t_0)$ at an age t_0 , the age of concrete at the time of loading. The creep deformation of concrete $\epsilon_{cc}(\infty,t_0)$ at time $t = \infty$ for a constant compressive stress σ_c applied at the concrete age t_0 , is given by[7];

$$\epsilon_{cc}(\infty, t_0) = \varphi(\infty, t_0) * \left(\frac{\sigma_c}{E_c}\right) \quad (2.43)$$

When the compressive stress of concrete at an age t_o exceeds the value $0.45 f_{ck}(t_o)$ then creep non-linearity should be considered. Such a high stress can occur as a result of pretensioning, e.g. in precast concrete members at tendon level. In such cases the non-linearity notional creep coefficient should be obtained as follows [12];

$$\varphi_{ni}(\infty, t_o) = \varphi(\infty, t_o) \exp (1.5(K_\sigma - 0.45)) \quad (2.44)$$

Where:

$\varphi_{ni}(\infty, t_o)$ is the non-linear notional creep coefficient, which replaces $\varphi(\infty, t_o)$,

K_σ is the stress-strength ratio $\sigma_c/f_{ck}(t_o)$, where σ_c is the compressive stress and $f_{ck}(t_o)$ is the characteristic compressive strength of at the time of loading.

2.6.3 TOTAL LOSSES

The total prestress losses are defined by the initial prestress force applied to a member P_o and by the effective prestress force at transfer αP_o and at design load βP_o . The value α indicates the short-term losses due to elastic shortening, anchorage draw-in and friction, whilst the value β indicates the long-term losses due to concrete creep and shrinkage and steel relaxation. During the initial design stage it is possible to approximate the prestress loss, which it will be refined later in the design process when more details of the prestressing steel are available [5].

The Percentage of the initial stress which is accounted in design computation is summarizes in Table 2.2 below [32].

Table 2.1: Total losses in prestress

TYPE OF LOSSES	PERCENTAGE LOSS OF STRESS	
	PRE-TENSIONED	POST-TENSIONED
Elastic shortening	4	1
Creep of concrete	6	5
Shrinkage of concrete	7	6
Stress relaxation	8	8
Total	25	20

CHAPTER 3

ANALYSIS AND DESIGN OF THE HOUSING PROJECTS USING CONVENTIONAL REINFORCED CONCRETE

3.1 INTRODUCTION

A representative architectural design that is currently adopted for the 40/60 housing project is selected in this research to compare the economic and design aspect of conventional reinforced concrete and precast prestressed concrete. The architectural design of this housing project is not the best fit for using it for precast prestressed system. The major reason for this is the structure represents a divided H shape, which makes the structure irregular in plan. To avoid this plan irregularity, the design for this project was made using two expansion joints dividing the building into three parts. To change the three parts of this reinforced concrete design directly to precast prestressed concrete will lead to undue comparison between these two systems. Since the current architectural design of this housing project is not suitable for precast prestressed systems architectural adjustment become mandatory. This adjustment basically was done to change the H shape in to rectangular shape by providing supporting members. The initial and modified typical floors drawing of this research is provided in Annex c drawing 1/5-and drawing 3/5 respectively. Therefore, for comparison purpose, design of reinforced concrete used for this modified architectural layout should be carried out first. For the analysis and design of this structure ETABS version 9.7.3 along with excel 2007 are used.

3.2 MATERIAL PROPERTY AND LOAD COMBINATIONS

The material properties were defined in accordance with Euro code and are listed in Tables 3.1 below. Consequently, the loads and load combinations are set by taking into account the self weight of the structure, imposed dead loads, live loads and earthquake loads. The load cases, seismic data and load combination used are summarized in Table 3.2, Table 3.3 and Table 3.4 respectively.

Table 3.1: Material properties for concrete and steel

Concrete grade	C-30	C-25
Material type	isotropic	isotropic
Cylindrical strength (f_{ck}) [MPa]	24	20
Modulus of elasticity [GPa]	32	29
Mass per volume	2.548	2.548
Weight per unit volume	25	25
Poisson's ratio	0.2	0.2
Coefficient of thermal expansion [α_c]	9.9×10^{-6}	9.9×10^{-6}
Shear modulus [GPa]	13.3	12.08
Steel grade	S-400	S-300
Material type	isotropic	isotropic
Minimum yield strength [MPa]	400	300
Modulus of elasticity [GPa]	200	200
Mass per volume [kg/m^3]	2.548	2.548
Weight per unit volume [kN/m^3]	25	25
Poisson's ratio	0.2	0.2
Coefficient of thermal expansion [α_c]	9.9×10^{-6}	9.9×10^{-6}
Shear modulus [GPa]		

Table 3.2: Load cases

Case	Type	Self Weight Multiplier	Auto Load
SELFW	DEAD	1	
LIVE	LIVE	0	
OTHERDL	DEAD	0	
EQXP	QUAKE	0	UBC94
EQXN	QUAKE	0	UBC94
EQYP	QUAKE	0	UBC94
EQYN	QUAKE	0	UBC94

Table 3.3: Seismic Data

Case	Dir	Eccentricity Ratio	Period Calculation	User T	Rw	Z	S	I
EQXP	X + EccY	0.05	User Defined	0.87	3.3	0.1	1.2	1.2
EQXN	X - EccY	0.05	User Defined	0.87	3.3	0.1	1.2	1.2
EQYP	Y + EccX	0.05	User Defined	0.87	3.3	0.1	1.2	1.2
EQYN	Y - EccX	0.05	User Defined	0.87	3.3	0.1	1.2	1.2

Table 3.4: Load combinations

Combinations	Type	Case	Factor	Case Type
COMB1	ADD	SELFW	1	Static
		OTHERDL	1	Static
		LIVE	1	Static
COMB2	ADD	SELFW	1.3	Static
		OTHERDL	1.3	Static
		LIVE	1.6	Static
COMB3	ADD	COMB2	0.75	Combo
		EQXP	1	Static
COMB4	ADD	COMB2	0.75	Combo
		EQXP	-1	Static
COMB5	ADD	COMB2	0.75	Combo
		EQXN	1	Static
COMB6	ADD	COMB2	0.75	Combo
		EQXN	-1	Static
COMB7	ADD	COMB2	0.75	Combo
		EQYP	1	Static
COMB8	ADD	COMB2	0.75	Combo
		EQYP	-1	Static
COMB9	ADD	COMB2	0.75	Combo
		EQYN	1	Static
COMB10	ADD	COMB2	0.75	Combo
		EQYN	-1	Static
ENVELOPE	ENVE	COMB1	1	Combo
		COMB2		Combo
		COMB3		Combo
		COMB4		Combo
		COMB5		Combo
		COMB6		Combo
		COMB7		Combo
		COMB8		Combo
		COMB9		Combo
		COMB10		Combo
ENVX	ENVE	COMB3	1	Combo
		COMB4		Combo
		COMB5		Combo
		COMB6		Combo
ENVY	ENVE	COMB7	1	Combo
		COMB8		Combo
		COMB9		Combo
		COMB10		Combo

3.3. MODELLING AND DESIGN ASPECTS

- The wall and frame are connected by pin connectors at every floor.
- Rigid floor diaphragms are assigned at every floor levels
- Live load reduction factors are considered as per EBCS-1. Which is determined as follows

$$\alpha_n = \frac{2+(n-2)\psi_0}{n}$$

Where: n is the number of storey (>2) above the loaded structural element from the same category
 ψ_0 is factor according to EBCS1,1995 Table 1.3

Table 3.5: Live load reduction factors

Number of Story's Supported	Reduction factor
3	0.9
4	0.85
5	0.82
6	0.8
7	0.78571
8	0.775
9	0.76667
10	0.76
11	0.75454
12	0.75
13	0.74615
14	0.74286
15	0.74

- Stiffness modification factors were adjusted according to table 3.5

Table 3.6: Stiffness modification factors

	Modification factor
Beams	0.35
Slabs	0.35
Columns	0.7
Cracked walls	0.35
Un-cracked walls	0.7

- Stability index

Table 3.7: Stability index values

Story	P [KN]	story shear VX [KN]	Story shear VY [KN]	Drift-X	Drift- Y	Stability index θ , X	Stability index θ , Y
TTB	9355.27	1430.92	1430.92	0.0065	0.0075	0.0424	0.0487
12TH	22995.98	2619.22	2619.21	0.0068	0.0075	0.0601	0.0659
11TH	36637.33	3722.62	3722.6	0.0074	0.0077	0.0727	0.0755
10TH	50278.04	4741.09	4741.07	0.0079	0.0078	0.0834	0.0825
9TH	63919.4	5674.74	5674.72	0.0083	0.0078	0.0933	0.0882
8TH	77560.75	6523.51	6523.49	0.0087	0.0078	0.1030	0.0931
7TH	91202.1	7287.41	7287.38	0.0090	0.0078	0.1122	0.0970
6TH	104843.45	7966.43	7966.4	0.0091	0.0076	0.1196	0.0994
5TH	118484.8	8560.57	8560.54	0.0090	0.0072	0.1242	0.0996
4TH	132125.51	9069.8	9069.77	0.0086	0.0067	0.1247	0.0972
3RD	145772.07	9493.77	9493.74	0.0077	0.0059	0.1176	0.0911
2ND	159296.41	9766.36	9766.33	0.0064	0.0050	0.1039	0.0817
1ST	173213.04	9976.38	9976.35	0.0050	0.0039	0.0875	0.0677
GR	189545.17	10140.03	10139.99	0.0035	0.0026	0.0653	0.0485
BSMT1	205435.56	10219.2	10219.17	0.0015	0.0010	0.0293	0.0205

- P- Δ analysis

Second- order effects (P- Δ effects) is checked according to conditions fulfilled in Euro code 8 EN 1998-1 sections 4.4.2.2 and EBCS 8 1995 section 2.4.2.2.

- Inter story drift

Inter storey drifts is checked according to conditions fulfilled in Euro code 8 EN 1998-1 sections 4.4.3.2 and EBCS 8 1995 section 2.4.3.2.

CHAPTER 4

ANALYSIS AND DESIGN OF THE HOUSING PROJECTS USING PRECAST PRESTRESSED CONCRETE UNITS

4.1 DIMENSIONING AND SETTING UP STRUCTURAL ARRANGEMENTS

The structural arrangement of the precast prestressed system is shown in drawing ST 4/5 and ST 5/5 in Annex D of this research. It is noticed that the column spacing are altered. With such arrangement the analysis and design of the precast prestressed units is carried out.

4.2 SETTING MATERIAL PROPERTY AND LOADING

The material properties were defined according to euro code and similar to chapter 3, the loads and load combinations are set by taking in to account the self weight of the structure, imposed dead loads, live loads and earthquake loads. The load cases, seismic data and load combination as used in chapter 3 and are summarized in Table 3.2, Table 3.3 and Table 3.4 respectively.

4.3 ASSUMPTIONS

- Structural analysis is performed on the basis of the nominal cross-section area of the prestressing steel and the characteristic values f_{pk} and ϵ_{uk} .
- The design value for the modulus of elasticity, E_p may be assumed equal to 205 GPa for the strands.
- The design value for the modulus of elasticity, E_p may be assumed equal to 200GPa for bars.
- The mean density of prestressing tendons for the purposes of design may normally be taken as 7850 kg/m^3 .
- According to the specification manual (provided in Annex C) of the strand mass of the strand is 1086.0g/m .

4.4 ANALYSIS AND DESIGN FOR THE SLAB PANELS

Ribbed slab type is selected for the design of the slabs. Here the rib sections are selected to be T-sections. The T-sections rest on the girder beam in such a manner as shown in Figure 4.1 below. For the initial design of the T-sections, minimum dimension are selected according to the Eurocode manual [10]. First according to the arranged structural layout (provided in Annex C drawing 3/5 and 4/5) of the building there exist four types of T-section ribs that vary in span. The initial dimension for the four types of the rib beams are listed in Table 4.1.



Figure 4.1: Layout of the T-section and the main girder beam [courtesy of white oak precast concrete semiconductor plant]

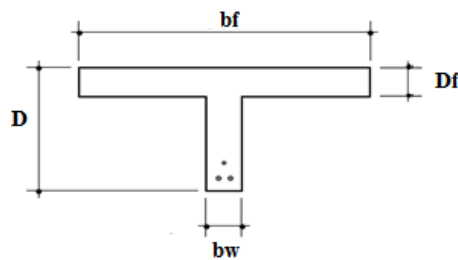


Table 4.1: T-sections dimensions for initial stage design

Type	bf [mm]	D [mm]	Df [mm]	bw [mm]	Length [m]
1	900	375	100	110	9.7
2	900	320	100	160	8.3
3	900	215	100	160	5.5
4	900	215	100	160	4.4

For initial design the material properties material properties is selected according to Eurocode manual [10] is listed on Table 4.2 below

Table 4.2: Material properties used for initial design

Type	Strength [Mpa]
Concrete	C40/50
reinforcement bar f_{pk}	460
links f_{pk}	250

▪ **Loadings**

The live load computed from the slab per 900mm rib spacing is 1.76kN/m and the other dead load (excluding self weight of the T-section) is 4.81kN/m. The Eurocode manual [10] specifies the minimum imposed loads to be:

- Loads due to floor finishes: 1.8kN/m²
- Loads due to ceiling and service loads: 0.5kN/m²
- Allowance for demountable light weight partitions: 1.0kN/m²
- Block partitions: 2.5kN/m²

Total loads = 5.8 kN/m²

Minimum total loads per rib (T-sections) to be used in design= 5.2kN/m

▪ **Design parameters for the prestressed concrete slab panel**

For transfer at 7 days, the compressive strength of concrete for C50 is;

$$f_{ck}^7 = 36 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable compressive concrete stress at transfer, f_{max}^7 , is;

$$f_{max}^7 = 18 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable tensile concrete stress at transfer, f_{min}^7 , is

$$f_{min}^7 = -3.5 \text{ N/mm}^2$$

For serviceability state, the compressive strength of concrete, f_{ck} , is

$$f_{ck} = 40 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable concrete stress allowed under service is,

$$(f_{max})_s = 24 \text{ N/mm}^2$$

Taking : $\alpha = 1 - 0.04 = 0.96$ and $\beta = 1 - 0.21 = 0.79$

$$\gamma = 24 \text{ kN/m}^3$$

▪ **Design for the prestressed concrete beams for Type -1**

The design for the T-section ribbed slab is done using the procedure explained in chapter 2, the calculation procedure for type 1 rib section is shown in this section. The rest of the design for the rib sections is provided in Annex B of this research.

The only applied loading at transfer is the self weight which is (density of concrete) × (area). Hence;

$$\text{self weight} = \gamma A_c = 24 \times 120.2 \times 10^3 \times 10^{-6} = 2.89 \text{ kN/m}$$

The maximum moment due to this loading is: transfer moment,

$$M_t = 2.89 (9.7)^2 / 8 = 34 \text{ kNm}$$

Uniformly distributed load due to self-weight of the beam and other dead loads per meter width, w_o

$$w_o = \gamma A_c + w_d = (24 \times 120.2 \times 10^3 \times 10^{-6}) + 5.2 = 8.1 \text{ kN/m}$$

The total loading at SLS is this plus the imposed loading, i.e.: SLS moment,

$$M_s = (8.1 + 1.76)(9.7)^2 / 8 = 116 \text{ kNm}$$

▪ **Elastic sectional moduli**

From inequalities 2.15 and 2.16 we can obtain the elastic section moduli. Required about the top and bottom fibers, Z_t and Z_b , as

$$Z_t \geq \frac{\alpha M_s - \beta M_t}{\alpha (f_{max})_s - \beta f_{min}} = \frac{0.96 * 116.00 * 10^6 - 0.79 * 34 * 10^6}{0.96 * 24.00 - 0.79 * (-3.5)} = \frac{84.5 * 10^6}{25.805} = 3.274 * 10^6 \text{ mm}^3 (< Z_t = 10.9253 * 10^6 \text{ mm}^3)$$

=>Ok!

$$Z_b \geq \frac{\alpha M_s - \beta M_t}{\beta f'_{max} - \alpha f_{min}} = \frac{0.96 * 116 * 10^6 - 0.79 * 34 * 10^6}{0.79 * 18 - 0.96 * (-3.5)} = \frac{84.5 * 10^6}{17.58} = 4.806 * 10^6 \text{ mm}^3 (< Z_b = 3.821 * 10^6 \text{ mm}^3)$$

=>Not ok!

The depth should be adjusted so as it can meet the required bottom section moduli. The depth is modified to 430mm and the corresponding geometric properties are listed below.

Table 4.3: Modified geometric property

Label	Type 1
Area [mm ²]*10 ³	126.3
Inertia[mm ⁴]*10 ⁶	1600.1
y _t [mm]	112
y _b [mm]	318
Z _t [mm ³]*10 ³	14313.2
Z _b [mm ³]*10 ³	5028.6

▪ **Modified design loads for the prestressed concrete beams**

The only applied loading at transfer is the self weight which is (density of concrete) × (area). Hence;

$$\text{self weight} = \gamma A_c = 24 \times 126.3 \times 10^3 \times 10^{-6} = 3.031 \text{ kN/m}$$

The maximum moment due to this loading is: transfer moment,

$$M_t = 3.031 (9.7)^2 / 8 = 35.65 \text{ kNm}$$

Uniformly distributed load due to self-weight of the beam and other dead loads per meter width, w_o

$$w_o = \gamma A_c + w_d = (24 \times 126.3 \times 10^3 \times 10^{-6}) + 5.2 = 8.231 \text{ kN/m}$$

The total loading at SLS is this plus the imposed loading, i.e.: SLS moment,

$$M_s = (8.231 + 1.76)(9.7)^2 / 8 = 117.507 \text{ kNm}$$

▪ **Elastic sectional moduli**

Required about the top and bottom fibers, Z_t and Z_b , as

$$Z_t \geq \frac{\alpha M_s - \beta M_t}{\alpha (f_{max})_s - \beta f_{min}} = \frac{0.96 * 117.507 * 10^6 - 0.79 * 35.65 * 10^6}{0.96 * 24.00 - 0.79 * (-3.5)} = \frac{84.643 * 10^6}{25.805} = 3.28 * 10^6 \text{ mm}^3 (< Z_t = 14.313 * 10^6 \text{ mm}^3)$$

=> Ok!

$$Z_b \geq \frac{\alpha M_s - \beta M_t}{\beta f'_{max} - \alpha f_{min}} = \frac{0.96 * 117.507 * 10^6 - 0.79 * 35.65 * 10^6}{0.79 * 18 - 0.96 * (-3.5)} = \frac{84.643 * 10^6}{17.58} = 4.815 * 10^6 \text{ mm}^3 (< Z_b = 5.0286 * 10^6 \text{ mm}^3)$$

=> Ok!

▪ **Determination of prestress forces and eccentricity**

From inequalities 2.18a, we get

$$\frac{1}{P_o} \leq \frac{\alpha \left(\frac{Z_t}{A_c} - e \right)}{(Z_t f'_{min} - M_t)} = \frac{0.96 \left(\frac{14.313 * 10^6}{126300} - e \right)}{(14.313 * 10^6 * (-3.5) - 35.65 * 10^6)} = \frac{0.96(113.325 - e)}{-85.7455 * 10^6} = \frac{(126.88 - 1.12e) * 10^{-8}}{-1} \frac{1}{N}$$

Note that the denominator is negative. Dividing both sides of n inequality by a negative number has effect of changing the sign of the inequality. Thus, the above inequality can be simplified to;

$$\frac{10^8}{P_o} \geq 1.12e - 126.88 \dots\dots (i)$$

From inequalities 2.18 b,

$$\frac{1}{P_o} \geq \frac{\alpha \left(\frac{Z_b}{A_c} + e \right)}{(Z_b f'_{max} + M_t)} = \frac{0.96 \left(\frac{5.0286 * 10^6}{126300} + e \right)}{(5.0286 * 10^6 * (18) + 35.65 * 10^6)} = \frac{0.96(39.8147 + e)}{126.1648 * 10^6} = \frac{(30.292 + 0.7609e) * 10^{-8}}{1} \frac{1}{N}$$

Thus, the above inequality can be simplified to;

$$\frac{10^8}{P_o} \geq 0.7609e + 30.292 \dots\dots (ii)$$

From inequalities 2.18 c,

$$\frac{1}{P_o} \geq \frac{\beta\left(\frac{Z_t}{A_c} - e\right)}{(Z_t(f_{max})_s - M_s)} = \frac{0.79\left(\frac{14.313 \cdot 10^6}{126300} - e\right)}{(14.313 \cdot 10^6 \cdot (24) - 117.507 \cdot 10^6)} = \frac{0.79(113.325 - e)}{226.005 \cdot 10^6} = \frac{(39.613 - 0.3495e) \cdot 10^{-8}}{1} \frac{1}{N}$$

Thus, the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq -0.3495e + 39.613 \dots\dots (iii)$$

From inequalities 2.18 d,

$$\frac{1}{P_o} \leq \frac{\beta\left(\frac{Z_b}{A_c} - e\right)}{(Z_b f_{min} - M_s)} = \frac{0.79\left(\frac{5.0286 \cdot 10^6}{126300} - e\right)}{(5.0286 \cdot 10^6 \cdot (-3.5) - 117.507 \cdot 10^6)} = \frac{0.79(39.8147 - e)}{-135.1071 \cdot 10^6} = \frac{(23.281 - 0.5847e) \cdot 10^{-8}}{-1} \frac{1}{N}$$

Note that the denominator is negative. Dividing both sides of n inequality by a negative number has effect of changing the sign of the inequality. Thus, the above inequality can be simplified to;

$$\frac{10^8}{P_o} \geq 0.5847e - 23.281 \dots\dots (iv)$$

Now putting all the four inequalities together:

$$\frac{10^8}{P_o} \geq 1.12e - 126.88 \dots\dots (i)$$

$$\frac{10^8}{P_o} \geq 0.7609e + 30.292 \dots\dots (ii)$$

$$\frac{10^8}{P_o} \geq -0.3495e + 39.613 \dots\dots (iii)$$

$$\frac{10^8}{P_o} \geq 0.5847e - 23.281 \dots\dots (iv)$$

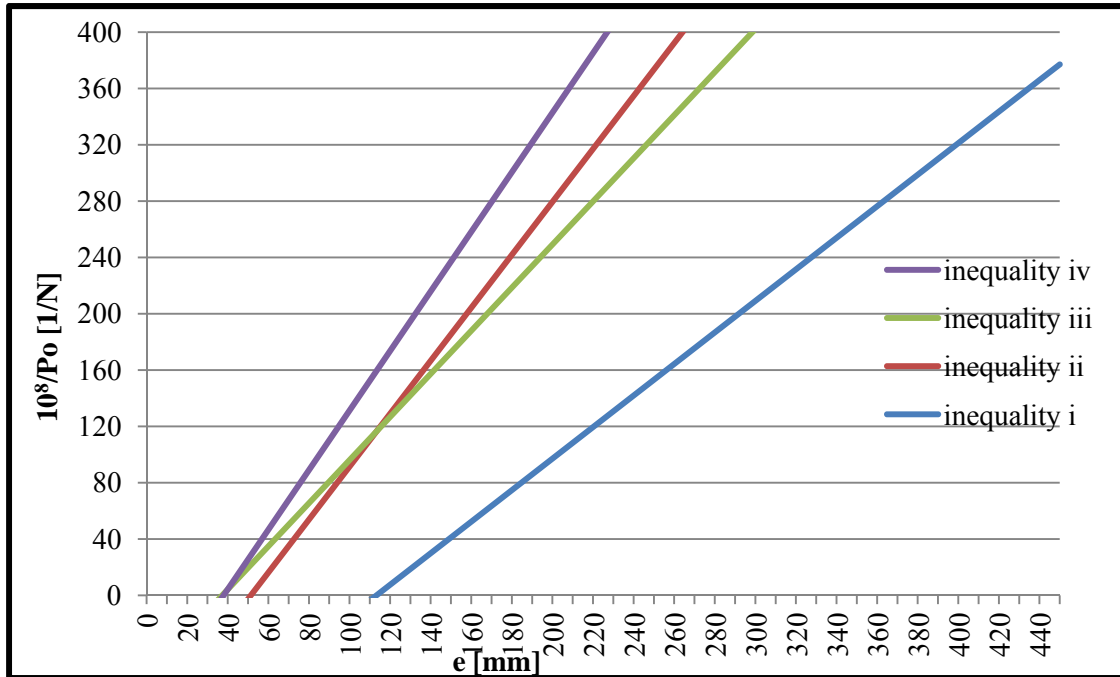


Figure 4.2: Magnel diagram for type 1

In this case, the distance from the neutral axis to the soffit is 318mm. If one logically consider the distance from the center of prestressing steel to the soffit as 80mm taking in to account the cover requirement, the maximum value of eccentricity for the permissible zone is;

$$e = 318 - 80 = 238\text{mm taking it to be } 240\text{mm}$$

The corresponding prestressing force will be obtained by;

$$\frac{10^8}{P_o} \geq 1.12e - 126.88 = 1.12 * 240 - 126.88 = 163.52 \text{ 1/N}$$

$$P_o \leq 10^8/163.52 = 611.546 \text{ KN}$$

$$\frac{10^8}{P_o} \geq 0.7609e + 30.292 = 0.7609 * 240 + 30.292 = 212.908 \text{ 1/N}$$

$$P_o \leq 10^8/212.908 = 469.686 \text{ KN}$$

$$\frac{10^8}{P_o} \geq -0.3495e + 39.613 = -0.3495 * 240 + 39.613 = -44.267 \text{ 1/N}$$

$$P_o \leq 10^8/-44.267 = -2259.02 \text{ KN}$$

$$\frac{10^8}{P_o} \geq 0.5847e - 23.281 = 0.5847 * 240 - 23.281 = 117.047 \text{ 1/N}$$

$$P_o \leq 10^8/117.047 = 854.358 \text{ KN}$$

Thus; $P_o \leq 469.686 \text{ KN}$ and $P_o = 460 \text{ KN}$ can be chosen because which satisfies the solution set.

▪ **Selection of prestressing steel**

According to Euro code manual, when checking the stresses at transfer, the prestressing force required for the in-service condition should be increased by 60%, as the long-term (time-independent) losses will not have occurred, and no tension should be allowed in concrete.

$$P_{\text{req}} = P_o/0.6 = 460/0.6 = 766.667 \text{ kN}$$

Here one assumes 4No.7-wire super strands, so the required characteristic load per strand will be:

$$P_{\text{req}}/3 = 766.667/3 = 255.556 \text{ kN/ strand}$$

From steel manufacturer specification manual selecting 3 No-7- wire drawn strands of 15.2mm nominal diameter with $P_o = 259.0 \text{ kN /strand}$ with nominal strength $f_{pk} = 1860 \text{ N/mm}^2$ and cross sectional area of $A_p = 139.0 \text{ mm}^2$.

The total prestress force P_u is calculated as;

$$P_u = 259.0 * 3 = 777 \text{ kN}$$

The total area of the prestressing steel A_{pu} is calculated as;

$$A_{pu} = A_p * 3 = 139.0 \text{ mm}^2 * 3 = 417 \text{ mm}^2$$

The actual prestress force P_o is now becomes

$$P_o = 0.6 * P_u = 0.6 * 777 = 466.2 \text{ kN}$$

▪ **Calculation of losses**

Before checking the concrete stress at transfer and service the estimated loss should be calculated to get a fairly accurate result.

▪ **Short time losses (Immediate losses)**

○ **Elastic shortening:**

The stress in the prestressing steel due to elastic shortening is given by equation 2.37 as:

$$\sigma_{ci} = \frac{f_{po}}{m + \frac{A_c}{A_p(1 + \frac{e^2}{r^2})}} - \frac{M_o e}{I_c} = \frac{1118 \frac{N}{mm^2}}{5.86 + \frac{126.3 * 10^3 mm^2}{417 mm^2 (1 + \frac{240^2}{112.56^2})}} - \frac{35.65 * 10^6 Nmm * 240 mm}{1600.1 * 10^6 mm^4}$$

$$= \frac{1118 \frac{N}{mm^2}}{5.86 + 54.61} - 5.35 \frac{N}{mm^2} = 13.139 \frac{N}{mm^2}$$

Where:

$$f_{po} = P_o / A_p = 466.2 * 10^3 N / 417 mm^2 = 1118 MPa$$

$$m = E_s / E_{cm} = 205 GPa / 35 GPa = 5.86$$

$$r = (I_c / A_c)^{0.5} = (1600.1 * 10^6 mm^4 / 126.3 * 10^3 mm^2)^{0.5} = 112.56 mm$$

$$e = 240 mm$$

The prestress force loss due to elastic shortening is then computed as;

$$\Delta P_{el} = m * \sigma_{ci} * A_p = 5.86 * 13.139 * 417 = 32106.72 N = 32.107 kN$$

Therefore the percentage loss due to elastic shortening is 6.87%

- **Long time losses (Differed losses)**

Shrinkage

To compute the prestress loss due to shrinkage, the value of concrete shrinkage after tensioning ϵ_r must be known to get $\Delta \epsilon_r$ because once $\Delta \epsilon_r$ is known using equation 2.38 and 2.39 the value of the stress can be computed.

$$\epsilon_r = \epsilon_{cd, \infty} * \frac{j}{j + 0.04 * h_o^{1.5}} + \epsilon_{ca, \infty} * 1 - \exp(-0.2 * \sqrt{j}) \quad (2.41)$$

ϵ_{cd} according to Euro Code 2, EN 1992-1-1 section 3.1.4 is 0.46‰

$$\epsilon_{ca, \infty} = 2.5 * (f_{ck} - 10) * 10^{-4} = 2.5 * (40 - 10) * 10^{-6} = 7.5 * 10^{-5}$$

$$j = 50 \text{ years} = 18250 \text{ days}$$

$$h_o = \frac{2A_c}{U} = \frac{2 * 126.3 * 10^3 mm^2}{[110 + (2 * 330) + (2 * 100) + 900 + (2 * 395)] mm} = \frac{2 * 126.3 * 10^3 mm^2}{[110 + (2 * 330) + (2 * 100) + 900 + (2 * 395)] mm} = 94.96 mm$$

$$\epsilon_{r, 50 \text{ Yrs}} = 0.46 * 10^{-3} * \frac{18250}{18250 + 0.04 * 94.96^{1.5}} + 7.5 * 10^{-5} * 1 - \exp(-0.2 * \sqrt{18250})$$

$$= 4.59 * 10^{-4} * + 7.5 * 10^{-5} = 5.34 * 10^{-4}$$

$$\epsilon_{r,28\text{days}} = 0.46 * 10^{-3} * \frac{28}{28 + 0.04 * 94.96^{1.5}} + 7.5 * 10^{-5} * 1 - \exp(-0.2 * \sqrt{28})$$

$$= 1.98 * 10^{-4} * + 4.9 * 10^{-5} = 2.47 * 10^{-4}$$

$$\Delta\epsilon_r = \epsilon_{r,50\text{Yrs}} - \epsilon_{r,28\text{days}} = (5.34 * 10^{-4}) - (2.47 * 10^{-4}) = 2.87 * 10^{-4}$$

$$\text{Using Equation 2.21, } \Delta P_p = -A_p E_p \Delta\epsilon_r = -417 \text{mm}^2 * 205 * 10^3 \frac{\text{N}}{\text{mm}^2} * 2.87 * 10^{-4}$$

$$\Delta P_p = -24534.195 \text{N} = -21.53 \text{kN}$$

Therefore the percentage loss due to shrinkage is 4.62%

Stress relaxation

According to the specification manual attached in Annex C of the prestressing tendon, the maximum relaxation loss is given to be 2.5%.

Creep

To calculate the deformation due to the sustained compressive load which further facilitates the computation of prestress loss due to creep, first the stress σ_c should be computed,

$$\sigma_c = \frac{P}{A_c} + \frac{P * e_o^2}{I} + \frac{M * e_o}{I}$$

$$\sigma_c = \frac{466.2 * 10^3 \text{N}}{126.3 * 10^3 \text{mm}^2} + \frac{466.2 * 10^3 \text{N} * 240^2 \text{mm}^2}{1600.1 * 10^6 \text{mm}^4} + \frac{-35.65 * 10^6 \text{Nmm} * 240 \text{mm}}{1600.1 * 10^6 \text{mm}^4}$$

$$= 3.69 \frac{\text{N}}{\text{mm}^2} + 16.78 \frac{\text{N}}{\text{mm}^2} - 5.35 \frac{\text{N}}{\text{mm}^2} = 15.12 \text{MPa}$$

$$0.45 f_{ck} = 0.45 * 40 \text{MPa} = 18 \text{MPa} > \sigma_c$$

i.e. the creep strain can be computed using equation 2.43; $\epsilon_{cc}(\infty, t_o) = \varphi(\infty, t_o) \cdot \left(\frac{\sigma_c}{E_c}\right)$ where the creep coefficient can be read from the Figure 3.1 provided in Euro code 2 EN 1991-1-1

The value of $\varphi(\infty, t_o)$ from the graph is then; $\varphi(\infty, t_o) = 2$

The creep strain can then be computed as; $\epsilon_{cc}(\infty, t_o) = \varphi(\infty, t_o) \cdot \left(\frac{\sigma_c}{E_c}\right) = 2 * \left(\frac{15.12}{35 * 10^3}\right) = 8.64 * 10^{-4}$

The stress due to the creep stain is then evaluated as; $\Delta\sigma_p = E_p \epsilon_{CC}$

$$= 205 \times 10^3 \text{ N/mm}^2 \times 8.64 \times 10^{-4} = 177.12 \text{ N/mm}^2$$

The loss in the prestress force is $\Delta P_p = \Delta\sigma_p A_p = 177.12 \text{ N/mm}^2 \times 417 \text{ mm}^2 = 73859.04 \text{ N} = 73.859 \text{ kN}$

Therefore the percentage loss due to Creep is 15.84%

Table 4.4: Percentage losses calculation results for type 1.

TYPE OF LOSSES	PERCENTAGE LOSS OF STRESS
	TYPE-1
Elastic shortening	6.87
Creep of concrete	15.84
Shrinkage of concrete	4.62
Stress relaxation	2.5
Short time losses [%]	6.87
Long time losses [%]	22.96

P_0 is computed taking: $\alpha = 1 - 0.04 = 0.96$ and $\beta = 1 - 0.21 = 0.79$

Now $\alpha = 1 - 0.0687 = 0.9313$ and $\beta = 1 - 0.2296 = 0.7704$

The assumed valued and the computed losses are comparatively accurate.

- **Concrete stress at transfer**

From equation 2.13a, the stress at the top fiber f_t is calculated as;

$$f'_t = \frac{\alpha P_0}{A_c} - \frac{\alpha P_0 e}{Z_t} + \frac{M_o}{Z_t}$$

$$f'_t = \frac{0.9313 \times 466.2 \times 10^3 \text{ N}}{126.3 \times 10^3 \text{ mm}^2} - \frac{0.9313 \times 466.2 \times 10^3 \text{ N} \times 240 \text{ mm}}{14313.2 \times 10^3 \text{ mm}^3} + \frac{35.65 \times 10^6 \text{ Nmm}}{14313.2 \times 10^3 \text{ mm}^3}$$

$$f'_t = 3.44 \frac{\text{N}}{\text{mm}^2} - 7.28 \frac{\text{N}}{\text{mm}^2} + 2.49 \frac{\text{N}}{\text{mm}^2}$$

$$f'_t = -1.35 \frac{\text{N}}{\text{mm}^2} > f'_{min} = -3.5 \frac{\text{N}}{\text{mm}^2} \text{ OK!}$$

From equation 2.13 b, the stress at the bottom fiber f_b is calculated as;

$$f'_b = \frac{\alpha P_0}{A_c} + \frac{\alpha P_0 e}{Z_b} - \frac{M_o}{Z_b}$$

$$f'_b = \frac{0.9313 * 466.2 * 10^3 N}{126.3 * 10^3 mm^2} + \frac{0.9313 * 466.2 * 10^3 N * 240 mm}{5028.6 * 10^3 mm^3} - \frac{35.65 * 10^6 Nmm}{5028.6 * 10^3 mm^3}$$

$$f'_b = 3.44 \frac{N}{mm^2} + 20.72 \frac{N}{mm^2} - 7.09 \frac{N}{mm^2}$$

$$f'_b = 17.07 \frac{N}{mm^2} < f'_{max} = 18 \frac{N}{mm^2} \text{ OK!}$$

▪ **Concrete stress at service**

From equation 2.13 c, the stress at the top fiber f'_t is calculated as;

$$f'_{t,service} = \frac{\beta P_0}{A_c} - \frac{\beta P_0 e}{Z_t} + \frac{M_s}{Z_t}$$

$$f'_{t,service} = \frac{0.7704 * 466.2 * 10^3 N}{126.3 * 10^3 mm^2} - \frac{0.7704 * 466.2 * 10^3 N * 240 mm}{14313.2 * 10^3 mm^3} + \frac{117.507 * 10^6 Nmm}{14313.2 * 10^3 mm^3}$$

$$f'_{t,service} = 2.84 \frac{N}{mm^2} - 6.022 \frac{N}{mm^2} + 8.21 \frac{N}{mm^2}$$

$$f'_{t,service} = 5.028 \frac{N}{mm^2} < (f'_{max})_{serv} = 24 \frac{N}{mm^2} \text{ OK!}$$

From equation 2.13 d, the stress at the top fiber f'_b is calculated as;

$$f'_{b,service} = \frac{\beta P_0}{A_c} + \frac{\beta P_0 e}{Z_b} - \frac{M_s}{Z_b}$$

$$f'_{b,service} = \frac{0.7704 * 466.2 * 10^3 N}{126.3 * 10^3 mm^2} + \frac{0.7704 * 466.2 * 10^3 N * 240 mm}{5028.6 * 10^3 mm^3} - \frac{117.507 * 10^6 Nmm}{5028.6 * 10^3 mm^3}$$

$$f'_{b,service} = 2.84 \frac{N}{mm^2} + 17.14 \frac{N}{mm^2} - 23.37 \frac{N}{mm^2}$$

$$f'_{b,service} = -3.39 \frac{N}{mm^2} > f'_{min} = -3.5 \frac{N}{mm^2} \text{ Ok!}$$

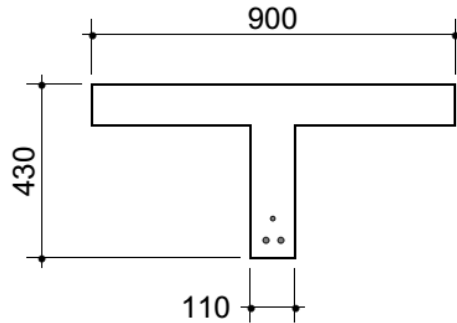


Figure 4.3: Cross section for Type 1

▪ **Bending Moment resistance**

The initial stress in tendons according to Euro code 2 EN 1992-1-1:2004 is

$$f_{po} = 0.7f_{pk} = 0.7 \cdot 1860 \text{ N/mm}^2 = 1302 \text{ N/mm}^2$$

The elastic modulus of steel is $E_p = 205 \text{ kN/mm}^2$

Using the procedure for design of flanged beam

- $f_{po}/f_{pk} = 0.7$
- $K = M/bd^2f_{ck} = [((1.3 \cdot 8.231) + (1.6 \cdot 1.76))9.7^2/8] \cdot 10^6 / (900 \cdot 352^2 \cdot 40) = 0.036$
- Using Figure A1 in Annex A, $x/d = 0.15$ which implies that $x = 52.8 \text{ mm}$. Checking the value of x/d with the limiting value obtained from Table A1, $0.15 < 0.35 \Rightarrow \text{ok!}$
- $0.8x \leq h_f$, $42.24 \text{ mm} \leq 100 \text{ mm} \Rightarrow A_p$ is determined as for a rectangular beam of breadth b
- Using Figure A2 in Annex A, the ratio of $K_{lim} = A_p f_{pk} / bdf_{ck} = 0.04$ and $K = A_p f_{pk} / bdf_{ck} = 417 \cdot 1860 / (900 \cdot 352^2 \cdot 40) = 0.000174$ i.e. $K < K_{lim}$ which means area of reinforcement provided for serviceability is satisfactory at ultimate limit state.

▪ **Minimum reinforcement**

The minimum area of longitudinal reinforcement in order to avoid brittle failure should not be less than either $0.6b_t d / f_{yk}$ or $0.0015b_t d$.

$$0.6b_t d / f_{yk} = 17.84 \text{ and } 0.0015b_t d = 58.05 \text{ which in both case is satisfied.}$$

Providing minimum area of compression reinforcement for lifting and placing of the T-sections into their position: use $5\phi 8c/c 200 \text{ mm}$

▪ **Tendon spacing**

According to manual for the design of reinforced concrete building structures to EC2,

- Distance between the pretensioned tendons is taken as 25mm
- Clear vertical distance between tendons is taken as 25mm
- According to table A2 of Annex A the cover requirement is not fulfilled meaning that the width of the web should be increased to 125mm as shown in Figure 4.4 below.

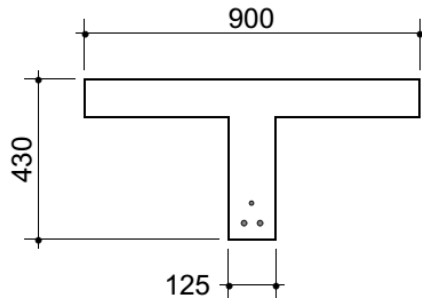


Figure 4.4: modified cross section for Type 1

▪ **Shear design**

Table 4.5: Modified geometric property after increased web width

Label	Type 1
Area [mm ²]*10 ³	131.3
Inertia[mm ⁴]*10 ⁶	1756.9
y _t [mm]	118
y _b [mm]	312
Z _t [mm ³]*10 ³	14942.8
Z _b [mm ³]*10 ³	5623.2

▪ **Modified design loads for the prestressed concrete beams**

$$\text{self weight} = \gamma A_c = 24 \times 131.3 \times 10^3 \times 10^{-6} = 3.151 \text{ kN/m}$$

The maximum moment due to this loading is: transfer moment,

$$M_t = 3.151 (9.7)^2 / 8 = 37.06 \text{ kNm}$$

Uniformly distributed load due to self-weight of the beam and other dead loads per meter width, w_o

$$w_o = \gamma A_c + w_d = (24 \times 131.3 \times 10^3 \times 10^{-6}) + 5.2 = 8.351 \text{ kN/m}$$

The total loading at SLS is this plus the imposed loading, i.e.: SLS moment,

$$M_s = (8.231 + 1.76)(9.7)^2 / 8 = 117.507 \text{ kNm}$$

Total loading at ultimate limit state, w_{ult}

$$w_{ult} = 1.3 * 8.351 + 1.6 * 1.76 = 13.67 \text{ kN/m}$$

The maximum moment due to this loading is: ultimate moment,

$$M_{ult} = 13.67 (9.7)^2 / 8 = 160.78 \text{ kNm}$$

Design ultimate shear force, V_{sd}

$$V_{sd} = w_{ult} * (L/2 - d) = 13.67 * (9.7/2 - 0.352) = 61.49 \text{ kN}$$

$$V_{Rd2,red} = 1.67 V_{Rd2} \left(\frac{1 - 1.5 \sigma_{cp,eff}}{f_{ck}} \right) \leq V_{Rd2}$$

$$V_{Rd2} = 0.15 f_{ck} b_{w,nom} d = 0.15 * 40 \frac{N}{mm^2} * 125 \text{ mm} * 352 \text{ mm} = 264000 \text{ N} = 264 \text{ kN}$$

$$b_{w,nom} = 125 \text{ mm}$$

$$\sigma_{cp,eff} = 1.2 P_o / A_c = 1.2 * 466.2 * 10^3 \text{ N} / 131.3 * 10^3 \text{ mm}^2 = 4.26 \text{ MPa}$$

$$V_{Rd2,red} = 1.67 * 264 * 10^3 \text{ N} \left(\frac{1 - 1.5 * 4.26 \text{ MPa}}{40 \text{ MPa}} \right) \leq V_{Rd2}$$

$$V_{Rd2,red} = -59.408 \text{ kN} \leq 264 \text{ kN}$$

$$V_{Rd1} = v_{Rd1} b_w d \text{ referring figure A3 in annex A the value of } v_{Rd1} = 1.3 \text{ N/mm}^2$$

$$V_{Rd1} = 1.3 \frac{N}{mm^2} * 125 \text{ mm} * 352 \text{ mm} = 57.2 \text{ kN}$$

The shear reinforcement using standard method

$$A_{sw} = \frac{1.28 s (V_{sd} - V_{Rd1})}{f_{yw} d} = \frac{1.28 * 200 \text{ mm} (61.49 * 10^3 \text{ N} - 57.2 * 10^3 \text{ N})}{300 \frac{N}{mm^2} * 352 \text{ mm}} = 10.4 \text{ mm}^2$$

use $\emptyset 6$ c/c 200mm

Minimum reinforcement requirement $A_{sw} = \rho_w s b_w = 0.0028 * 200 * 125 = 70.0 \text{ mm}^2$, ρ is obtained from Table A3 in Annex A, use $\emptyset 10$ c/c 200mm

Reinforcement distribution in a flanged beam should be according to shown in Figure 4.5 below.

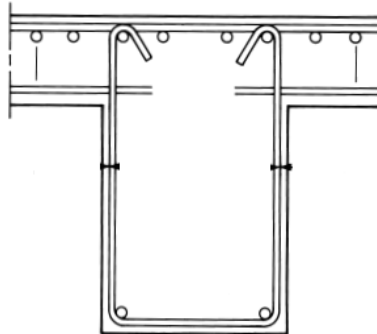


Figure 4.5: Distribution of reinforcement in the rib (T-section) [10]

Final design result for the rib section labeled type 1, type 2, type 3 and type 4 can be summarized in the Table 4.6 below.

Table 4.6: Design summary for the rib (T-sections)

Section name	Type -1	Type -2	Type -3	Type -4
b_f [mm]	900	900	900	900
b_w [mm]	125	160	160	160
D [mm]	430	320	215	215
D_f [mm]	100	100	100	100
Area (A_c) [mm^2]* 10^3	126.3	125.2	108.4	108.4
I [mm^4]* 10^6	1600.1	864.7	271.8	271.8
y_t [mm]	112	95	68	68
y_b [mm]	318	225	147	147
Z_t [mm^3]* 10^3	14313.2	9104.1	3982.9	3982.9
Z_b [mm^3]* 10^3	5028.6	3843	1852.2	1852.2
e [mm]	240	145	70	70
A_p [mm^2]	417	417	417	417
Prestressing force [kN]	466.2	545.4	518.4	518.4
Tendon spacing [mm]	25	25	25	25
longitudinal compression reinforcement and spacing [mm]	5Ø8 c/c 200	5Ø8 c/c 200	5Ø8 c/c 200	5Ø8 c/c 200
shear reinforcement and spacing [mm]	Ø10 c/c 200	Ø12 c/c 200	Ø12 c/c 200	Ø12 c/c 200

4.5 ANALYSIS AND DESIGN OF THE GIRDER BEAM

4.5.1 ARRANGEMENTS AND INITIAL DESIGN FOR THE GIRDER BEAM

The ribbed slab types selected for the design of the slabs are supported in the girder beams having an arrangement shown in Figure 4.1. The girder beams are then connected to the column to give an overall frame system shown in Figure 4.6 below. For the initial design of the girder beam, minimum dimension are selected according to the Eurocode manual [10] and the depth requirement to suite the ribbed T-sections. First according to the arranged structural layout of the building there exist ten types of girder beams that vary in span. The initial dimension for the ten types of the girder beams are listed in Table 4.7 below. The initial depth for the beams is computed using the Table A5 in Annex A. Accordingly for the purpose of assessing the self weight of the beam, the width can be taken as half of the depth but usually not less than 300mm.

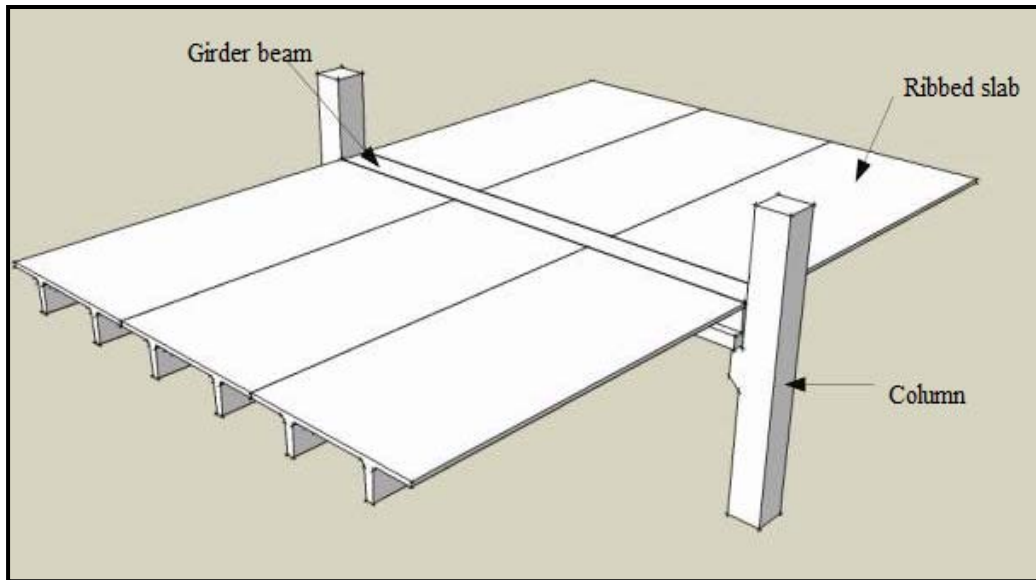


Figure 4.6: Connection of the ribbed slab to girder beam and girder beam to column. [Courtesy of conceptual design for a pre-cast concrete hotel in Iraq]

Table 4.7: Initial dimensions of the girder beam

Girder Beam	Length [m]	D [mm]	b [mm]	Type of beam	
Ax 6-8	12.5	569	300	continuous	Along X-axis
Ax 4-5'	8.9	495	300	simply supported	
Ax 4-6	11	500	300	continuous	
Ax 4-5 & 5-6	5.5	250	300	continuous	
Ax 2'-4	8.95	407	300	continuous	
Ax 2-2'	2.45	137	300	simply supported	
Ax 2-4	11.4	519	300	continuous	
Ax A-B & Ax A'-B'	4.4	245	300	continuous	Along Y-axis
Ax B-D & AX B'-E	9.7	441	300	simply supported	
Ax C-D & Ax C'-E	5.5	250	300	continuous	
Ax D-E	8.3	378	300	continuous	

The shape of the girder beam should be in such a manner that it is convenient to support the rib slabs (T-sections) and create a flat serviceable floor surface. To support the T-sections the girder beam should be designed in an inverted T-section. At manufacturing site the girder beams would be produced as shown in Figure 4.1 above. The width of inverted flange section of the girder beam is taken to be 800mm and depth

of the flange is taken to be 100mm for initial design. The initial data for the width and depth of the inverted flange is taken from PCI design handbook [8].

4.5.2 ANALYSIS OUTPUTS OF GIRDER BEAM USING SOFTWARE

The frame for the precast prestressed concrete unit is analyzed using ETABS v9.7.3. The analysis result is then used to design the girder beams. While using pre-cast units, the design for the beams should be done in such a manner that member typicality should play a major role. For such a reason for the similar floor system the maximum analysis result is taken for the design of the girder beams. But, in cases where the analysis result varies in an enormous amount special design for that specific member is done. While tabulating the table 4.9 below engineering judgment is done to group the similar beams according to the analysis result and to select the analysis result accordingly.

While analyzing the following modeling considerations were in use;

- The wall and frame are connected by pin connectors at every floor.
- Rigid floor diaphragms are assigned at every floor levels
- Live load reduction factors are considered as per EBCS-1. This is determined in Table 3.5.
- Stiffness modification factors were adjusted according to Table 4.8

Table 4.8: Stiffness modification factors

	Modification factor
Girder Beam	1
Ribbed slab	1
Columns	0.7
Cracked walls	0.35
Un-cracked walls	0.7

In Table 4.9,

- M_t = transfer moment at the bottom
- M_{sb} = service moment at the bottom
- M_{stL} = service moment at top left
- M_{stR} = service moment at top right
- M_{ub} = ultimate moment at the bottom
- M_{utL} = ultimate moment at top left
- M_{utR} =ultimate moment at top right
- V_{ult} = ultimate shear force

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

Table 4.9: Analysis result for the girder beams

Girder beam			At service load			At ultimate load				No. of beams
ID	Transverse axis	similar stories	Msb	MsttL	MstR	Mub	MutL	MutR	Vult	
Ax 4-5'	Ax A&A'	1st-TTB	207.31	-235.91	-491.39	558.32	-1022.53	-1022.53	538.57	28
Ax 2'-4	Ax B&B'	1st-TTB	458.99	0.00	-618.57	624.62	0.00	-1017.46	420.43	28
Ax 4-6	Ax B&B'	1st-TTB	476.95	-828.13	-828.13	722.60	-1617.62	-1617.62	-489.96	28
Ax 6-8	Ax B&B'	1st-TTB	942.82	-1090.13	0.00	1013.96	-1592.56	0.00	753.99	28
Ax 2-2'	Ax C&C'	1st-TTB	16.06	-6.42	0.00	32.41	-37.18	0.00	513.50	28
Ax 2'-4	Ax D	1st-5th, TTB	290.31	0.00	-447.03	549.86	0.00	662.53	472.14	14
Ax 4-5	Ax D	1st-5th, TTB	87.62	-148.40	-148.40	195.02	-311.09	-311.09	251.95	14
Ax 5-6	Ax D	1st-5th, TTB	92.65	-153.91	-153.91	146.45	-364.71	-364.71	264.33	14
Ax 6-8	Ax D	1st-5th, TTB	532.08	-1015.61	-1015.61	957.38	-947.80	-947.80	565.58	14
Ax 2-4	Ax E	1st -TTB	457.13	-770.20	-770.20	683.37	-937.71	-937.71	646.49	14
Ax 4-4'	Ax E	2nd -TTB	59.80	-29.32	0.00	80.10	-55.59	0.00	119.62	14
Ax 5-6	Ax E	3rd -TTB	90.45	0.00	-132.75	175.79	0.00	-344.60	673.35	14
Ax 6-7	Ax E	4th -TTB	632.29	-531.30	0.00	1040.15	-1427.68	0.00	566.96	14
Ax C-D	Ax 2	1st-TTB	27.28	0.00	-19.80	60.53	0.00	-100.30	52.22	14
Ax D-E	Ax 2	1st-TTB	41.72	-45.93	0.00	246.92	-346.72	0.00	81.72	14
Ax E-C'	Ax 2	1st-TTB	27.28	-450.63	-50.93	127.31	-157.14	-157.14	92.29	14
Ax C'-B'/C-B	Ax 2	GR, 1st, TTB	12.36	-6.68	1.50	7.63	-13.37	-13.37	8.77	6
Ax A-B	Ax 4	1st -TTB	44.70	44.70	-60.30	178.25	-199.49	-199.49	105.51	14
Ax B-D	Ax 4	1st -TTB	81.14	81.41	-141.15	626.83	-815.67	-815.67	222.59	14
Ax D-E	Ax 4	1st -TTB	21.43	-46.78	21.43	482.06	-550.50	-550.50	132.06	14
Ax E-B'	Ax 4	1st -TTB	81.14	-141.15	-46.78	626.83	-815.67	-550.50	222.59	14
Ax B'-A	Ax 4	1st -TTB	44.70	-60.30	44.70	178.25	-199.49	-199.49	105.51	14
Ax A-B/A'-B'	Ax 5'	1st -TTB	84.89	-102.22	84.89	218.56	-220.07	-220.07	59.92	28
Ax B-D	Ax 6	1st -TTB	72.92	-244.13	-62.95	669.68	-868.67	-868.67	245.89	14
Ax D-E	Ax 6	1st -TTB	95.57	-193.07	-103.69	475.14	-668.92	-668.92	297.93	14
Ax E-B	Ax 6	1st -TTB	72.92	-86.28	-143.45	669.68	-868.67	-868.67	245.89	14
Ax C-D	Ax 8	1st -TTB	32.47	-21.95	0.00	95.60	-129.37	0.00	51.09	14
Ax D-E	Ax 8	1st -TTB	38.04	0.00	-54.98	280.10	0.00	-397.19	83.44	14
Ax E-C'	Ax 8	1st -TTB	49.37	0.00	0.00	64.18	0.00	0.00	47.05	14

4.5.3 DESIGN OF THE PRE-CAST PRESTRESSED GIRDER BEAM

For the design of the girder beam concrete of C50/60 is used. The design parameters are listed below; Detail design sample for girder beam is attached n Annex B2 of this research. An excel template is prepared to design the girder beams and the design output is listed in Table 4.10 below.

Design parameters for the prestressed concrete girder beam

For transfer at 7 days, the compressive strength of concrete for C60 is;

$$f_{ck} = 47 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable compressive concrete stress at transfer, f_{max}^* , is;

$$f_{max}^* = 22.5 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable tensile concrete stress at transfer, f_{min}^* , is

$$f_{min}^* = -4.1 \text{ N/mm}^2$$

For serviceability state, the compressive strength of concrete, f_{ck} , is

$$f_{ck} = 50 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable concrete stress allowed under service is,

$$(f_{max})_s = 30 \text{ N/mm}^2$$

Using the same design procedure the design for girder beams is done, and the calculation is presented in Annex B of this research. The summary for the calculation can be presented in Table 4.10 below.

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

Table 4.10: Design summary for the girder beams

Section name	Area (A _c) [mm ²]*10 ³	I [mm ⁴]*10 ⁶	y _t [mm]	y _b [mm]	Z _t [mm ³]*10 ³	Z _b [mm ³]*10 ³	e [mm]	A _p [mm ²]	Pu[kN]	longitudinal compression reinforcement and spacing [mm]	shear reinforcement and spacing [mm]	
Ax 4 - 5'	246.25	4507.73	272.47	227.53	16543.68	19811.99	147.00	1350.00	1625.40	3Ø24 c/c125	Ø16 c/c 200	Ax A/A'
Ax 2' - 4	307.50	10303.93	367.93	232.07	28005.37	36529.27	202.00	1200.00	1632.00	3Ø20 c/c100	Ø16 c/c 200	Ax B/B'
Ax 4-6	315.00	8320	371	229	22444.1	36288.1	149.00	1200.00	1468.80	5Ø24 c/c100	Ø16 c/c 200	
Ax 6-8	501.4	23105.54	464.96	335.04	49693.59	68963.54	255.00	1500.00	2004.00	3Ø20 c/c110	Ø16 c/c 200	
Ax 2-2'	92.25	352.41	126.36	93.64	2788.9	3763.53	13.00	600.00	741.60	3Ø16 c/c125	Ø16 c/c 200	Ax C/C'
Ax 2' - 4	315.00	8320.00	371.00	229.00	22444.00	36288.10	149.00	1650.00	2105.40	4Ø24 c/c100	Ø16 c/c 200	Ax D
Ax 4-5	315.00	8320.00	371.00	229.00	22444.00	36288.10	149.00	900.00	1152.00	2Ø16 c/c110	Ø16 c/c 200	
Ax 5-6	315.00	8320.00	371.00	229.00	22444.00	36288.10	149.00	900.00	1152.00	2Ø16 c/c110	Ø16 c/c 200	
Ax 6-8	501.4	23105.54	464.96	335.04	49693.59	68963.54	255.00	1650.00	2006.40	2Ø20 c/c125	Ø16 c/c 200	
Ax 2- 4	501.4	23105.54	464.96	335.04	49693.59	68963.54	255.00	2100.00	2578.80	2Ø20 c/c125	Ø16 c/c 200	Ax E
Ax 4-4'	92.25	352.41	126.36	93.64	2788.9	3763.53	13.00	1350.00	1695.60	2Ø12 c/c125	Ø16 c/c 200	
Ax 5-6	315.00	8320.00	371.00	229.00	22444.00	36288.10	149.00	900.00	1148.40	2Ø16 c/c110	Ø16 c/c 200	
Ax 6-7'	501.40	23105.54	464.96	335.04	49693.59	68963.54	255.00	1950.00	2379.00	3Ø20 c/c125	Ø16 c/c 200	Ax 2
Ax C-D	90.00	675.00	150.00	150.00	4500.00	4500.00	70.00	450.00	482.40	4Ø16 c/c125	Ø16 c/c 200	
Ax-D-E	120.00	1600.00	200.00	200.00	8000.00	8000.00	120.00	450.00	505.80	3Ø20 c/c125	Ø16 c/c 200	
Ax E-C'	90.00	675.00	150.00	150.00	4500.00	4500.00	70.00	450.00	482.40	4Ø16 c/c125	Ø16 c/c 200	Ax 4
Ax C'-B'/C-B	120.00	1600.00	200.00	200.00	8000.00	8000.00	120.00	450.00	505.80	3Ø20 c/c125	Ø16 c/c 200	
Ax A-B	90.00	675.00	150.00	150.00	4500.00	4500.00	70.00	600.00	667.20	3Ø24 c/c125	Ø16 c/c 200	Ax 4
Ax B-D	165.00	4159.38	275.00	275.00	15125.00	15125.00	195.00	600.00	653.60	4Ø24 c/c125	Ø16 c/c 200	
Ax D-E	135.00	2278.13	225.00	225.00	10125.00	10125.00	145.00	300.00	362.40	5Ø24 c/c125	Ø16 c/c 200	
Ax E-B'	165.00	4159.38	275.00	275.00	15125.00	15125.00	195.00	600.00	653.60	4Ø24 c/c125	Ø16 c/c 200	Ax 5'
Ax B'-A	90.00	675.00	150.00	150.00	4500.00	4500.00	70.00	600.00	667.20	3Ø24 c/c125	Ø16 c/c 200	
Ax A-B/B'-A'	90.00	675.00	150.00	150.00	4500.00	4500.00	70.00	600.00	667.20	3Ø24 c/c125	Ø16 c/c 200	Ax 6
Ax B-D	165.00	4159.38	275.00	275.00	15125.00	15125.00	195.00	600.00	653.60	4Ø24 c/c125	Ø16 c/c 200	
Ax D-E	135.00	2278.13	225.00	225.00	10125.00	10125.00	145.00	300.00	362.40	5Ø24 c/c125	Ø16 c/c 200	
Ax E-B'	165.00	4159.38	275.00	275.00	15125.00	15125.00	195.00	600.00	653.60	4Ø24 c/c125	Ø16 c/c 200	Ax 8
Ax C-D	90.00	675.00	150.00	150.00	4500.00	4500.00	70.00	450.00	482.40	4Ø16 c/c125	Ø16 c/c 200	
Ax-D-E	120.00	1600.00	200.00	200.00	8000.00	8000.00	120.00	450.00	505.80	3Ø20 c/c125	Ø16 c/c 200	
Ax E-C'	90.00	675.00	150.00	150.00	4500.00	4500.00	70.00	450.00	482.40	4Ø16 c/c125	Ø16 c/c 200	

4.6 CONNECTIONS AND DETAILINGS

The major part for application of pre-cast concrete is detailing required at the column-beam junction. As noted in this research detail design and cost analysis regarding connection of beam-column is not covered. In this section possible connection details for the pre-cast prestressed systems is recommended based on the previous researches done.

- French et al. (1981) developed connectors using post-tensioned bars to connect the beams to columns. The post-tensioned bars were designed to relocate yielding away from the interface.
- A "drop in" beam system, built by Rockwin Corporation in the 1980s used a monolithic concrete technology by building precast concrete frames with member splices away from anticipated regions of inelastic action. While these types of details have been to behave similarly to monolithic concrete frames, they are often difficult and expensive to implement in the field. In addition these systems require awkward precast concrete members (cruciform, "H" shaped or tees) that increase transportation costs. The cost associated with creating well-built connectors, the over strength required in connectors becomes large as the hinge location is moved from the column face.
- Ductile link connectors are an alternative approach to create monolithic concrete frames. This connector allows the beam and columns to be cast independently and joined at column face by bolting. An example for this kind of connection is provided in figure 4.7a and 4.7b below.

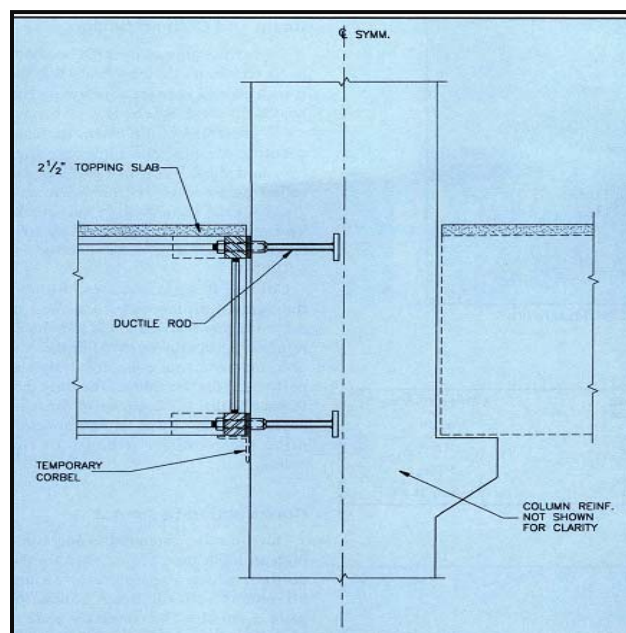


Figure 4.7 a: Frame beam to column connection detail elevation [25].

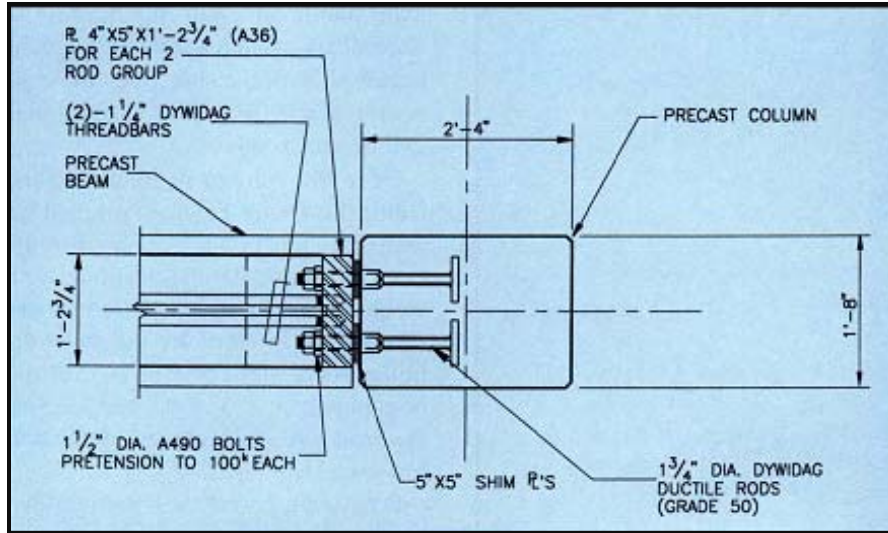


Figure 4.7 b: Frame beam to column connection detail plan [25].

- Using a continuous reinforcement for the longitudinal beam is one of the options to join the column and beam. Figure 4.8 below shows a specimen for such an arrangement ready for a cyclic loading test to study the performance of the connection. The simplicity of this connection was reflected in its low cost.

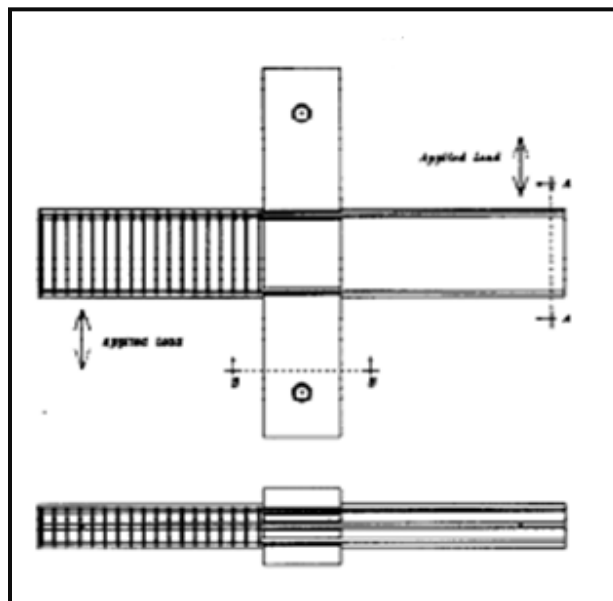


Figure 4.8: Frame beam to column connection [22].

CHAPTER 5

ECONOMIC ANALYSIS

5.1 BACKGROUND OF REINFORCED CONCRETE

Ethiopia's construction industry is one of the countries booming industries at hand. The development of many infrastructures in railway, housing, and bridges can be a witness for the development of the industry. And it is pointed that the construction industry has a significant contribution to the country's economy. The function of reinforced concrete in the construction industry plays the leading role. Conditions that help to improve or develop the usage of reinforced concrete will directly impact the economic aspect of the project as well as the country. Industrialization of the construction industry can hence be done by providing an effective solution to the problems of the industry at hand. One of the prospective solutions can be using prefabricated units. Pre-cast prestressed concrete has a potential to change the industry if a detail study on the cost of the installation of such industry is done in detail. In the following section assessment of the current usage of prestressed concrete is noted followed by the results of the site visit on the current condition of the prestressed concrete manufacture.

5.2 CURRENT USAGE OF PRESTRESSED CONCRETE

Currently prestressed concrete is primarily used in production of railway slippers and bridge girders. The projects are mainly organized under Ethiopian Railway Corporation (ERC) and executed by China Railway Engineering Corporation (CREC). A site visit was done to the manufacturing site of CREC. CREC is a China railway construction industry established before three and half years and the site is located in Adama, Mermersa. The industry owns prestressing equipments, transportation equipments, cranes and concrete mixing plants. The main products of this site are railway slippers, bridge girders that run about 25-32meter span and electric poles. Pictures that show the produced units, the overall site layout, equipments and storage silos are attached in Annex F of this research. Detail data about the site is also provided in the questionnaire paper attached in Annex E. Currently there is no movement regarding the usage of prestressed concrete for a building project. This research mainly focuses on the prospective of using pre-cast prestressed members for the housing projects. Economic aspect of this system regarding quality, time and cost is dealt on the following sub-chapter.

5.3 QUALITY

Considering housing projects quality assurance it gets ambiguous because these projects are intended to be low cost projects and quality assurance is vaguely addressed. The already serving condominium houses are tormented with quality problems. Handling the concrete mixes and concrete grade at the site is not an easy task as compared to using pre-manufactured units. Because while using a pre-cast units concrete mixes are controlled and designed to meet the required grade.

Figure 6.1a and Figure 6.1b below shows the usual problems encountered on these housing projects. Serviceability of the slabs is at risk and maintenance is required without serving service year of the structure. The ascetic value of the rooms gets questionable due to the pipes that run along the kitchen and toilet area.



Figure 5.1: Problem in the current construction slabs

[Picture taken from Gergi condominium G+ 4 building sites]

Another major quality problem is encountered on the stairs of these housing projects. The stairs are made of steel structure frame and the treads are concrete units. Figure 6.2 shows the connection problem of the stairs that are currently used in G+4 condominium sites. Because of less quality control, close maintenance and service of the composite stairs, the stairs are highly damaged in some units of these formerly constructed condominium sites.



Figure 5.2: problems in stair connection of condominium site

[Picture taken from Gergi condominium G+ 4 buildings sites]

The above problems are avoided while using pre-cast units, the pipes can be installed in the ribs hence creating aesthetically pleasing roof for the rooms. Although design of stair is not studied in this research it is recommended that studies should be done on the economic aspect of using pre-cast prestressed stairs. The pre-cast stairs will have a tendency to contribute for the quality assurance.

5.4 TIME

Time delay in construction is variable depending upon the type of construction, the financial supplier, material supply and the site condition at hand. The main reasons for delay in construction can be sign changes, weather changes, site condition, shortage of equipments, late deliveries of materials, economic condition, increase in quantities; owners' interference, inadequate contractor experience, labor productivity, slow decision making, improper planning and subcontractors are among the important factors causing delay.

The 40/60 housing project at Gergi site is executed by different contractors. Depending upon the contractor efficiency the project is currently at different stage. The fastest ongoing construction is currently in 12th floor concrete work and lift and stair case construction for 8th floor and above. Another contractor is at the level of 7-8th floor concrete work and there are site that terminated the construction at ground level because of disputes between the contractor and the owner.

The major advantage of using pre-cast units is their construction is quick as compared to the conventional reinforced concrete. This is mainly because there is no waste in time while curing floors and time regarding the installation of formwork is eliminated. Such advantage of this project will play a major role

in the housing project to reduce or eliminate the problem regarding time delay. The time delay produced affects the total cost of the building by increasing the total cost and leading to unexpected expenses. In addition, the precast process takes place in a controlled environment and is unaffected by weather.

5.5 MATERIAL COST

For the purpose of cost evaluation the following tasks are first carried out.

- Design of the 40/60 housing project using the conventional reinforced concrete is done using ETABS V9.7.3
- Design of the 40/60 housing project using the pre-cast prestressed concrete is done using ETABS V9.7.3 for the frame analysis and manual calculation for the design of the specific members.
- Quantity for the ribbed slab and girder beams using the conventional reinforced concrete is calculated and compared with the price for ribbed slab (T-sections) using the pre-cast prestressed units.
- Quantity for the columns using the conventional reinforced concrete is calculated and compared with the price for columns using the pre-cast prestressed units. Note: while calculating total mass in kilo gram for the reinforcement an excel program that calculate rebar quantity using ETABS output developed by Atul Tegar, M.Tech. Civil (Building Science and Technology) from IIT Roorkee is used. The code for the excel program is attached in Annex B of this research.
- Finally comparison is done based on the quantity and current unit rate for the materials.
- Comparison excludes maintenance costs of both systems.

5.5.1 MATERIAL COST FOR SLAB

Material cost for the two systems can be compared by taking the quantity values tabulated in the tables below and the rebar quantity is computed in Annex B of this research. Concrete grade used for reinforced concrete system is C25 (which is concrete strength having 25MPa of cube strength). And concrete grade used for reinforced concrete system is C50 (which is concrete strength having 50MPa of cube strength). Table 5.1, Table 5.2, Table 5.3 and Table 5.4 below shows the cost of reinforcing bar, prestressing strand and concrete. Detail calculation of quantities for the rebar is provided in Annex B of this research.

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

Table 5.1: Cost summary for the reinforced concrete system slab

Item	Description	Unit	Quantity	Unit price [Birr/m³]	Amount [Birr]
1	Ground floor slab				
	170mm thick Ground Floor Slab	m ³	122.07	2500.00	305,167.65
	180mm thick Ground Floor Slab	m ³	31.66	2500.00	79,143.75
	Formwork				
	formwork to 170mm thick	m ²	1,047.62	147.00	154,000.14
	Formwork to 180mm thick	m ²	893.92	147.00	131,406.24
	Rebar				
	Dia 10 mm deformed bar	kg	35,174.14	35.00	1,231,094.90
	Dia 12 mm deformed bar	kg	68,559.99	35.00	2,399,599.65
2	First Floor Slab upto 12th floor slab				
	Ribbed slab for suspended floor slab made of 60 mm thick concrete slab in C-25,60x120 mm one way concrete girder with c/c spacing of 600mm ,220x520 mm hollow concrete ribbed block and all according to the detail structural drawing.				
		m ³	532.09	3966.66	2,110,618.53
2.1	Mesh				
	Dia 8mm	kg	61,601.04	35.00	2,156,036.40
2.2	Support reinforcements				
	Dia 14mm	kg	26,571.16	35.00	929,990.75
3	Pre-cast elements (PB1) to the size of 5500x120x80mm made of 0.0528m ³ of concrete in C-25 /beam.Price includes beam erecting, bending, and placing in position, welding of reinforcement bar and mould. According to structural detail drawing.				
		Pcs	1,860.00	1,800.00	3,348,000.00
	(a) Dia 6mm plain bar (5.544kg)				
	(b) Dia 12 mm deformed bar (5.2332kg)				
	(c) Dia 14 mm deformed bar (20.038kg)				
	to be placed in position.				

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

Item	Description	Unit	Quantity	Unit price [Birr/m3]	Amount [Birr]
4	Pre-cast elements (PB2) to the size of 4400x120x80mm made of 0.042m ³ of concrete in C-25 /beam. Price includes beam erecting, bending, placing in position, welding of reinforcement bar and mould. According to structural detail drawing.	Pcs	1,754.00	1,700.00	2,981,800.00
	(a) Dia 6mm plain bar (5.28kg)				
	(b) Dia 12 mm deformed bar (10.68kg)				
	to be placed in position.				
5	Pre-cast elements (PB3) to the size of 4200x120x80mm made of 0.040m ³ of concrete in C-25 /beam. Price includes beam erecting, bending, placing in position, welding of reinforcement bar and mould. According to structural detail drawing.	Pcs	468.00	1,650.00	772,200.00
	(a) Dia 6mm plain bar (5.28kg)				
	(b) Dia 12 mm deformed bar (10.68kg)				
	to be placed in position.				
	Total				16,599,058.02

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

Table 5.2: Cost summary for the concrete used in Pre-cast prestressed concrete system slab

Item	Description	Unit	Pcs of T-beam	Area of concrete For one T-beam(m ²)	Volume of concrete (m ³)	Total	Unit price [Birr]	Amount [Birr]
1	Ground floor T-beams(C-50)							
	B/n axis 4-5' and A-B	m ³	8.00	0.11	0.44	3.52	3500.00	12,320.00
	B/n axis 2-8 and B-D	m ³	37.00	0.13	1.22	45.21	3500.00	158,249.00
	B/n axis 2-8 and D-E	m ³	25.00	0.13	1.04	25.94	3500.00	90,781.25
	B/n axis 2-8 and E-B'	m ³	37.00	0.13	1.22	45.14	3500.00	157,990.00
	B/n axis 4-5' and B'-A'	m ³	8.00	0.11	0.44	3.52	3500.00	12,320.00
	First floor-12th floor							
2	T-beams(C-50)							
	B/n axis 4-5' and A-B	m ³	96.00	0.11	0.44	42.24	3500.00	147,840.00
	B/n axis 2-8 and B-D	m ³	408.00	0.13	1.22	497.76	3500.00	1,742,160.00
	B/n axis 2-8 and E-B'	m ³	408.00	0.13	1.22	497.76	3500.00	1,742,160.00
	B/n axis 4-5' and B'-A'	m ³	96.00	0.11	0.44	42.24	3500.00	147,840.00
	B/n axis 2-2' and C-D	m ³	36.00	0.11	0.55	19.80	3500.00	69,300.00
	B/n axis 2-8 and D-E	m ³	300.00	0.13	1.04	312.00	3500.00	1,092,000.00
Total						1,535.13		5,372,960.25

Table 5.3: Cost summary for the re-bar used in Pre-cast prestressed concrete system slab

Item	Description	unit	Quantity	Unit Rate [Birr/Kg]	Amount [Birr]
1	Longitudinal and shear reinforcement in T-beams				
	Dia 8mm rebar	kg	28,509.32	35	997,826.2
	Dia 10mm rebar	kg	4,120.63	35	144,222.05
	Dia 12mm rebar	kg	1,858.53	35	65,048.55
Total					1,207,096.8

Table 5.4: Cost summary for the prestressing strand used in Pre-cast prestressed concrete system slab

Item	Description	Unit	Quantity in number	Mass/Length,kg/m	Length [m]	Unit price [Birr/kg]	Amount [Birr]
	Ground Floor						
2	Pre-stressed Concrete						
	Type 1	Kg	222	1.086	9.7	43.75	102,313.4
	Type 2	Kg	75	1.086	8.3	43.75	29,576.53
	Type 3	Kg	0	1.086	5.5	43.75	0
	Type 4	Kg	48	1.086	4.4	43.75	10,034.64
	First Floor up to 12 th floor						
2	Type 1	Kg	2448	1.086	9.7	43.75	1,128,213
	Type 2	Kg	900	1.086	8.3	43.75	354,918.4
	Type 3	Kg	108	1.086	5.5	43.75	28,222.43
	Type 4	Kg	384	1.086	4.4	43.75	80,277.12
Total							1,128,213

5.5.2 MATERIAL COST FOR COLUMN

Material cost for the two systems can be compared by taking the quantity values computed in Annex B of this research. Concrete grade used for both systems is C30, which is concrete strength having 30Mpaof cube strength. Table 5.5 and Table 5.6 below show the cost of steel and concrete respectively.

Table 5.5: Cost summary for the steel used in columns

	Total mass [kg]	Price per kg [Birr]	Total cost [Birr]
For reinforced concrete	45979.14	35	1,609,270
For pre-cast prestressed system, reinforcement bar	57485.37	35	2,011,988

Table 5.6: Cost summary for the concrete used in columns

	Total volume of concrete [m³]	Price per m³[Birr]	Total cost [Birr]
For reinforced concrete	715.36	2,700	1,931,472
For pre-cast prestressed system,	548.83	2,700	1,481,841

5.5.3 MATERIAL COST FOR GIRDER BEAM

Comparison regarding girder beam is not an exact or appropriate because here the research did not cover the expense regarding the connection details. Since the detailing is the major deal in the frame action comparison made regarding the material cost of the girder beam should be revised by doing further research on the connection detailing expenses. Material cost for the two systems can be compared by taking the quantity values computed in Annex B of this research. Concrete grade used for reinforced concrete systems is C25, which is concrete strength having 25Mpa of cube strength. Concrete grade used for pre-cast prestressed systems is C60, which is concrete strength having 60Mpa of cube strength. Table 5.7 and Table 5.8 below show the cost of steel and concrete respectively.

Table 5.7: Cost summary for the steel used in girder beams

	Total mass [kg]	Price per kg [Birr]	Total cost [Birr]
For reinforced concrete	238806.36	35	8,358,223
For pre-cast prestressed concrete reinforcement bar	29255.14	35	1,023,930
For pre-cast prestressed concrete prestressing strand	98866.22	43.75	4,325,397

Table 5.8: Cost summary for the concrete used in girder beams

	Total volume of concrete [m ³]	Price per m ³ [Birr]	Total cost [Birr]
For reinforced concrete	801.27	2,500	2,003,175
Formwork for the reinforced concrete [m ²]	7700.14	147	1,131,921
For pre-cast prestressed concrete	973.50	3,700	3,600,100

5.6 DISCUSSION

The total cost for the slab, girder beam and columns can be summarized in the Figure 5.3 below. For fair comparison between the pre-cast prestressed concrete and reinforced concrete 10% of the total cost of pre-cast prestressed is added to account transportation cost is provided. As it represent the total cost of the pre-cast prestressed slab as compared reinforced concrete slabs are lower. The system performance regarding quality is good. So the application of precast prestressed ribbed slabs for the housing project can serve as a positive solution in reducing time, reaching higher quality and maintaining low cost. Regarding the cost of the girder beam, such conclusion is hard to draw since the cost of connection detailing from beam to column s not addressed here. Although if a cost efficient method of connection is studied the longer span of girder beams are advantageous in a way that they provide architectural freedom and best quality performance. Finally the columns seem to cost more or less comparable value. The major advantage here is the column spacing which gives a wide room without any column interruptions.

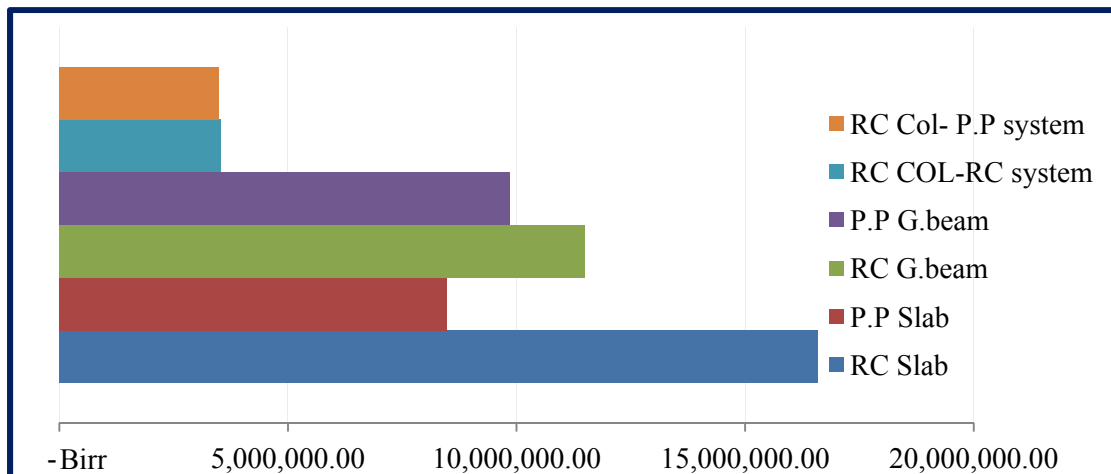


Figure 5.3: Cost summary

5.7 COMPARISON OF HOUSING PROJECTS USING REINFORCED CONCRETE AND PRE-CAST PRESTRESSED CONCRETE

Reinforced concrete

- Normal strength materials are used
- Lower value of section modifiers, full capacity of the section is not utilized
- Homogenous joints easily attained
- Cracks occurrence might not be controllable
- Dimension of members are higher as compared to prestressed unit.
- Structural layouts are restricted based on the maximum allowable of column dimensions and depth of beam.
- Similarity of structural members is optional
- Expensive equipment are less needed as compared to pre-cast prestressed units
- Skilled laborers' are less needed as compared to reinforced concrete system
- Formwork cost is higher than the pre-cast units.
- Joints are done with a lesser cost as compared to the pre-cast units
- Construction is done at the site
- Poor quality control
- Higher maintenance cost

Pre-cast prestressed concrete

- High strength materials are used
- Higher value of section modifiers, full capacity of the section is utilized
- Construction of joints might not be easy
- Occurrence of Cracks is controllable
- Member dimensions are reduced in a significant quantity.
- Layout flexibility,
- Similarity of structural members in order to be suitable for pre-cast industry is necessary
- Cranes and other machineries is mandatory
- Skilled laborers' are needed as compared to reinforced concrete system
- Formwork cost is reduced and moulds are used.
- Expensive detailing of joints
- Manufacturing site and mixing silos are required
- Quality control is easily achieved.
- High fire resistance property
- Expensive seismic detailing
- Minimum operation cost

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

The research intended to study the current housing projects in Ethiopia in view of cost effective design, timely delivery and use of high tech construction, to perform structural analysis and design of prestressed concrete and to economically compare the newly developed prestressed concrete system with the current construction systems used. The following conclusions and recommendations can be made regarding the results.

6.1 CONCLUSIONS

- By using Pre-cast prestressed concrete slab 49% of cost of reinforced concrete ribbed slab can be saved.
- By using Pre-cast prestressed concrete girder beam is 14% of cost of reinforced concrete beams can be saved without taking account price of connection expenses.
- Cost of columns using precast prestressed structural arrangement and reinforced concrete structural arrangement is similar.
- Application of the pre-cast prestressed ribbed slabs should be considered in this 40/60 housing projects.
- For housing projects development in assurance of quality plays a vital role in decreasing maintenance cost.
- Architectural design for pre-cast systems should be specially done hand in hand with the structural engineer to avoid uneconomical layouts and to ease the construction process.
- Reducing the time of the overall project will contribute to the cost for the housing project application of pre-cast units will have a say the speed of construction.
- One of the advantages of using pre-cast prestressed systems is the freedom of using large column spacing which in turn gives large room space without entrapment of columns, which is it gives design flexibility.
- Future hope of designing sophisticated architectural designs, ascetic flexibility will be visible. It will help the construction industry grow in an enormous rate.

6.2 RECOMMENDATIONS

- Partial application of pre-cast prestressed systems should also be considered to improve the industry and specifically the quality and production time of the housing projects step by step.
- Visibility of these research should be proceed to study the industrial capacity of the prestressed company at hand to incorporate manufacturing of pre-cast prestressed units for the housing projects.
- Global response of the pre-cast prestressed frame system should be analyzed in detail.
- Effect of adopting no moment resisting frame system should also be investigated.
- The design and cost comparison of lateral wall systems should be investigated
- Application of pre-cast prestressed systems on a cantilever system should be studied.
- Studies regarding the effect of changing the planer H shape and removing the expansion joints of the building on economic analysis should be done to see the effect of choice architectural design on the cost of the housing project. .
- Detail research regarding design and economic analysis of the connections should be studied.
- Performance of connection in earthquake should be specifically considered and analyzed.
- Adopting precast units for wall should also be further investigated to reduce the time for these housing projects.
- Effect of pre-cast panels along with steel frames for the stairs should be studied.
- Different options regarding cost effectiveness with different grades of concrete should be assessed.
- Study on using pretensioned cast-insitu concrete should be done in order to decrease the usage of expensive connection details.
- Different options regarding cost effectiveness with method of application of prestress force that is partial or full application of prestressing should be assessed.
- Architectural design should be specially done hand in hand with the structural engineer to avoid uneconomical layouts and to ease the construction process, and further study on the economic aspect of the architectural design of this housing project should be done.

7. REFERENCES

- [1]. American concrete institute standard, ACI 318M-11, Building code requirements for structural concrete and commentary, September 2011
- [2]. Ahmed Baharuddin, Abd. Rahamn, Dennis Chan Pal Leong, A.Aziz Saim, Mohd.Hanim Osman, Hybrid beam to column connections for precast concrete frames, Proceedins of the 6th Asia-Pacific Structural engineering and construction conference (APSEC 2006), Kuala Lumpur, Malaysia, 5-6 September
- [3]. A.J Bond, O Brooker, A.J Harris, T.Harrison, R.M.Moss, R.S.Narayanan, R.Webster, How to design concrete structures using Eurocode 2, A cement and concrete industry publication, December 2006
- [4]. Amlan K.Segupta,PhD PE, Prestressed concrete structures, lecture note, Department of civil engineering, Indian institute of technology Madras
- [5]. Anthony Rochefort, Analysis and design of prestress concrete members to Eurocodes (EC2) considering the temperature, Dissertation submitted in total fulfillment of the requirements of the degree of Msc structural engineering, School of engineering and the building environment Napier university, September 2008
- [6]. Arcelor Mittal, prestressed concrete wire and stands specification manual, 2010
- [7]. Asnake Adamu ,PhD, Prestressed concrete structures, lecture note, Department of civil and environmental engineering, Addis Ababa institute of technology, Addis Ababa, Ethiopia
- [8]. A.W.Beeby and R.S.Narayanan, Designers' guide to EN1992-1-1 and EN1992-1-2 Eurocode 2: Design of concrete structures general rules for buildings and structural fire design, Thomas telford Ltd, London, 2009
- [9]. CSI educational services, From start to finish: Model, design and optimize a multi-story concrete structures using ETABS, University Avenue Berkeley, California, United states of America, 1995
- [10]. Dr. P. Ritz, P. Matt, Ch. Tellenbach, P. Schlub and H. U. Aeberhard, Post-tensioned Slabs ,4.2 report series, VSL International LTD, Berne, Switzerland, 1985
- [11]. European standard, Eurocode 1: Actions on structures-part 1-1: General ations densities, self-weight, imposed loads for buildings, prEN 1991-1-1, July 2001
- [12]. European standard, Eurocode 2: Design of concrete structures- part 1: General rules and rules for buildings, prEN 1992-1-1, November 2002
- [13].Evan Bentz and Machael P.Collins, Response-2000 Shell-2000 Triax-2000 Membrane-2000 User manual, September 2001

- [14]. Group of the New Zealand concrete society and the New Zealand society for Earthquake Engineering, Guidelines for the use of structural precast concrete in buildings, 2nd edition, Published by Center for advanced engineering, New Zealand, 1999
- [15]. Hossein Parastesh, Iman hajirasouliha, Reza Ramezani, Anew ductile moment-resisting connection for precast concrete frames in seismic regions: an experimental investigation, White Rose research, 2014
- [16]. [http:// www.eiabc.edu.et](http://www.eiabc.edu.et)
- [17]. Hubert Bachmann, Alfred Steinle, Precast concrete structures, Ernst and Sohn a Wiley Company, Berlin Germany, 2011
- [18]. Irish precast concrete association IPCA, Precast concrete frames guide, Newlands Business park, Clondalkin, Dublin 22
- [19]. L.F. Maya and L. Albajar, Beam- column connection for precast concrete frames using high performance fiber reinforced cement composites, Department of continuum mechanics and structures, Spain
- [20]. L.Palmieri and C.French, Performance of ductile connection in precast concrete frames, Eleventh world conference on earthquake engineering, Department of civil engineering, university of Minnesota, paper No.15, ISBN:0080428223, Minneapolis, MN, unites states of America
- [21]. Mekonen Webe Ermed, Addis Ababa Integrated Housing Development Program: A strategy for Urban Poverty Reduction and Sustainable Socio-Economic Transformation, presentation, Addis Ababa, Ethiopia
- [22]. Michael P.Collins, Denis Mitchell, Prestressed concrete structures, Response publication, Canada, 1997
- [23]. N Krishhna Raju, Prestressed concrete, 4th edition, The Mc-Graw-Hill publishing company limited, New Delhi, 2007
- [24]. PCI industry handbook committee, PCI design handbook precast and prestressed concrete, 5th edition, printed in United states of America, 1999
- [25]. Platform European concrete, Eurocode 2 worked examples, Published by the European concrete platform ASBL, May 2008
- [26]. Radomir Folic, Damir Zennovic, Nesib Residbegovic, Strength of connections in precast concrete structures, series: Faculty of technical sciences, Architectural and civil engineering Vol.9, No.2, Novi Sad, serbia, 2011
- [27]. Robert Benaim, The design of prestressed concrete bridges concepts and principles, Taylor and Francis, London and New York, 2008

- [28]. Standing committee on structural engineering of the federal construction council building research advisory board division of engineering National research council, Expansion joints in buildings, Technical report No.65, National Academy of sciences, Washington, D.C, 1974
- [29]. Suzanne Dow Nakaki S.E, Robert E Englekirk Ph.D S.E, Juergen L.Plaehn P.E, Ductile connectors for precast concrete frame, PCI journal, September - October 1994
- [30]. Teresa Guevara -Perez, Precast concrete construction, Svetlana Brzev, British Columbia institute of technology, Canada
- [31]. The institution of structural engineers, The institution of Civil engineers, Manual for the design of reinforced concrete building structures to EC2, published for the institution of structural engineers, March 2000
- [32]. T.Y.Lin, Ned H.Burns, Design of prestressed concrete structures, 3rd edition, John Wiley and sons, United states of America, 1981

ANNEX-A
DESIGN FIGURES AND TABLES

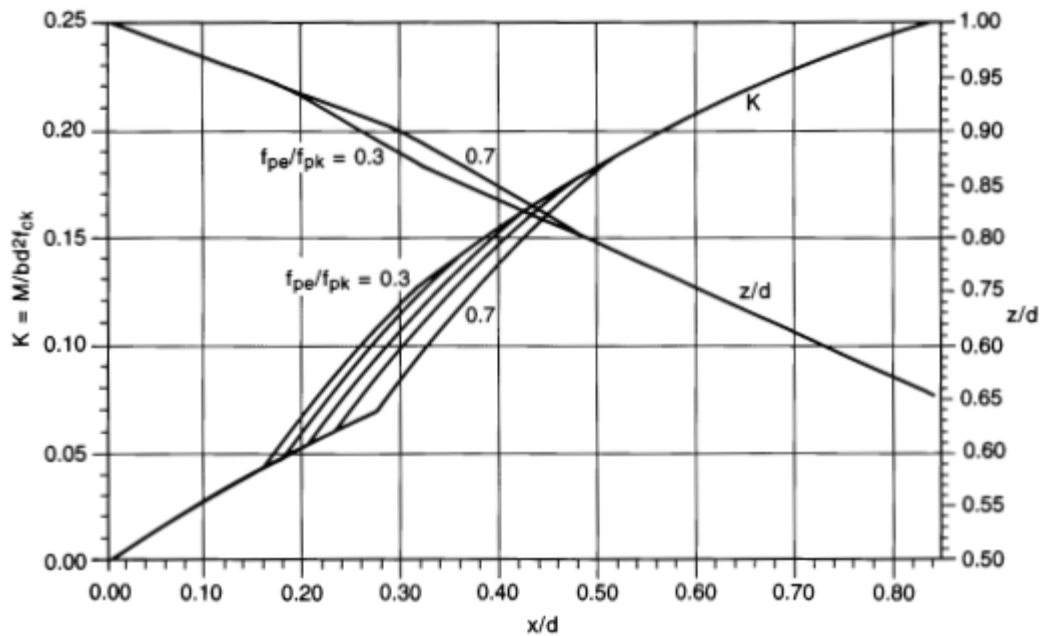


Figure A1: Neutral axis depth and lever arm factors for prestressed concrete rectangular sections with bonded tendons [10]

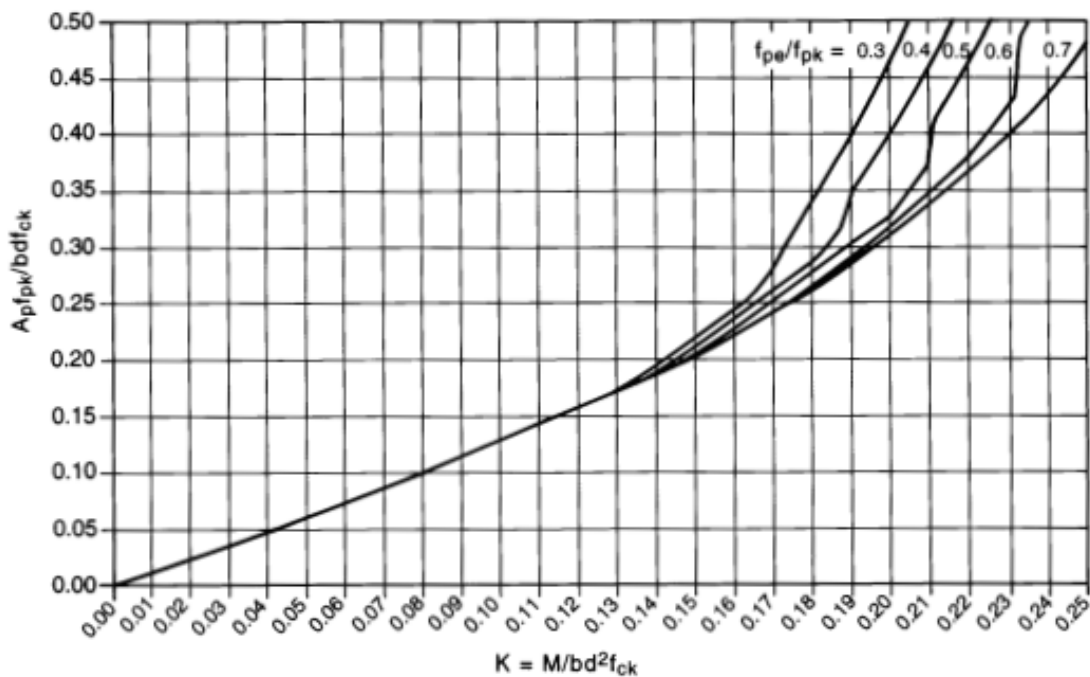


Figure A2: Design chart for prestressed concrete rectangular sections with bonded tendons [10]

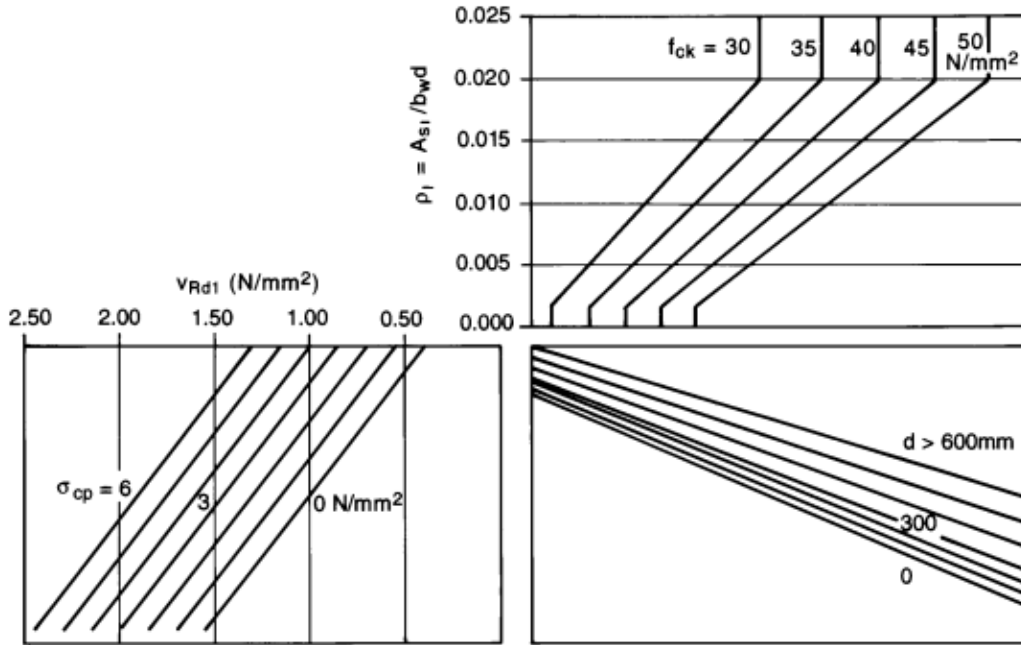


Figure A3: Design chart to determine v_{Rd1} for prestressed concrete [10]

Table A1: limiting values of x/d [10]

percentage of moment distribution, %	0	5	10	15	20	25	30
$f_{ck} \leq 35\text{N/mm}^2$	0.45	0.41	0.37	0.33	0.29	0.25	0.21
$f_{ck} \geq 40\text{N/mm}^2$	0.35	0.31	0.27	0.23	0.19	0.15	0.11

Table A2: Minimum member sizes and covers for initial design of prestressed members [10]

Member	Minimum dimension, mm				
	for fire resistance of		4h	2h	1h
Beams	width		280	200	200
	Cover	simply supported	90	60	45
		continuous	80	45	45
Plain soffit solid slab and flanges of T-section beams and slabs	Depth (including non-combustible finishes)		170	125	100
	Cover	simply supported	65	40	30
		continuous	55	35	30
Rib of T section slabs with no stirrups	Width of ribs		200	150	110
	Cover	simply supported	75	55	35
		continuous	65	45	35

Table A3: Minimum shear reinforcement ratio, ρ_w [10]

Concrete class	Ratio, ρ_w	
	$f_{yk}=460\text{N/mm}^2$	$f_{yk}=250\text{N/mm}^2$
C30/37 to C35/45	0.0012	0.0022
C40/50 to C50/60	0.0015	0.0028

Table A4: Maximum spacing of shear reinforcement [10]

Design shear force, V_{sd}	Longitudinal spacing, [mm]	Transverse spacing, [mm]
$\leq 0.2 V_{Rd2 \text{ red}}$	$0.8d \leq 300$	$d \leq 800$
$> 0.2 V_{Rd2 \text{ red}}$ and $\leq 0.67 V_{Rd2 \text{ red}}$	$0.6d \leq 300$	
$> 0.67 V_{Rd2 \text{ red}}$	$0.3 \leq 200$	

Table A5: Span/effective depth ratio for initial design of beams [10]

Cantilever	8
Simply supported	18
Continuous	22

ANNEX-B
MANUAL CALCULATION

ANNEX B.1 MANUAL CALCULATION FOR T-SECTION TYPE 2, 3 AND 4 SLAB.

For transfer at 7 days, the compressive strength of concrete for C50 is;

$$f_{ck} = 36 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable compressive concrete stress at transfer, f_{max}^* , is;

$$f_{max}^* = 18 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable tensile concrete stress at transfer, f_{min}^* , is

$$f_{min}^* = -3.5 \text{ N/mm}^2$$

For serviceability state, the compressive strength of concrete, f_{ck} , is

$$f_{ck} = 40 \text{ N/mm}^2$$

From part allowable stresses, the maximum allowable concrete stress allowed under service is,

$$(f_{max})_s = 24 \text{ N/mm}^2$$

$$\text{Taking : } \alpha = 1 - 0.04 = 0.96 \text{ and } \beta = 1 - 0.21 = 0.79$$

$$\gamma = 24 \text{ kN/m}^3$$

▪ **Design loads for the prestressed concrete beams for Type -2**

The only applied loading at transfer is the self weight which is (density of concrete) \times (area). Hence;

$$\text{self weight} = \gamma A_c = 24 \times 125.2 \times 10^3 \times 10^{-6} = 3.0 \text{ kN/m}$$

The maximum moment due to this loading is: transfer moment,

$$M_t = 3.0 (8.3)^2 / 8 = 25.83 \text{ kNm}$$

Uniformly distributed load due to self-weight of the beam and other dead loads per meter width, w_o

$$w_o = \gamma A_c + w_d = (24 \times 125.2 \times 10^3 \times 10^{-6}) + 5.2 = 8.2 \text{ kN/m}$$

The total loading at SLS is this plus the imposed loading, i.e.: SLS moment,

$$M_s = (8.2 + 1.76)(8.3)^2 / 8 = 85.77 \text{ kNm}$$

▪ **Elastic sectional moduli**

From inequalities 2.15 and 2.16 one can obtain the elastic section moduli. Required about the top and bottom fibers, Z_t and Z_b , as

$$Z_t \geq \frac{\alpha M_s - \beta M_t}{\alpha (f_{max})_s - \beta f_{min}^*} = \frac{0.96 * 85.77 * 10^6 - 0.79 * 25.83 * 10^6}{0.96 * 24.00 - 0.79 * (-3.5)} = \frac{61.93 * 10^6}{25.805} = 2.40 * 10^6 \text{ mm}^3 (< Z_t = 9.10 * 10^6 \text{ mm}^3)$$

=> Ok!

$$Z_b \geq \frac{\alpha M_s - \beta M_t}{\beta f_{max}^* - \alpha f_{min}^*} = \frac{0.96 * 85.77 * 10^6 - 0.79 * 25.83 * 10^6}{0.79 * 18 - 0.96 * (-3.5)} = \frac{61.93 * 10^6}{17.58} = 3.52 * 10^6 \text{ mm}^3 (< Z_b = 3.84 * 10^6 \text{ mm}^3)$$

=> Ok!

Table B.1: Geometric property

Label	Type 1
Area [mm ²]*10 ³	125.2
Inertia[mm ⁴]*10 ⁶	864.7
y _t [mm]	95
y _b [mm]	225
Z _t [mm ³]*10 ³	9104.1
Z _b [mm ³]*10 ³	3843.0

▪ **Determination of prestress forces and eccentricity**

From inequality 2.18 a,

$$\frac{1}{P_o} \leq \frac{\alpha \left(\frac{Z_t}{A_c} - e \right)}{(Z_t f'_{min} - M_t)} = \frac{0.96 \left(\frac{9.1041 \cdot 10^6}{125200} - e \right)}{(9.1041 \cdot 10^6 \cdot (-3.5) - 25.83 \cdot 10^6)} = \frac{0.96(72.71 - e)}{-57.69 \cdot 10^6} = \frac{(121.0 - 1.66e) \cdot 10^{-8}}{-1} \frac{1}{N}$$

Note that the denominator is negative. Dividing both sides of an inequality by a negative number has effect of changing the sign of the inequality. Thus the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq 1.66e - 121.0 \dots\dots (i)$$

From inequality 2.18 b,

$$\frac{1}{P_o} \geq \frac{\alpha \left(\frac{Z_b}{A_c} + e \right)}{(Z_b f'_{max} + M_t)} = \frac{0.96 \left(\frac{3.843 \cdot 10^6}{125200} + e \right)}{(3.843 \cdot 10^6 \cdot (18) + 25.83 \cdot 10^6)} = \frac{0.96(30.69 + e)}{95.00 \cdot 10^6} = \frac{(37.08 + 1.01e) \cdot 10^{-8}}{1} \frac{1}{N}$$

Thus, the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq 1.01e + 37.08 \dots\dots (ii)$$

From inequality 2.18c,

$$\frac{1}{P_o} \geq \frac{\beta \left(\frac{Z_t}{A_c} - e \right)}{(Z_t (f_{max})_s - M_s)} = \frac{0.79 \left(\frac{9.104 \cdot 10^6}{125200} - e \right)}{(9.104 \cdot 10^6 \cdot (24) - 85.77 \cdot 10^6)} = \frac{0.79(72.72 - e)}{132.726 \cdot 10^6} = \frac{(43.28 - 0.6e) \cdot 10^{-8}}{1} \frac{1}{N}$$

Thus, the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq -0.60e + 43.28 \dots\dots (iii)$$

From inequality 2.18 d,

$$\frac{1}{P_o} \leq \frac{\beta\left(\frac{Z_b}{A_c} - e\right)}{(Z_{bf\ min} - M_s)} = \frac{0.79\left(\frac{3.843 \cdot 10^6}{125200} - e\right)}{(3.843 \cdot 10^6 \cdot (-3.5) - 85.77 \cdot 10^6)} = \frac{0.79(30.69 - e)}{-99.22 \cdot 10^6} = \frac{(24.43 - 0.8e) \cdot 10^{-8}}{-1} \frac{1}{N}$$

Note that the denominator is negative. Dividing both sides of an inequality by a negative number has effect of changing the sense of the inequality. Thus, the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq 0.8e - 24.43 \dots\dots (iv)$$

Now putting all the four inequalities together:

$$\frac{10^8}{P_o} \geq 1.66e - 121.0 \dots\dots\dots (i)$$

$$\frac{10^8}{P_o} \geq 1.01e + 37.08 \dots\dots\dots (ii)$$

$$\frac{10^8}{P_o} \geq -0.60e + 43.28 \dots\dots\dots (iii)$$

$$\frac{10^8}{P_o} \geq 0.8e - 24.43 \dots\dots\dots (iv)$$

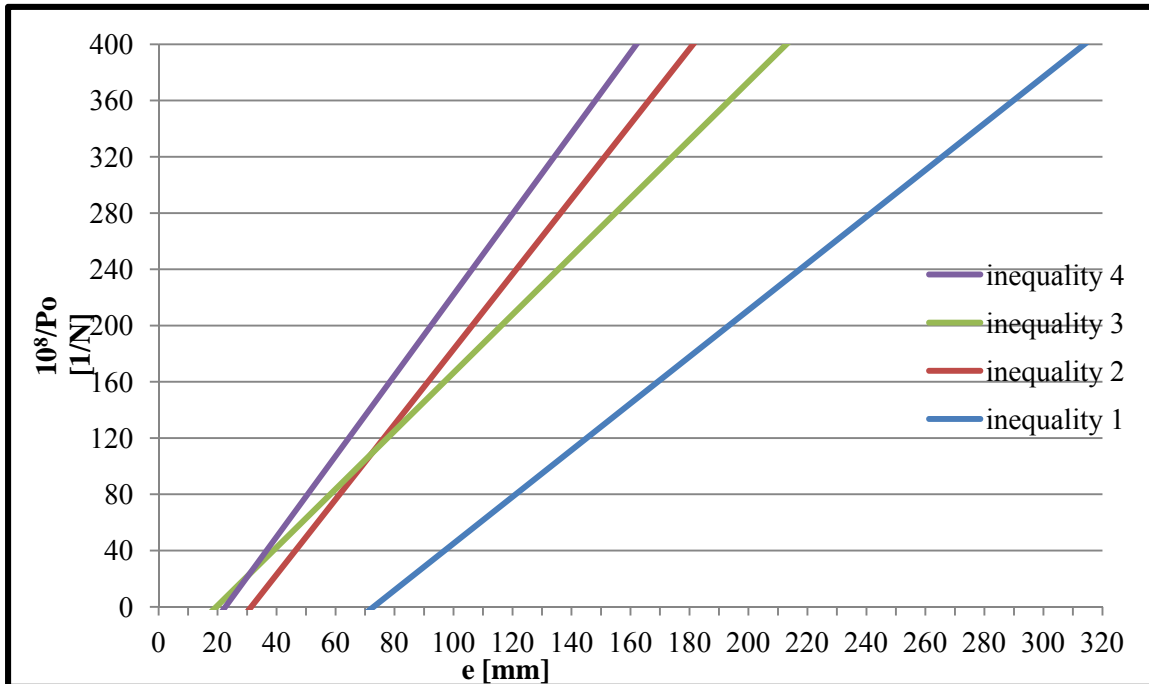


Figure B.1: Magnel diagram for type 2

In this case, the distance from the neutral axis to the soffit is 147mm. if we logically take the distance from the center of prestressing steel to the soffit as 80mm taking in to account the cover requirement, the maximum value of eccentricity for the permissible zone is;

$$e = 225 - 80 = 145 \text{ mm}$$

The corresponding prestressing force will be obtained by;

$$\frac{10^8}{P_o} \geq 1.66e - 121.0 = 1.66 * 145 - 121.0 = 119.7 \text{ 1/N}$$

$$P_o \leq 10^8 / 119.7 = 835.42 \text{ KN}$$

$$\frac{10^8}{P_o} \geq 1.01e + 37.08 = 1.01 * 145 + 37.08 = 183.53 \text{ 1/N}$$

$$P_o \leq 10^8 / 183.53 = 544.870 \text{ KN}$$

$$\frac{10^8}{P_o} \geq -0.60e + 43.28 = -0.6 * 145 + 43.28 = -130.28 \text{ 1/N}$$

$$P_o \leq 10^8 / -130.28 = -767.58 \text{ KN}$$

$$\frac{10^8}{P_o} \geq 0.8e - 24.43 = 0.8 * 145 - 24.43 = 91.57 \text{ 1/N}$$

$$P_o \leq 10^8 / 91.57 = 1092.06 \text{ KN}$$

Thus; $P_o \leq 544.870 \text{ KN}$ and $P_o = 545 \text{ KN}$ can be chosen because which satisfies the solution set.

- **Selection of prestressing steel**

According to Euro code manual, when checking the stresses at transfer, the prestressing force required for the in-service condition should be increased by 60%, as the long-term (time-independent) losses will not have occurred, and no tension should be allowed in concrete.

$$P_{\text{req}} = P_o / 0.6 = 545 / 0.6 = 908.33 \text{ kN}$$

Here one assume 4No.7-wire super strands, so the required characteristic load per strand will be:

$$P_{\text{req}} / 3 = 908.33 / 3 = 302.78 \text{ kN/ strand}$$

From steel manufacturer specification manual selecting 3 No-7- wire drawn strands of 15.2mm nominal diameter with $P_o = 303 \text{ kN /strand}$ with nominal strength $f_{pk} = 1860 \text{ N/mm}^2$ and cross sectional area of $A_p = 139.0 \text{ mm}^2$.

The total prestress force P_u is calculated as;

$$P_u = 303 * 3 = 909 \text{ kN}$$

The total area of the prestressing steel A_{pu} is calculated as;

$$A_{pu} = A_p * 3 = 139.0 \text{ mm}^2 * 3 = 417 \text{ mm}^2$$

The actual prestress force P_o is now becomes

$$P_o = 0.6 * P_u = 0.6 * 909 = 545.4 \text{ kN}$$

▪ **Calculation of losses**

Before checking the concrete stress at transfer and service the estimated loss should be calculated to get a fairly accurate result.

▪ **Short time losses (Immediate losses)**

○ **Elastic shortening:**

The stress in the prestressing steel due to elastic shortening is given by equation 2.37 as:

$$\begin{aligned} \sigma_{ci} &= \frac{f_{po}}{m + \frac{A_c}{A_p(1 + \frac{e^2}{r^2})}} - \frac{M_o e}{I_c} = \frac{1307.91 \frac{\text{N}}{\text{mm}^2}}{5.86 + \frac{125.2 * 10^3 \text{ mm}^2}{417 \text{ mm}^2 (1 + \frac{145^2}{83.09^2})}} - \frac{25.83 * 10^6 \text{ Nmm} * 140 \text{ mm}}{864.7 * 10^6 \text{ mm}^4} \\ &= \frac{1307.91 \frac{\text{N}}{\text{mm}^2}}{5.86 + 80.94} - 4.18 \frac{\text{N}}{\text{mm}^2} = 10.89 \frac{\text{N}}{\text{mm}^2} \end{aligned}$$

Where:

$$f_{po} = P_o / A_p = 545.4 * 10^3 \text{ N} / 417 \text{ mm}^2 = 1307.91 \text{ MPa}$$

$$m = E_s / E_{cm} = 205 \text{ GPa} / 35 \text{ GPa} = 5.86$$

$$r = (I_c / A_c)^{0.5} = (864.7 * 10^6 \text{ mm}^4 / 125.2 * 10^3 \text{ mm}^2)^{0.5} = 83.09 \text{ mm}$$

$$e = 145 \text{ mm}$$

The prestress force loss due to elastic shortening is then computed as;

$$\Delta P_{el} = m * \sigma_{ci} * A_p = 5.86 * 10.89 * 417 = 26611.02 \text{ N} = 26.61 \text{ kN}$$

Therefore the percentage loss due to elastic shortening is 4.89%

- **Long time losses (Differed losses)**
 - **Shrinkage**

To compute the prestress loss due to shrinkage, the value of concrete shrinkage after tensioning ϵ_r must be known to get $\Delta\epsilon_r$ because once $\Delta\epsilon_r$ is known using equation 2.38 and 2.39 the value of the stress can be computed.

$$\epsilon_r = \epsilon_{cd,\infty} * \frac{j}{j+0.04*h_o^{1.5}} + \epsilon_{ca,\infty} * 1 - \exp(-0.2 * \sqrt{j}) \quad (2.41)$$

ϵ_{cd} according to Euro Code 2, EN 1992-1-1 section 3.1.4 is 0.46‰

$$\epsilon_{ca,\infty} = 2.5 * (f_{ck} - 10) * 10^{-4} = 2.5 * (40 - 10) * 10^{-6} = 7.5 * 10^{-5}$$

$$j = 50 \text{ years} = 18250 \text{ days}$$

$$h_o = \frac{2A_c}{U} = \frac{2 * 125.2 * 10^3 \text{ mm}^2}{[160 + (2 * 220) + (2 * 100) + 900 + (2 * 370)] \text{ mm}} = 102.62 \text{ mm}$$

$$\begin{aligned} \epsilon_{r,50\text{Yrs}} &= 0.46 * 10^{-3} * \frac{18250}{18250 + 0.04 * 102.62^{1.5}} + 7.5 * 10^{-5} * 1 - \exp(-0.2 * \sqrt{18250}) \\ &= 4.59 * 10^{-4} + 7.5 * 10^{-5} = 5.34 * 10^{-4} \end{aligned}$$

$$\begin{aligned} \epsilon_{r,28\text{days}} &= 0.46 * 10^{-3} * \frac{28}{28 + 0.04 * 102.62^{1.5}} + 7.5 * 10^{-5} * 1 - \exp(-0.2 * \sqrt{28}) \\ &= 1.85 * 10^{-4} + 4.9 * 10^{-5} = 2.34 * 10^{-4} \end{aligned}$$

$$\Delta\epsilon_r = \epsilon_{r,50\text{Yrs}} - \epsilon_{r,28\text{days}} = (5.34 * 10^{-4}) - (2.34 * 10^{-4}) = 3.0 * 10^{-4}$$

$$\text{Using Equation 2.21, } \Delta P_p = -A_p E_p \Delta\epsilon_r = -417 \text{ mm}^2 * 205 * 10^3 \frac{\text{N}}{\text{mm}^2} * 3.0 * 10^{-4}$$

$$\Delta P_p = -25645.5 \text{ N} = -25.64 \text{ kN}$$

Therefore the percentage loss due to shrinkage is 4.70%

○ **Stress relaxation**

According to the specification manual attached in Annex c of the prestressing tendon, the maximum relaxation loss is given to be 2.5%.

○ **Creep**

To calculate the deformation due to the sustained compressive load which further facilitates the computation of prestress loss due to creep, first the stress σ_c should be computed,

$$\sigma_c = \frac{P}{A_c} + \frac{P * e_o^2}{I} + \frac{M * e_o}{I}$$

$$\sigma_c = \frac{545.4 * 10^3 N}{125.2 * 10^3 mm^2} + \frac{545.4 * 10^3 N * 145^2 mm^2}{864.7 * 10^6 mm^4} + \frac{-25.83 * 10^6 Nmm * 145 mm}{864.7 * 10^6 mm^4}$$

$$= 4.36 \frac{N}{mm^2} + 13.26 \frac{N}{mm^2} - 4.33 \frac{N}{mm^2} = 13.29 MPa$$

$$0.45f_{ck} = 0.45 * 40 MPa = 18 MPa > \sigma_c$$

i.e. the creep strain can be computed using equation 2.25; $\epsilon_{cc}(\infty, t_o) = \varphi(\infty, t_o) \cdot \left(\frac{\sigma_c}{E_c}\right)$ where the creep coefficient can be read from the Figure 3.1 provided in Euro code 2 EN 1991-1-1

The value of $\varphi(\infty, t_o)$ from the graph is then; $\varphi(\infty, t_o) = 2$

The creep strain can then be computed as; $\epsilon_{cc}(\infty, t_o) = \varphi(\infty, t_o) \cdot \left(\frac{\sigma_c}{E_c}\right) = 2 * \left(\frac{13.29}{35 * 10^3}\right) = 7.59 * 10^{-4}$

The stress due to the creep stain is then evaluated as; $\Delta\sigma_p = E_p \epsilon_{cc}$

$$= 205 * 10^3 N/mm^2 * 7.59 * 10^{-4} = 155.60 N/mm^2$$

The loss in the prestress force is $\Delta P_p = \Delta\sigma_p A_p = 155.60 N/mm^2 * 417 mm^2 = 64885.2 N = 64.88 kN$

Therefore the percentage loss due to Creep is 11.89%

Table B.2: Percentage losses calculation results for type 3.

TYPE OF LOSSES	PERCENTAGE LOSS OF STRESS
	TYPE-3
Elastic shortening	4.89
Creep of concrete	11.89
Shrinkage of concrete	4.70
Stress relaxation	2.50
Short time losses [%]	4.89
Long time losses [%]	19.09

P_0 is computed taking: $\alpha = 1 - 0.04 = 0.96$ and $\beta = 1 - 0.21 = 0.79$

Now $\alpha = 1 - 0.0489 = 0.951$ and $\beta = 1 - 0.191 = 0.809$

The assumed valued and the computed losses are comparatively accurate.

▪ **Concrete stress at transfer**

From equation 2.13 a, the stress at the top fiber f_t is calculated as;

$$f'_t = \frac{\alpha P_0}{A_c} - \frac{\alpha P_0 e}{Z_t} + \frac{M_o}{Z_t}$$

$$f'_t = \frac{0.951 * 545.4 * 10^3 N}{125.2 * 10^3 mm^2} - \frac{0.951 * 545.4 * 10^3 N * 145 mm}{9104.1 * 10^3 mm^3} + \frac{25.83 * 10^6 Nmm}{9104.1 * 10^3 mm^3}$$

$$f'_t = 4.14 \frac{N}{mm^2} - 8.26 \frac{N}{mm^2} + 2.83 \frac{N}{mm^2}$$

$$f'_t = -1.29 \frac{N}{mm^2} > f'_{min} = -3.5 \frac{N}{mm^2} \text{ OK!}$$

From equation 2.13 b, the stress at the bottom fiber f_b is calculated as;

$$f'_b = \frac{\alpha P_0}{A_c} + \frac{\alpha P_0 e}{Z_b} - \frac{M_o}{Z_b}$$

$$f'_b = \frac{0.951 * 545.4 * 10^3 N}{125.2 * 10^3 mm^2} - \frac{0.951 * 545.4 * 10^3 N * 145 mm}{3843.0 * 10^3 mm^3} + \frac{25.83 * 10^6 Nmm}{3843.0 * 10^3 mm^3}$$

$$f'_b = 4.14 \frac{N}{mm^2} + 19.57 \frac{N}{mm^2} - 6.72 \frac{N}{mm^2}$$

$$f'_b = 16.99 \frac{N}{mm^2} < f'_{max} = 18 \frac{N}{mm^2} \text{ OK!}$$

Concrete stress at service

From equation 2.13c, the stress at the top fiber f'_t is calculated as;

$$f'_{t\text{service}} = \frac{\beta P_0}{A_c} - \frac{\beta P_0 e}{Z_t} + \frac{M_s}{Z_t}$$

$$f'_{t\text{service}} = \frac{0.809 * 545.4 * 10^3 N}{125.2 * 10^3 mm^2} - \frac{0.809 * 545.4 * 10^3 N * 145 mm}{9104.1 * 10^3 mm^3} + \frac{85.77 * 10^6 Nmm}{9104.1 * 10^3 mm^3}$$

$$f'_{t\text{service}} = 3.52 \frac{N}{mm^2} - 7.03 \frac{N}{mm^2} + 9.42 \frac{N}{mm^2}$$

$$f'_{t\text{service}} = 5.91 \frac{N}{mm^2} < (f_{max})_{serv} = 24 \frac{N}{mm^2} \text{ OK!}$$

From equation 2.13 d, the stress at the top fiber f'_b is calculated as;

$$f'_{b\text{service}} = \frac{\beta P_0}{A_c} + \frac{\beta P_0 e}{Z_b} - \frac{M_s}{Z_b}$$

$$f'_{b\text{service}} = \frac{0.809 * 545.4 * 10^3 N}{125.2 * 10^3 mm^2} + \frac{0.809 * 545.4 * 10^3 N * 145 mm}{3843.0 * 10^3 mm^3} - \frac{85.77 * 10^6 Nmm}{3843.0 * 10^3 mm^3}$$

$$f'_{b\text{service}} = 3.52 \frac{N}{mm^2} + 16.65 \frac{N}{mm^2} - 22.32 \frac{N}{mm^2}$$

$$f'_{b\text{service}} = -2.15 \frac{N}{mm^2} > f'_{min} = -3.5 \frac{N}{mm^2} \text{ Ok!}$$

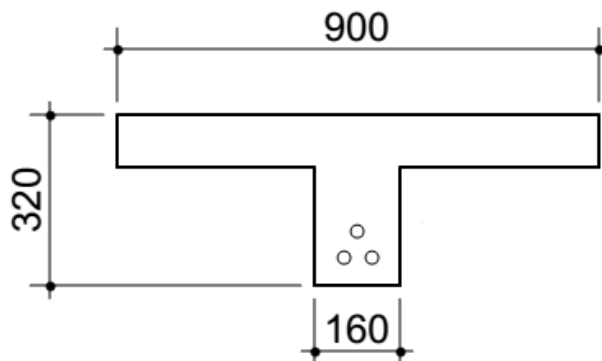


Figure B.2: Cross section for Type 2

▪ **Bending Moment resistance**

The initial stress in tendons according to Euro code 2 EN 1992-1-1:2004 is

$$f_{po} = 0.7f_{pk} = 0.7 * 1860 \text{ N/mm}^2 = 1302 \text{ N/mm}^2$$

The elastic modulus of steel is $E_p = 205 \text{ kN/mm}^2$

Using the procedure for design of flanged beam

- $f_{po}/f_{pk} = 0.7$
- $K = M/bd^2f_{ck} = [((1.3*8.2)+(1.6*1.76))*8.3^2/8]*10^6 / (900*240^2*40) = 0.056$
- Using figure A1 in Annex A, $x/d = 0.24$ which implies that $x = 57.6 \text{ mm}$. Checking the value of x/d with the limiting value obtained from Table A1, $0.24 < 0.35 \Rightarrow \text{ok!}$
- $0.8x \leq h_f$, $46.08 \text{ mm} \leq 100 \text{ mm} \Rightarrow A_p$ is determined as for a rectangular beam of breadth b
- Using figure A2 in annex A, the ratio of $K_{lim} = A_p f_{pk} / bdf_{ck} = 0.075$ and $K = A_p f_{pk} / bdf_{ck} = 417 * 1860 / (900 * 240 * 40) = 0.0897$ i.e. $K > K_{lim}$ which means compression reinforcement is required.
- Where $d' = 40 + 6 = 46$
 $d' > \left(1 - \frac{f_{ck}}{800}\right)x = \left(1 - \frac{40}{800}\right)46.08 = 43.776$, use $700 \left(1 - \frac{d'}{x}\right) = 700 \left(1 - \frac{46}{46.08}\right) = 1.22$ in lieu of $0.87f_{yk}$
- $A'_s = \frac{M - K_{lim} b d^2 f_{ck}}{1.22 f_{yk} (d - d')} = \frac{116.04 * 10^6 - 0.075 * 900 * 240^2 * 40}{1.22 * 460 (240 - 46)} = 362.62 \text{ mm}^2$
 Use $4\phi 12$ c/c 170mm

Minimum reinforcement

The minimum area of longitudinal reinforcement in order to avoid brittle failure should not be less than either $0.6b_t d / f_{pk}$ or $0.0015b_t d$.

- $0.6b_t d / f_{pk} = 7.122$ and $0.0015b_t d = 33.12$ which in both case is satisfied.

Providing minimum area of compression reinforcement for lifting and placing of the T-sections into their position: use $5\phi 8$ c/c 200mm

Tendon spacing

According to manual for the design of reinforced concrete building structures to EC2,

- Distance between the pretensioned tendons is taken as 25mm

- Clear vertical distance between tendons is taken as 25mm
- According to Table A2 of Annex A the cover requirement is fulfilled

Shear design

$$\text{self weight} = \gamma A_c = 24 \times 125.2 \times 10^3 \times 10^{-6} = 3.00 \text{ kN/m}$$

The maximum moment due to this loading is: transfer moment,

$$M_t = 3.00 (8.3)^2 / 8 = 25.83 \text{ kNm}$$

Uniformly distributed load due to self-weight of the beam and other dead loads per meter width, w_o

$$w_o = \gamma A_c + w_d = (24 \times 125.2 \times 10^3 \times 10^{-6}) + 5.2 = 8.2 \text{ kN/m}$$

The total loading at SLS is this plus the imposed loading, i.e.: SLS moment,

$$M_s = (8.20 + 1.76)(8.3)^2 / 8 = 85.77 \text{ kNm}$$

Total loading at ultimate limit state, w_{ult}

$$w_{ult} = 1.3 * 8.20 + 1.6 * 1.76 = 13.48 \text{ kN/m}$$

The maximum moment due to this loading is: ultimate moment,

$$M_{ult} = 13.48 (8.3)^2 / 8 = 116.04 \text{ kNm}$$

Design ultimate shear force, V_{sd}

$$V_{sd} = w_{ult} * (L/2 - d) = 13.48 * (8.3/2 - 0.240) = 52.71 \text{ kN}$$

$$V_{Rd2,red} = 1.67 V_{Rd2} \left(\frac{1 - 1.5 \sigma_{cp,eff}}{f_{ck}} \right) \leq V_{Rd2}$$

$$V_{Rd2} = 0.15 f_{ck} b_{w,nom} d = 0.15 * 40 \frac{N}{mm^2} * 160 \text{ mm} * 240 \text{ mm} = 230400 \text{ N} = 230.4 \text{ kN}$$

$$b_{w,nom} = 160 \text{ mm}$$

$$\sigma_{cp,eff} = 1.2 P_o / A_c = 1.2 * 545.4 * 10^3 \text{ N} / 125.2 * 10^3 \text{ mm}^2 = 5.23 \text{ MPa}$$

$$V_{Rd2,red} = 1.67 * 230.4 * 10^3 \text{ N} \left(\frac{1 - 1.5 * 5.23 \text{ MPa}}{40 \text{ MPa}} \right) \leq V_{Rd2}$$

$$V_{Rd2,red} = -65.84 \text{ kN} \leq 264 \text{ kN}$$

$V_{Rd1} = v_{Rd1} b_w d$ referring figure A3 in annex A the value of $v_{Rd1} = 1.66 \text{ N/mm}^2$

$$V_{Rd1} = 1.66 \frac{N}{mm^2} * 160 \text{ mm} * 240 \text{ mm} = 63.74 \text{ kN}$$

$V_{sd} < V_{Rd1}$, design for shear reinforcement is not required

minimum reinforcement requirement $A_{sw} = \rho_w s b_w = 0.0028 * 200 * 160 = 89.6 \text{ mm}^2$

use $\emptyset 12$ c/c 200mm

DESIGN FOR THE PRESTRESSED CONCRETE RIB (T-SECTION) TYPE -3 & 4

The design of type 4 rib is taken to be identical since the initial dimension is similar and the span variation is minimum.

The only applied loading at transfer is the self weight which is (density of concrete) × (area). Hence;

$$\text{self weight} = \gamma A_c = 24 \times 108.4 \times 10^3 \times 10^{-6} = 2.60 \text{ kN/m}$$

The maximum moment due to this loading is: transfer moment,

$$M_t = 2.60 (5.5)^2 / 8 = 9.83 \text{ kNm}$$

Uniformly distributed load due to self-weight of the beam and other dead loads per meter width, w_o

$$w_o = \gamma A_c + w_d = (24 \times 108.4 \times 10^3 \times 10^{-6}) + 5.2 = 7.8 \text{ kN/m}$$

The total loading at SLS is this plus the imposed loading, i.e.: SLS moment,

$$M_s = (7.8 + 1.76)(5.5)^2 / 8 = 36.15 \text{ kNm}$$

o **Elastic sectional moduli**

From inequalities 2.15 and 2.16 we can obtain the elastic section moduli. Required about the top and

bottom fibers, Z_t and Z_b , as

$$Z_t \geq \frac{\alpha M_s - \beta M_t}{\alpha (f_{max})_s - \beta f_{min}} = \frac{0.96 * 36.15 * 10^6 - 0.79 * 9.83 * 10^6}{0.96 * 24.00 - 0.79 * (-3.5)} = \frac{26.94 * 10^6}{25.805} = 1.043 * 10^6 \text{ mm}^3 (< Z_t = 3.982 * 10^6 \text{ mm}^3)$$

=> Ok!

$$Z_b \geq \frac{\alpha M_s - \beta M_t}{\beta f'_{max} - \alpha f_{min}} = \frac{0.96 * 36.15 * 10^6 - 0.79 * 9.83 * 10^6}{0.79 * 18 - 0.96 * (-3.5)} = \frac{26.94 * 10^6}{17.58} = 1.532 * 10^6 \text{ mm}^3 (< Z_b = 1.852 * 10^6 \text{ mm}^3)$$

=> ok!

Table B.3: Geometric property

Label	Type 1
Area [mm ²]*10 ³	108.4
Inertia[mm ⁴]*10 ⁶	271.8
y _t [mm]	68
y _b [mm]	147
Z _t [mm ³]*10 ³	3982.9
Z _b [mm ³]*10 ³	1852.2

o **Determination of prestress forces and eccentricity**

From inequality 2.18 a, we get

$$\frac{1}{P_o} \leq \frac{\alpha \left(\frac{Z_t}{A_c} - e \right)}{(Z_t f'_{min} - M_t)} = \frac{0.96 \left(\frac{3.983 \cdot 10^6}{108400} - e \right)}{(3.983 \cdot 10^6 \cdot (-3.5) - 9.83 \cdot 10^6)} = \frac{0.96(36.74 - e)}{-23.77 \cdot 10^6} = \frac{(148.4 - 4.0e) \cdot 10^{-8}}{-1} \frac{1}{N}$$

Note that the denominator is negative. Dividing both sides of an inequality by a negative number has effect of changing the sign of the inequality. Thus the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq 4.0e - 148.4 \dots\dots (i)$$

From inequality 2.18 b,

$$\frac{1}{P_o} \geq \frac{\alpha \left(\frac{Z_b}{A_c} + e \right)}{(Z_b f'_{max} + M_t)} = \frac{0.96 \left(\frac{1.852 \cdot 10^6}{108400} + e \right)}{(1.852 \cdot 10^6 \cdot (18) + 9.83 \cdot 10^6)} = \frac{0.96(17.085 + e)}{43.166 \cdot 10^6} = \frac{(38.00 + 2.22e) \cdot 10^{-8}}{1} \frac{1}{N}$$

Thus, the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq 2.22e + 38.00 \dots\dots (ii)$$

From inequality 2.18 c,

$$\frac{1}{P_o} \geq \frac{\beta \left(\frac{Z_t}{A_c} - e \right)}{(Z_t (f_{max})_s - M_s)} = \frac{0.79 \left(\frac{3.982 \cdot 10^6}{108400} - e \right)}{(3.982 \cdot 10^6 \cdot (24) - 36.15 \cdot 10^6)} = \frac{0.79(136.73 - e)}{59.418 \cdot 10^6} = \frac{(181.79 - 1.33e) \cdot 10^{-8}}{1} \frac{1}{N}$$

Thus, the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq -1.33e + 181.79 \dots\dots (iii)$$

From inequality 2.18 d,

$$\frac{1}{P_o} \leq \frac{\beta \left(\frac{Z_b}{A_c} - e \right)}{(Z_b f_{min} - M_s)} = \frac{0.79 \left(\frac{1.852 \cdot 10^6}{108400} - e \right)}{(1.852 \cdot 10^6 \cdot (-3.5) - 36.15 \cdot 10^6)} = \frac{0.79(17.085 - e)}{-42.632 \cdot 10^6} = \frac{(31.66 - 1.85e) \cdot 10^{-8}}{-1} \frac{1}{N}$$

Note that the denominator is negative. Dividing both sides of an inequality by a negative number has effect of changing the sense of the inequality. Thus, the above inequality can be simplified as;

$$\frac{10^8}{P_o} \geq 1.85e - 31.66 \dots\dots (iv)$$

Now putting all the four inequalities together:

$$\frac{10^8}{P_o} \geq 4.0e - 148.4 \dots\dots\dots (i)$$

$$\frac{10^8}{P_o} \geq 2.22e + 38.00 \dots\dots\dots (ii)$$

$$\frac{10^8}{P_o} \geq -1.33e + 181.79 \dots\dots\dots (iii)$$

$$\frac{10^8}{P_o} \geq 1.85e - 31.66 \dots\dots\dots (iv)$$

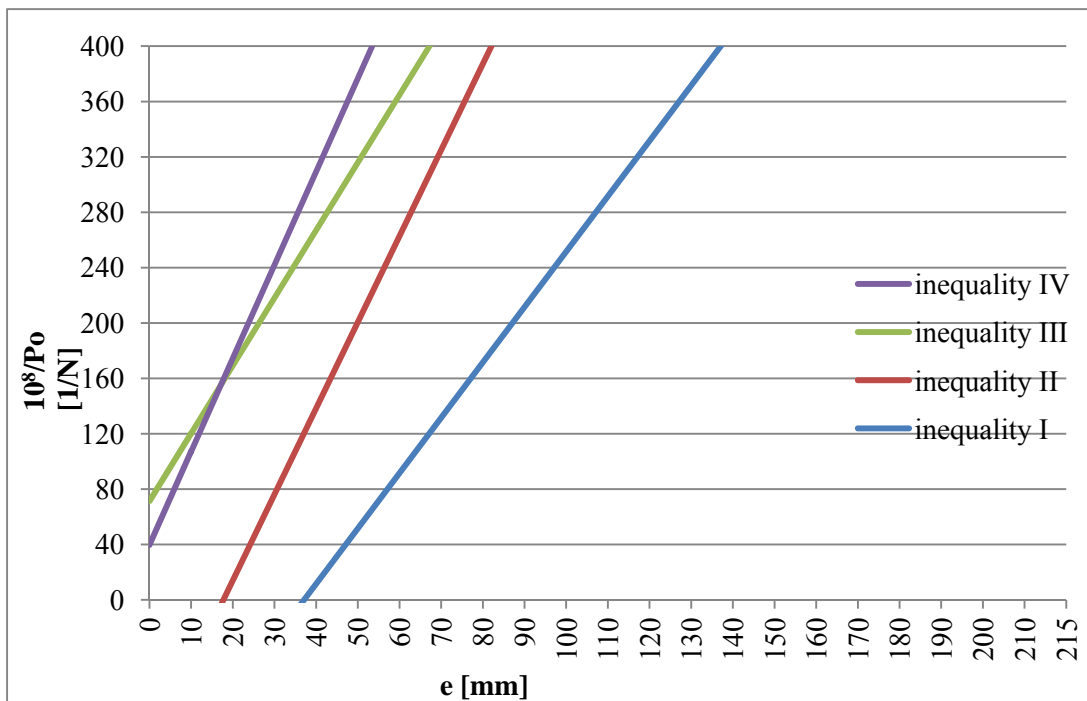


Figure B.3: Magnel diagram for type 3

In this case, the distance from the neutral axis to the soffit is 147mm. if we logically take the distance from the center of prestressing steel to the soffit as 80mm taking in to account the cover requirement, the maximum value of eccentricity for the permissible zone is;

$$e = 147 - 80 = 67\text{mm taking it to be } 70\text{mm}$$

The corresponding prestressing force will be obtained by;

$$\frac{10^8}{P_o} \geq 4.0e - 148.4 = 4.0 * 70 - 148.4 = 131.6 \text{ 1/N}$$

$$P_o \leq 10^8/131.6 = 759.88 \text{ KN}$$

$$\frac{10^8}{P_o} \geq 2.22e + 38.00 = 2.22 * 70 + 38.00 = 193.4 \text{ 1/N}$$

$$P_o \leq 10^8/193.4 = 517.063 \text{ KN}$$

$$\frac{10^8}{P_o} \geq -1.33e + 181.79 = -1.33 * 70 + 39.613 = -53.487 \text{ 1/N}$$

$$P_o \leq 10^8/-53.487 = -1869.613 \text{ KN}$$

$$\frac{10^8}{P_o} \geq 1.85e - 31.66 = 1.85 * 70 - 23.281 = 106.219 \text{ 1/N}$$

$$P_o \leq 10^8/106.219 = 941.45 \text{ KN}$$

Thus; $P_o \leq 517.063 \text{ KN}$ and $P_o = 517 \text{ KN}$ can be chosen because which satisfies the solution set.

Selection of prestressing steel

According to Euro code manual, when checking the stresses at transfer, the prestressing force required for the in-service condition should be increased by 60%, as the long-term (time-independent) losses will not have occurred, and no tension should be allowed in concrete.

$$P_{\text{req}} = P_o/0.6 = 517/0.6 = 861.67 \text{ kN}$$

Here one assumes 3No.7-wire super strands, so the required characteristic load per strand will be:

$$P_{\text{req}}/3 = 861.67/3 = 287.22 \text{ kN/ strand}$$

From steel manufacturer specification manual selecting 3 No-7- wire drawn strands of 15.2mm nominal diameter with $P_o = 288 \text{ kN /strand}$ with nominal strength $f_{pk} = 1860\text{N/mm}^2$ and cross sectional area of $A_p = 139.0 \text{ mm}^2$.

The total prestress force P_u is calculated as;

$$P_u = 288 * 3 = 864 \text{ kN}$$

The total area of the prestressing steel A_{pu} is calculated as;

$$A_{pu} = A_p * 3 = 139.0 \text{ mm}^2 * 3 = 417 \text{ mm}^2$$

The actual prestress force P_o is now becomes

$$P_o = 0.6 * P_u = 0.6 * 864 = 518.4 \text{ kN}$$

o **Calculation of losses**

Before checking the concrete stress at transfer and service the estimated loss should be calculated to get a fairly accurate result.

o **Short time losses (Immediate losses)**

o **Elastic shortening:**

The stress in the prestressing steel due to elastic shortening is given by equation 2.37 as:

$$\begin{aligned} \sigma_{ci} &= \frac{f_{po}}{m + \frac{A_c}{A_p(1 + \frac{e^2}{r^2})}} - \frac{M_o e}{I_c} = \frac{1243.165 \frac{N}{mm^2}}{5.86 + \frac{108.4 * 10^3 mm^2}{417 mm^2 (1 + \frac{70^2}{50.074^2})}} - \frac{9.83 * 10^6 Nmm * 70 mm}{271.8 * 10^6 mm^4} \\ &= \frac{1243.165 \frac{N}{mm^2}}{5.86 + 87.994} - 2.53 \frac{N}{mm^2} = 10.716 \frac{N}{mm^2} \end{aligned}$$

Where:

$$f_{po} = P_o / A_p = 518.4 * 10^3 N / 417 \text{ mm}^2 = 1243.165 \text{ MPa}$$

$$m = E_s / E_{cm} = 205 \text{ GPa} / 35 \text{ GPa} = 5.86$$

$$r = (I_c / A_c)^{0.5} = (271.8 * 10^6 \text{ mm}^4 / 108.4 * 10^3 \text{ mm}^2)^{0.5} = 50.074 \text{ mm}$$

$$e = 70 \text{ mm}$$

The prestress force loss due to elastic shortening is then computed as;

$$\Delta P_{el} = m * \sigma_{ci} * A_p = 5.86 * 10.716 * 417 = 26185.83 \text{ N} = 26.185 \text{ kN}$$

Therefore the percentage loss due to elastic shortening is 5.05%

o **Long time losses (Differed losses)**

o **Shrinkage**

To compute the prestress loss due to shrinkage, the value of concrete shrinkage after tensioning ϵ_r must be known to get $\Delta \epsilon_r$ because once $\Delta \epsilon_r$ is known using equation 2.38 and 2.39 the value of the stress can be computed.

$$\epsilon_r = \epsilon_{cd, \infty} * \frac{j}{j + 0.04 * h_o^{1.5}} + \epsilon_{ca, \infty} * 1 - \exp(-0.2 * \sqrt{j}) \quad (2.41)$$

ϵ_{cd} according to Euro Code 2, EN 1992-1-1 section 3.1.4 is 0.46‰

$$\epsilon_{ca,\infty} = 2.5 * (f_{ck} - 10) * 10^{-4} = 2.5 * (40 - 10) * 10^{-6} = 7.5 * 10^{-5}$$

$$j = 50 \text{ years} = 18250 \text{ days}$$

$$h_o = \frac{2A_c}{U} = \frac{2 * 108.4 * 10^3 \text{ mm}^2}{[160 + (2 * 115) + (2 * 100) + 900 + (2 * 370)] \text{ mm}} = 97.22 \text{ mm}$$

$$\begin{aligned} \epsilon_{r,50 \text{ Yrs}} &= 0.46 * 10^{-3} * \frac{18250}{18250 + 0.04 * 97.22^{1.5}} + 7.5 * 10^{-5} * 1 - \exp(-0.2 * \sqrt{18250}) \\ &= 4.59 * 10^{-4} + 7.5 * 10^{-5} = 5.34 * 10^{-4} \end{aligned}$$

$$\begin{aligned} \epsilon_{r,28 \text{ days}} &= 0.46 * 10^{-3} * \frac{28}{28 + 0.04 * 97.22^{1.5}} + 7.5 * 10^{-5} * 1 - \exp(-0.2 * \sqrt{28}) \\ &= 1.94 * 10^{-4} + 4.9 * 10^{-5} = 2.43 * 10^{-4} \end{aligned}$$

$$\Delta \epsilon_r = \epsilon_{r,50 \text{ Yrs}} - \epsilon_{r,28 \text{ days}} = (5.34 * 10^{-4}) - (2.43 * 10^{-4}) = 2.91 * 10^{-4}$$

$$\text{Using Equation 2.21, } \Delta P_p = -A_p E_p \Delta \epsilon_r = -417 \text{ mm}^2 * 205 * 10^3 \frac{\text{N}}{\text{mm}^2} * 2.91 * 10^{-4}$$

$$\Delta P_p = -24876.135 \text{ N} = -24.88 \text{ kN}$$

Therefore the percentage loss due to shrinkage is 4.79%

○ **Stress relaxation**

According to the specification manual attached in Annex of C the prestressing tendon, the maximum relaxation loss is given to be 2.5%.

○ **Creep**

To calculate the deformation due to the sustained compressive load which further facilitates the computation of prestress loss due to creep, first the stress σ_c should be computed,

$$\sigma_c = \frac{P}{A_c} + \frac{P * e_o^2}{I} + \frac{M * e_o}{I}$$

$$\sigma_c = \frac{518.4 * 10^3 \text{ N}}{108.4 * 10^3 \text{ mm}^2} + \frac{518.4 * 10^3 \text{ N} * 70^2 \text{ mm}^2}{271.8 * 10^6 \text{ mm}^4} + \frac{-9.83 * 10^6 \text{ Nmm} * 70 \text{ mm}}{271.8 * 10^6 \text{ mm}^4}$$

$$= 4.78 \frac{N}{mm^2} + 9.34 \frac{N}{mm^2} - 2.53 \frac{N}{mm^2} = 11.59 \text{MPa}$$

$$0.45f_{ck} = 0.45 * 40 \text{MPa} = 18 \text{MPa} > \sigma_c$$

i.e. the creep strain can be computed using equation 2.43; $\epsilon_{cc}(\infty, t_o) = \varphi(\infty, t_o) \cdot \left(\frac{\sigma_c}{E_c}\right)$ where the creep coefficient can be read from the Figure 3.1 provided in Euro code 2 EN 1991-1-1

The value of $\varphi(\infty, t_o)$ from the graph is then; $\varphi(\infty, t_o) = 2$

The creep strain can then be computed as; $\epsilon_{cc}(\infty, t_o) = \varphi(\infty, t_o) \cdot \left(\frac{\sigma_c}{E_c}\right) = 2 * \left(\frac{11.59}{35 * 10^3}\right) = 6.62 * 10^{-4}$

The stress due to the creep stain is then evaluated as; $\Delta\sigma_p = E_p \epsilon_{cc}$

$$= 205 * 10^3 \text{ N/mm}^2 * 6.62 * 10^{-4} = 135.71 \text{N/mm}^2$$

The loss in the prestress force is $\Delta P_p = \Delta\sigma_p A_p = 135.71 \text{N/mm}^2 * 417 \text{mm}^2 = 56591.07 \text{ N} = 56.59 \text{kN}$

Therefore the percentage loss due to Creep is 10.91%

Table B.4: Percentage losses calculation results for type 3.

TYPE OF LOSSES	PERCENTAGE LOSS OF STRESS
	TYPE-3
Elastic shortening	5.05
Creep of concrete	10.91
Shrinkage of concrete	4.79
Stress relaxation	2.5
Short time losses [%]	5.05
Long time losses [%]	18.2

P_o is computed taking: $\alpha = 1 - 0.04 = 0.96$ and $\beta = 1 - 0.21 = 0.79$

Now $\alpha = 1 - 0.0505 = 0.9495$ and $\beta = 1 - 0.182 = 0.818$

The assumed valued and the computed losses are comparatively accurate.

▪ **Concrete stress at transfer**

From equation 2.13 a, the stress at the top fiber f_t is calculated as;

$$f'_t = \frac{\alpha P_0}{A_c} - \frac{\alpha P_0 e}{Z_t} + \frac{M_o}{Z_t}$$

$$f'_t = \frac{0.9495 * 518.4 * 10^3 N}{108.4 * 10^3 mm^2} - \frac{0.9495 * 518.4 * 10^3 N * 70 mm}{3982.9 * 10^3 mm^3} + \frac{9.83 * 10^6 Nmm}{3982.9 * 10^3 mm^3}$$

$$f'_t = 4.54 \frac{N}{mm^2} - 8.65 \frac{N}{mm^2} + 2.47 \frac{N}{mm^2}$$

$$f'_t = -1.64 \frac{N}{mm^2} > f'_{min} = -3.5 \frac{N}{mm^2} \text{ OK!}$$

From equation 2.13 b, the stress at the bottom fiber f_b is calculated as;

$$f'_b = \frac{\alpha P_0}{A_c} + \frac{\alpha P_0 e}{Z_b} - \frac{M_o}{Z_b}$$

$$f'_b = \frac{0.9495 * 518.4 * 10^3 N}{108.4 * 10^3 mm^2} + \frac{0.9495 * 518.4 * 10^3 N * 70 mm}{1852.2 * 10^3 mm^3} - \frac{9.83 * 10^6 Nmm}{1852.2 * 10^3 mm^3}$$

$$f'_b = 4.54 \frac{N}{mm^2} + 18.60 \frac{N}{mm^2} - 5.31 \frac{N}{mm^2}$$

$$f'_b = 17.83 \frac{N}{mm^2} < f'_{max} = 18 \frac{N}{mm^2} \text{ OK!}$$

▪ **Concrete stress at service**

From equation 2.13 c, the stress at the top fiber f_t is calculated as;

$$f'_{tservice} = \frac{\beta P_0}{A_c} - \frac{\beta P_0 e}{Z_t} + \frac{M_s}{Z_t}$$

$$f'_{tservice} = \frac{0.818 * 518.4 * 10^3 N}{108.4 * 10^3 mm^2} - \frac{0.818 * 518.4 * 10^3 N * 70 mm}{3982.9 * 10^3 mm^3} + \frac{36.15 * 10^6 Nmm}{3982.9 * 10^3 mm^3}$$

$$f'_{tservice} = 3.91 \frac{N}{mm^2} - 7.45 \frac{N}{mm^2} + 9.08 \frac{N}{mm^2}$$

$$f'_{tservice} = 5.54 \frac{N}{mm^2} < (f_{max})_{serv} = 24 \frac{N}{mm^2} \text{ OK!}$$

From equation 2.13 d, the stress at the bottom fiber f_b is calculated as;

$$f'_{bserve} = \frac{\beta P_0}{A_c} + \frac{\beta P_0 e}{Z_b} - \frac{M_s}{Z_b}$$

$$f'_{bserve} = \frac{0.818 * 518.4 * 10^3 N}{108.4 * 10^3 mm^2} + \frac{0.818 * 518.4 * 10^3 N * 70 mm}{1852.2 * 10^3 mm^3} - \frac{36.15 * 10^6 Nmm}{1852.2 * 10^3 mm^3}$$

$$f'_{bserve} = 3.91 \frac{N}{mm^2} + 16.03 \frac{N}{mm^2} - 19.52 \frac{N}{mm^2}$$

$$f'_{bserve} = 0.42 \frac{N}{mm^2} > f'_{min} = -3.5 \frac{N}{mm^2} \text{ Ok!}$$

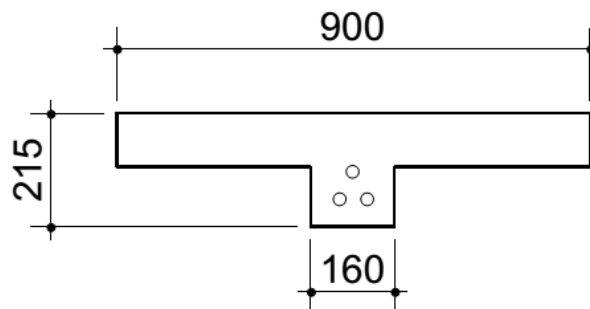


Figure B.4: cross section for Type 3

Bending Moment resistance

The initial stress in tendons according to Euro code 2 EN 1992-1-1:2004 is

$$f_{po} = 0.7f_{pk} = 0.7 * 1860 N/mm^2 = 1302 N/mm^2$$

The elastic modulus of steel is $E_p = 205 kN/mm^2$

Using the procedure for design of flanged beam

- $f_{po}/f_{pk} = 0.7$
- $K = M/bd^2f_{ck} = [((1.3*7.8)+(1.6*1.76))*5.5^2/8]*10^6 / (900*138^2*40) = 0.0714$
- Using Figure A1 in Annex A, $x/d = 0.28$ which implies that $x = 38.64 mm$. Checking the value of x/d with the limiting value obtained from Table A1, $0.28 < 0.35 \Rightarrow ok!$
- $0.8x \leq h_f$, $30.91 mm \leq 100 mm \Rightarrow A_p$ is determined as for a rectangular beam of breadth b
- Using Figure A2 in Annex A, the ratio of $K_{lim} = A_p f_{pk} / bdf_{ck} = 0.09$ and $K = A_p f_{pk} / bdf_{ck} = 417 * 1860 / (900 * 138^2 * 40) = 0.00113$ i.e. $K < K_{lim}$ which means area of reinforcement provided for serviceability is satisfactory at ultimate limit state.

Minimum reinforcement

The minimum area of longitudinal reinforcement in order to avoid brittle failure should not be less than either $0.6b_t d / f_{pk}$ or $0.0015b_t d$.

- $0.6b_t d / f_{pk} = 7.122$ and $0.0015b_t d = 33.12$ which in both case is satisfied.

Providing minimum area of compression reinforcement for lifting and placing of the T-sections into their position: use $5\phi 8c/c$ 200mm

Tendon spacing

According to manual for the design of reinforced concrete building structures to EC2,

- Distance between the pretensioned tendons is taken as 25mm
- Clear vertical distance between tendons is taken as 25mm
- According to Table A2 of Annex A the cover requirement is fulfilled

Shear design

$$\text{self weight} = \gamma A_c = 24 \times 108.4 \times 10^3 \times 10^{-6} = 2.60 \text{ kN/m}$$

The maximum moment due to this loading is: transfer moment,

$$M_t = 2.60 (5.5)^2 / 8 = 9.83 \text{ kNm}$$

Uniformly distributed load due to self-weight of the beam and other dead loads per meter width, w_o

$$w_o = \gamma A_c + w_d = (24 \times 108.4 \times 10^3 \times 10^{-6}) + 5.2 = 7.80 \text{ kN/m}$$

The total loading at SLS is this plus the imposed loading, i.e.: SLS moment,

$$M_s = (7.80 + 1.76)(5.5)^2 / 8 = 36.15 \text{ kNm}$$

Total loading at ultimate limit state, w_{ult}

$$w_{ult} = 1.3 * 7.80 + 1.6 * 1.76 = 12.96 \text{ kN/m}$$

The maximum moment due to this loading is: ultimate moment,

$$M_{ult} = 12.96 (5.5)^2 / 8 = 48.99 \text{ kNm}$$

Design ultimate shear force, V_{sd}

$$V_{sd} = w_{ult} * (L/2 - d) = 12.96 * (5.5/2 - 0.138) = 33.85 \text{ kN}$$

$$V_{Rd 2, red} = 1.67 V_{Rd2} \left(\frac{1 - 1.5 \sigma_{cp, eff}}{f_{ck}} \right) \leq V_{Rd 2}$$

$$V_{Rd 2} = 0.15 f_{ck} b_{w, nom} d = 0.15 * 40 \frac{N}{mm^2} * 160 \text{ mm} * 138 \text{ mm} = 132480 \text{ N} = 132.48 \text{ kN}$$

$$b_{w, nom} = 160 \text{ mm}$$

$$\sigma_{cp, eff} = 1.2 P_o / A_c = 1.2 * 518.4 * 10^3 \text{ N} / 108.4 * 10^3 \text{ mm}^2 = 5.74 \text{ MPa}$$

$$V_{Rd2,red} = 1.67 * 132.48 * 10^3 N \left(\frac{1-1.5*5.74MPa}{40MPa} \right) \leq V_{Rd2}$$

$$V_{Rd2,red} = -42.09kN \leq 264kN$$

$$V_{Rd1} = v_{Rd1} b_w d \text{ referring figure A3 in annex A the value of } v_{Rd1} = 1.65N/mm^2$$

$$V_{Rd1} = 1.65 \frac{N}{mm^2} * 160mm * 138mm = 36.43kN$$

$V_{sd} < V_{Rd1}$, design for shear reinforcement is not required

minimum reinforcement requirement $A_{sw} = \rho_w s b_w = 0.0028 * 200 * 160 = 89.6mm^2$

use $\emptyset 12$ c/c 200mm

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

ANNEX B.2 SAMPLE MANUAL CALCULATION FOR GIRDER BEAMS USING EXCEL.

Girder beam: Ax 4 - 5'		span length:	8.9 m						
Geometric property		Design parameters			Cross section dimation				
Area [mm ²]*10 ³	246.25	$\gamma =$	24	kN/m ³	D	500 mm			
Inertia[mm ⁴]*10 ⁶	4507.73	$\beta =$	0.79		Df	385 mm			
yt [mm]	272.47	$\alpha =$	0.96		bf	300 mm			
yb [mm]	227.53	$f_{ck} =$	47	N/mm ²	bw	800 mm			
Zt [mm ³]*10 ³	16543.68	$(f_{max})_s =$	30	N/mm ²	u	2100 mm			
Zb [mm ³]*10 ³	19811.99	$f_{ck} =$	50	N/mm ²					
		$f_{min} =$	-4.1	N/mm ²					
		$f_{max} =$	22.5	N/mm ²					
Design moments	Mtb	from ETABS output	Msb	MsttL	MstR	Mub	MutL	MutR	Vult
	58.52		207.31	-235.91	-491.39	558.32	-1022.53	-1022.53	538.57
Required about the top and bottom fibers, Z _t and Z _b ,									
Z _t [mm] =		4768864.63	Ok!						
Z _b [mm]=		7037430.51	Ok!						
Determination of prestress forces and eccentricity									
from inequality 2.18a	1/po ≤	64.50	minus	0.96 e	equals	51.05	minus	0.76 e x10 ⁻⁸	
		<hr/>				<hr/>			-1
		-126.345456	X10 ⁶						
	10 ⁸ /po ≥	0.76e-51.05	(i)						
from inequality 2.18 b	1/po ≤	77.24	minus	0.96 e	equals	15.32	minus	0.19 e x10 ⁻⁸	
		<hr/>				<hr/>			1
		504.2862658	X10 ⁶						
	10 ⁸ /po ≥	0.19e+15.32	(ii)						
from inequality 2.18c	1/po ≥	53.07	minus	0.79 e	equals	18.36	minus	0.27 e x10 ⁻⁸	
		<hr/>				<hr/>			1
		289.0002571	X10 ⁶						
	10 ⁸ /po ≥	18.36-0.27e	(iii)						
from inequality 2.18d	1/po ≤	63.56	minus	0.79 e	equals	22.03	minus	0.27 e x10 ⁻⁸	
		<hr/>				<hr/>			-1
		-288.5391778	X10 ⁶						
	10 ⁸ /po ≥	0.27e+22.03	(iv)						
magnel diagram									
Distance from the neutral axis to the soffit = 227.53 mm									

$e = 147.00 \text{ mm}$

from (i)	$10^8/p_o \geq 0.76e-51.05$	$P_o \leq 1648.261 \text{ kN}$
from (ii)	$10^8/p_o \geq 0.19e+15.32$	$P_o \leq 2312.139 \text{ kN}$
from (iii)	$10^8/p_o \geq 18.36-0.27e$	$P_o \leq -4688.23 \text{ kN}$
from (iv)	$10^8/p_o \geq 0.27e+22.03$	$P_o \leq 1620.22 \text{ kN}$

thus; $P_o = 1620.22 \text{ kN}$
 $P_{req} = 2700.37 \text{ kN}$

assume: 9 No.7 wire strands

characteristic load per strand = 300.04 kN

from specification manual: 9 No.7 wire strands of 15.7 mm diameter

$P_o = 301 \text{ kN}$

$A_p = 150 \text{ mm}^2$

$f_{pk} = 1860 \text{ Mpa}$

$P_u = 2709 \text{ kN}$

$A_{pu} = 1350 \text{ mm}^2$

Now actual $P_o = 1625.4 \text{ kN}$

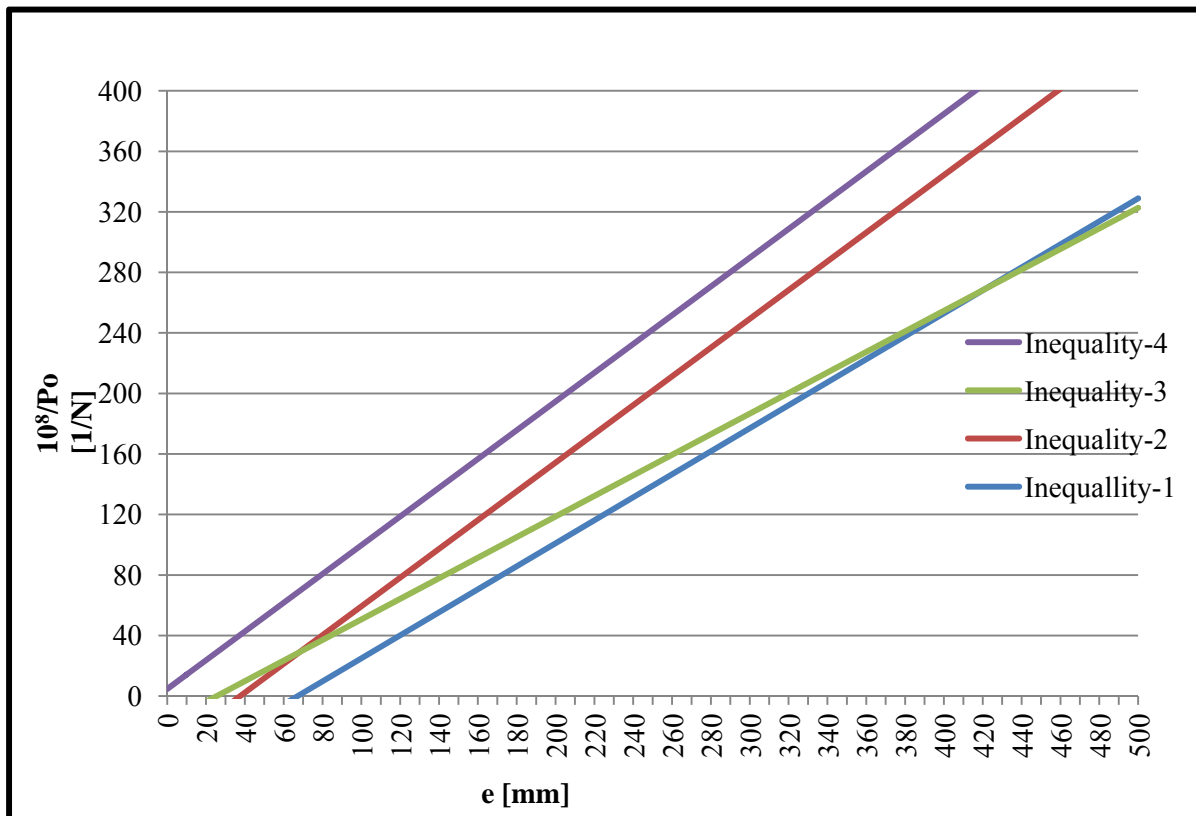


Figure B.5: Magnel diagram for Girder beam 4-5'

<u>Calculation of losses</u>				
Immediate losses:				
elastic shortening:	σ_{ci} =	11.59	Mpa	
	f_{po} =	1204	mm ²	
	r =	135.30	mm	
	$M_o * e / I_c$ =	1.90825668		
	$A_p(1+(e^2/r^2))$ =	2943.626807		
	prestress loss ΔP_{el} =	91.69	kN	
	therefore percentage loss =	5.64	%	
Diffreded losses:				
shrinkage:	ϵ_{cd} =	0.46	‰	
	h_o =	234.52	mm	
	$\epsilon_{r,50yrs}$ =	0.000456407		
	$\epsilon_{r,28days}$ =	7.51833E-05		
	$\Delta \epsilon_r$ =	0.000381224		
	ΔP_p =	-105.50	kN	
	therefore percentage loss =	6.49	%	
stress relaxion loss:		2.5	%	
creep:	σ_c =	12.48	Mpa	< 0.45f _{ck} =18Mpa
	$\varphi(\infty, t_o)$			
	=	2.1		
	$\epsilon_{cc(\infty, t_o)}$ =	0.000749		
	ΔP_p =	207.30	kN	
	therefore percentage loss =	12.75	%	
	Short time losses [%] =	5.64		
	Long time losses [%]=	14.63		
	α =	0.94		
	β =	0.85		
Concrete stress at transfer:				
	f_t =	-3.86	Mpa	Ok!
	f_b =	14.65	Mpa	Ok!
Concrete stress at service:				
	$f_{t\ service}$ =	5.84	Mpa	Ok!
	$f_{b\ service}$ =	5.47	Mpa	Ok!
Bending Moment resistance :				
	f_{po} =	1302.00	Mpa	
	E_p =	205.00	Gpa	kN/mm ²
	f_{po}/f_{pk} =	0.70		
	K =	0.08		
	x/d =	0.28		
	x =	117.60		
	$0.8x$ =	94.08		take b=web
using figure A2,	k_{lim} =	0.1		
	K =	0.3348		compression reinforcement is needed
	d' =	50.00		
	$d' >$	110.25	don't	700(1-(d'/x))

			use			
	A's=	1274.29				
			use	3	∅	24
						c/c 125mm
						double layer
Minimum reinforcement :						
	L. reinforcement=	225	mm ²			satisfied
	Compression rein.	300	mm ²			satisfied
tendon spacing: 25mm						
Shear design :						
	V _{sd} =	538.57	kN			
	V _{Rd 2} =	1732.5	kN			
	σ _{cp,eff} =	7.92	Mpa			
	V _{Rd 2,red} =	-629.64	kN			
	v _{Rd 1} =	2.2	N/mm ²			
	V _{Rd 1} =	508.2	kN			Design is required
minimum reinforcement:						
		168	mm ²	use	∅16	c/c 200mm

ANNEX B.3 QUANTITY CALCULATION RESULTS USING EXCEL.

Table B.5: total concrete quantity for the pre-cast prestressed girder beams

	Section name	No. of beams	Area (A_c) [mm ²]*10 ³	length of beams [m]	Volume of concrete [m ³]
Ax A/A'	Ax 4 - 5'	28	246.25	8.9	61.37
Ax B/B'	Ax 2' - 4	28	307.50	8.95	77.06
	Ax 4-6	28	315.00	11	97.02
	Ax 6-8	28	501.4	12.5	175.49
Ax C/C'	Ax 2-2'	28	92.25	2.45	6.33
Ax D	Ax 2' - 4	14	315.00	8.95	39.47
	Ax 4-5	14	315.00	5.5	24.26
	Ax 5-6	14	315.00	5.5	24.26
	Ax 6-8	14	501.4	12.5	87.75
Ax E	Ax 2- 4	14	501.4	11.4	80.02
	Ax 4-4'	14	92.25	3.27	4.22
	Ax 5-6	14	315.00	5.5	24.26
	Ax 6-7'	14	501.40	10	70.20
Ax 2	Ax C-D	14	90.00	5.5	6.93
	Ax-D-E	14	120.00	8.3	13.94
	Ax E-C'	14	90.00	5.5	6.93
	Ax C'-B'/C-B	6	120.00	4.2	3.02
Ax 4	Ax A-B	14	90.00	4.4	5.54
	Ax B-D	14	165.00	9.7	22.41
	Ax D-E	14	135.00	8.3	15.69
	Ax E-B'	14	165.00	9.7	22.41
	Ax B'-A'	14	90.00	4.4	5.54
Ax 5'	Ax A-B/B'-A'	28	90.00	4.4	11.09
Ax 6	Ax B-D	14	165.00	9.7	22.41
	Ax D-E	14	135.00	8.3	15.69
	Ax E-B'	14	165.00	9.7	22.41
Ax 8	Ax C-D	14	90.00	5.5	6.93
	Ax-D-E	14	120.00	8.3	13.94
	Ax E-C'	14	90.00	5.5	6.93
Total volume of concrete=					973.50

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

Table B.6: total steel quantity for the pre-cast prestressed girder beams

Section name	No. of beams	A_p [mm ²]	length of beams [m]	total length along the axis [m]	length of longitudinal compression reinforcement $\phi 16$	length of longitudinal compression reinforcement $\phi 20$	length of longitudinal compression reinforcement $\phi 24$	length of shear reinforcement $\phi 16$	number of shear reinforcement	total length of shear reinforcement $\phi 16$
Ax 4 - 5'	28	1350.00	8.9	249.2			806.4	2.8	1260	3528
Ax 2' - 4	28	1200.00	8.95	250.6		810.6		2	1267	2534
Ax 4-6	28	1200.00	11	308			1638	2.4	1554	3729.6
Ax 6-8	28	1500.00	12.5	350		1108.8		3	1764	5292
Ax 2-2'	28	600.00	2.45	68.6	264.6			1.3	357	464.1
Ax 2' - 4	14	1650.00	8.95	125.3			540.4	2.4	633.5	1520.4
Ax 4-5	14	900.00	5.5	77	107.8			2.4	392	940.8
Ax 5-6	14	900.00	5.5	77	107.8			2.4	392	940.8
Ax 6-8	14	1650.00	12.5	175		369.6		3.1	882	2734.2
Ax 2- 4	14	2100.00	11.4	159.6		338.8		3.1	805	2495.5
Ax 4-4'	14	1350.00	3.27	45.78	55.58			1.9	235.9	448.21
Ax 5-6	14	900.00	5.5	77	107.8			2.4	392	940.8
Ax 6-7'	14	1950.00	10	140		449.4		3	707	2121
Ax C-D	14	450.00	5.5	77	215.6			1.4	392	548.8
Ax-D-E	14	450.00	8.3	116.2		378		1.3	588	764.4
Ax E-C'	14	450.00	5.5	77	215.6			1.1	392	431.2
Ax C'-B'/C-B	6	450.00	4.2	25.2		88.2		1.3	129	167.7
Ax A-B	14	600.00	4.4	61.6			214.2	1.1	315	346.5
Ax B-D	14	600.00	9.7	135.8			582.4	1.5	686	1029
Ax D-E	14	300.00	8.3	116.2			630	1.3	588	764.4
Ax E-B'	14	600.00	9.7	135.8			582.4	1.5	686	1029
Ax B'-A	14	600.00	4.4	61.6			214.2	1.1	315	346.5
Ax A-B/B'-A'	28	600.00	4.4	123.2			428.4	1.1	630	693
Ax B-D	14	600.00	9.7	135.8			582.4	1.5	686	1029
Ax D-E	14	300.00	8.3	116.2			630	1.3	588	764.4
Ax E-B'	14	600.00	9.7	135.8			582.4	1.5	686	1029
Ax C-D	14	450.00	5.5	77	215.6			1.1	392	431.2
Ax-D-E	14	450.00	8.3	116.2		378		1.3	588	764.4
Ax E-C'	14	450.00	5.5	77	215.6			1.1	392	431.2
Total	25200.00			3690.68	1505.98	3921.4	7431.2			38259.11

Total mass in kg for prestressing stand = **29255.14**

Total mass in kg for reinforcement bar = **98866.22**

Table B.7: total columns concrete quantity for the pre-cast prestressed system

ID	Cross sectional area [m ²]	No of columns	length [m]	Total volume of concrete [m ³]
Along Ax-2	0.64	3	48	92.16
Along Ax-4	0.64	6	35.2	135.168
	0.64	13	3.2	26.624
	0.81	7	3.2	18.144
	1	4	3.2	12.8
Along Ax-5	0.64	1	48	30.72
Along Ax-5'	0.64	2	48	61.44
Along Ax-6	0.64	4	28.8	73.728
	0.64	4	3.2	8.192
	0.81	8	3.2	20.736
	1	12	3.2	38.4
Along Ax-8	0.64	1	48	30.72
Total				548.832

Table B.8: total columns concrete quantity for the reinforced concrete system

ID	Cross sectional area [m ²]	No of columns	length [m]	Total volume of concrete [m ³]
Along Ax-2	0.24	3	48	34.56
Along Ax-3	0.24	2	35.2	16.896
	0.3	10	32	96
	0.3	4	3.2	3.84
	0.49	10	3.2	15.68
	0.36	12	3.2	13.824
Along Ax-4	0.3	10	32	96
	0.3	18	3.2	17.28
	0.36	6	3.2	6.912
	0.49	14	3.2	21.952
Along Ax-5	0.24	2	35.2	16.896
	0.3	10	32	96
	0.3	14	3.2	13.44
Along Ax-st	0.09	1	48	4.32
Along Ax-5'	0.24	2	48	23.04
Along Ax-6	0.3	6	32	57.6
	0.3	8	3.2	7.68
	0.36	11	3.2	12.672
	0.49	10	3.2	15.68
Along Ax-7	0.3	10	32	96
	0.3	6	3.2	5.76
	0.36	14	3.2	16.128
	0.49	10	3.2	15.68
Along Ax-8	0.24	1	48	11.52
Total =				715.36

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

Table B.9: Total Re-bar quantity for the pre-cast prestressed ribbed slabs (T-beams)

Location	Bar No.	Dia.	Length	No. of Bars			Total No of Bars	Ø8	Ø10	Ø12
Ground Floor										
longitudinal reinforcement										
Type 1	14	8	9.7	5	222	1	1110	10767		
Type 2	12	8	8.3	5	75	1	375	3112.5		
Type 3	10	8	5.5	5	-	1	0	0		
Type 4	10	8	4.4	5	48	1	240	1056		
Shear reinforcement for web										
Type 1	10	10	0.895	50	37	2	3700		3311.5	
Type 2	12	12	0.71	43	25	1	1075			763.25
Type 3	12	12	0.5	29	0	0	0			0
Type 4	12	12	0.5	23	8	2	368			184
for flange										
Type 1	10	10	0.91	50	37	2	3,700.00		3367	
Type 2	12	12	0.91	43	25	1	1075			978.25
Type 3	12	12	0.91	29	0	1	0			0
Type 4	12	12	0.91	23	8	1	184.00			167.44
First floor-12th floor										
longitudinal reinforcement										
Type 1	14	8	9.7	5	408	2	4080	39576		
Type 2	12	8	8.3	5	300	1	1500	12450		
Type 3	10	8	5.5	5	36	1	180	990		
Type 4	10	8	4.4	5	96	2	960	4224		
Shear reinforcement For flange										
Type 1	10	10	0.90	50	408	2	40,800.00		36,516.00	
Type 2	12	12	0.71	43	300	1	12,900.00			9,159.00
Type 3	12	12	0.50	29	96	1	2,784.00			1,392.00
Type 4	12	12	0.50	23	96	2	4,416.00			2,208.00
For web										
Type 1	10	10	0.91	50	408	2	40,800.00		37,128.00	
Type 2	12	12	0.91	43	300	1	12,900.00			11,739.00
Type 3	12	12	0.91	29	36	1	1,044.00			950.04
Type 4	12	12	0.91	23	96	2	4,416.00			4,018.56
								72,175.50	6,678.50	2,092.94
								0.40	0.62	0.89
								28,509.32	4,120.63	1,858.53

Table B.10: Total beam concrete quantity and detail bar schedule for the reinforced concrete system

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

BILL OF QUANTITY					
ITEM	DESCRIPTION	UNIT	QTY	U.PRICE	TOTAL P.
	<u>B. SUPER-STRUCTURE</u> <u>2. CONCRETE WORK</u>				
2.1	Reinforced concrete				
	a) Beams	m3	801.27	2,500.00	2,003,175.00
2.2	Formwork				
	a) Beams	m2	7700.14	147.00	1,131,920.58
2.3	Reinforcement steel bars				
	a) dia. 8mm deformed bar	kg	3572.53	35.00	125,038.55
	b) dia. 10mm deformed bar	kg	28266.44	35.00	989,325.40
	c) dia. 12mm deformed bar	kg	2682.82	35.00	93,898.70
	d) dia. 14mm deformed bar	kg	2016.42	35.00	70,574.70
	e) dia. 16mm deformed bar	kg	16750.59	35.00	586,270.65
	f) dia. 20mm deformed bar	kg	103370.32	35.00	3,617,961.20
	g) dia. 24mm deformed bar	kg	82147.24	35.00	2,875,153.40
	TOTAL CARRIED TO SUMMARY.....				11,493,318.18

Take-off					
MSc THESIS		Date: Oct 2015			
Multiple	L/W/H	Volume/Area	Description		
Ground Floor Beam Formwork					
4	8.90 0.50		Asis-A /Sides/		
			Length	8.9	14
			Depth	0.5	
			17.80	m ²	
2	8.90 0.30		Asis-A /Bottom/		
			Length	8.9	
			Depth	0.3	
			5.34	m ²	
4	34.90 0.50		Asis-B /Sides/		
			Length	34.9	18
			Depth	0.5	
			69.80	m ²	
2	34.90 0.30		Asis-B /Bottom/		
			Length	34.9	
			Depth	0.3	
			20.94	m ²	
4	34.90 0.50		Asis-C /Sides/		
			Length	34.9	13
			Depth	0.5	
			69.80	m ²	
2	34.90 0.30		Asis-C /Bottom/		
			Length	34.9	
			Depth	0.3	
			20.94	m ²	
4	34.90 0.50		Asis-D /Sides/		
			Length	34.9	12
			Depth	0.5	
			69.80	m ²	
2	34.90 0.30		Asis-D /Bottom/		
			Length	34.9	
			Depth	0.3	
			20.94	m ²	
10	27.70 0.50		Asis-1 ,3,6,7 & 8/Sides/		
			Length	27.7	--
			Depth	0.5	
			138.50	m ²	
5	27.70 0.30		Asis-1 ,3,6,7 & 8 /Bottom/		
			Length	27.7	
			Depth	0.3	
			41.55	m ²	
4	36.50 0.50		Asis-4 & 5/Sides/		
			Length	36.5	14
			Depth	0.5	
			73.00	m ²	
2	36.50 0.30		Asis-4 & 5/Bottom/		
			Length	36.5	
			Depth	0.3	
			21.90	m ²	
TOTAL		570.31	m ²	of Formwork.	570.31

Ist&2nd-Floor Beam Formwork					
4	8.90 0.50	Asis-A /Sides/		14	
		Length	8.9		
		Depth	0.5		
		17.80	m ²		
2	8.90 0.25	Asis-A /Bottom/			
		Length	8.9		
		Depth	0.25		
		4.45	m ²		
4	34.90 0.50	Asis-B /Sides/		18	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-B /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
4	34.90 0.50	Asis-C /Sides/		13	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-C /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
4	34.90 0.50	Asis-D /Sides/		12	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-D /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
10	27.70 0.50	Asis-1 ,3,6,7 & 8/Sides/			
		Length	27.7	--	
		Depth	0.5		
		138.50	m ²		
5	27.70 0.25	Asis-1 ,3,6,7 & 8 /Bottom/			
		Length	27.7		
		Depth	0.25		
		34.63	m ²		
4	36.50 0.50	Asis-4 & 5/Sides/			
		Length	36.5	--	
		Depth	0.5		
		73.00	m ²		
2	36.50 0.25	Asis-4 & 5 /Bottom/			
		Length	36.5		
		Depth	0.25		
		18.25	m ²		
TOTAL OF 1ST & 2ND FLOOR		1096.76	m ²	of Formwork.	
				1096.76	

3rd to 12th-Floor Beam Formwork					
4	8.90 0.50	Asis-A /Sides/		14	
		Length	8.9		
		Depth	0.5		
		17.80	m ²		
2	8.90 0.25	Asis-A /Bottom/			
		Length	8.9		
		Depth	0.25		
		4.45	m ²		
4	34.90 0.50	Asis-B /Sides/		18	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-B /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
4	34.90 0.50	Asis-C /Sides/		13	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-C /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
4	34.90 0.50	Asis-D /Sides/		12	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-D /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
10	27.70 0.50	Asis-2,3,6,7 & 8/Sides/			
		Length	27.7	--	
		Depth	0.5		
		138.50	m ²		
5	27.70 0.25	Asis-1 ,3,6,7 & 8 /Bottom/			
		Length	27.7		
		Depth	0.25		
		34.63	m ²		
4	36.50 0.50	Asis-4 & 5/Sides/			
		Length	36.5	--	
		Depth	0.5		
		73.00	m ²		
2	36.50 0.25	Asis-4 & 5 /Bottom/			
		Length	36.5		
		Depth	0.25		
		18.25	m ²		
TOTAL OF 3RD & 12TH FLOOR		5483.80	m²	of Formwork.	5483.80

Top-Tie Beam Formwork					
4	8.90 0.50	Asis-A /Sides/		14	
		Length	8.9		
		Depth	0.5		
		17.80	m ²		
2	8.90 0.30	Asis-A /Bottom/			
		Length	8.9		
		Depth	0.3		
		5.34	m ²		
4	34.90 0.50	Asis-B /Sides/		18	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-B /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
4	34.90 0.50	Asis-C /Sides/		13	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-C /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
4	34.90 0.50	Asis-D /Sides/		12	
		Length	34.9		
		Depth	0.5		
		69.80	m ²		
2	34.90 0.25	Asis-D /Bottom/			
		Length	34.9		
		Depth	0.25		
		17.45	m ²		
10	27.70 0.50	Asis-2,3,6,7 & 8/Sides/		--	
		Length	27.7		
		Depth	0.5		
		138.50	m ²		
5	27.70 0.25	Asis-1 ,3,6,7 & 8 /Bottom/			
		Length	27.7		
		Depth	0.25		
		34.63	m ²		
4	36.50 0.50	Asis-4 & 5/Sides/		--	
		Length	36.5		
		Depth	0.5		
		73.00	m ²		
2	36.50 0.25	Asis-4 & 5 /Bottom/			
		Length	36.5		
		Depth	0.25		
		18.25	m ²		
TOTAL OF 3RD & 12TH FLOOR		549.27	m ²	of Formwork.	
					549.27

Concrete						
Ground Beam Concrete						
	2	8.90 0.50 0.30	Axis A			
			Length	8.9		
			Depth	0.5		
			Thickness	0.3		
		2.67	m ³			
	2	34.90 0.50 0.30	Axis B			
			Length	34.9		
			Depth	0.5		
			Thickness	0.3		
		10.47	m ³			
	2	34.90 0.50 0.30	Axis C			
			Length	34.9		
			Depth	0.5		
			Thickness	0.3		
		10.47	m ³			
	2	34.90 0.50 0.30	Axis D			
			Length	34.9		
			Depth	0.5		
			Thickness	0.3		
		10.47	m ³			
	5	27.70 0.50 0.30	Axis-1,3,6,7 & 8			
			Length	27.7		
			Depth	0.5		
			Thickness	0.3		
		20.78	m ³			
	2	36.50 0.50 0.30	Axis-4 & 5			
			Length	36.5		
			Depth	0.5		
			Thickness	0.3		
		10.95	m ³			
TOTAL			65.81	m ³	of Concrete.	65.81
1st&2nd-Floor Beam Concrete						
	2	8.90 0.50 0.25	Axis A			
			Length	8.9		
			Depth	0.5		
			Thickness	0.25		
		2.23	m ³			
	2	34.90 0.50 0.25	Axis B			
			Length	34.9		
			Depth	0.5		
			Thickness	0.25		
		8.73	m ³			
	2	34.90 0.50 0.25	Axis C			
			Length	34.9		
			Depth	0.5		
			Thickness	0.25		
		8.73	m ³			
	2	34.90 0.50 0.25	Axis D			
			Length	34.9		
			Depth	0.5		
			Thickness	0.25		
		8.73	m ³			
	5	27.70 0.50 0.25	Axis-1,3,6,7 & 8			
			Length	27.7		
			Depth	0.5		
			Thickness	0.25		
		17.31	m ³			
	2	36.50 0.50 0.30	Axis-4 & 5			
			Length	36.5		
			Depth	0.5		
			Thickness	0.3		
		10.95	m ³			
TOTAL OF 1ST & 2ND FLOOR			113.36	m ³	of Concrete.	113.36

3rd to 12th-Floor Beam Concrete						
2	8.90 0.50 0.25		Axis A		Length	8.9
			Depth	0.5		
			Thickness	0.25		
			2.23	m ³		
2	34.90 0.50 0.25		Axis B		Length	34.9
			Depth	0.5		
			Thickness	0.25		
			8.73	m ³		
2	34.90 0.50 0.25		Axis C		Length	34.9
			Depth	0.5		
			Thickness	0.25		
			8.73	m ³		
2	34.90 0.50 0.25		Axis D		Length	34.9
			Depth	0.5		
			Thickness	0.25		
			8.73	m ³		
5	27.70 0.50 0.25		Asis-2,3,6,7 & 8/Sides/		Length	27.7
			Depth	0.5		
			Thickness	0.25		
			17.31	m ³		
2	36.50 0.50 0.30		Asis-4 & 5		Length	36.5
			Depth	0.5		
			Thickness	0.3		
			10.95	m ³		
TOTAL OF 3RD & 12TH FLOOR		566.80	m³ of Concrete.		566.80	
Top-Tie Beam Concrete						
2	8.90 0.50 0.30		Axis A		Length	8.9
			Depth	0.5		
			Thickness	0.3		
			2.67	m ³		
2	34.90 0.50 0.25		Axis B		Length	34.9
			Depth	0.5		
			Thickness	0.25		
			8.73	m ³		
2	34.90 0.50 0.25		Axis C		Length	34.9
			Depth	0.5		
			Thickness	0.25		
			8.73	m ³		
2	34.90 0.50 0.25		Axis D		Length	34.9
			Depth	0.5		
			Thickness	0.25		
			8.73	m ³		
5	27.70 0.50 0.25		Asis-2,3,6,7 & 8/Sides/		Length	27.7
			Depth	0.5		
			Thickness	0.25		
			17.31	m ³		
2	36.50 0.50 0.25		Asis-4 & 5		Length	36.5
			Depth	0.5		
			Thickness	0.25		
			9.13	m ³		
0		55.30	m³ of Concrete.		55.30	

SUPER STRUCTURE--- Beam Bars																		
Location	Ref No.	Shape	f	Bar Length (mm)	Span Length(mm)	Spacing (mm)	No. of Bar	No Floors	No. of memb.	Diameter of Bars								
										6	8	10	12	14	16	20	24	30
Ground Floor Beam																		
Axis A		450 9450 450	14	10350			3	1	2				62.10					
			14	9450				3	1	2				56.70				
Axis B		450 9025 6550 6300 6550 10175 6800 11750 3830 1625 6075 450	16	9475			4	1	2					75.80				
			16	6550				2	1	2				26.20				
			16	6300				2	1	2				25.20				
			16	6550				2	1	2				26.20				
			16	10625				2	1	2				42.50				
			16	6800				2	1	2				27.20				
			16	11750				2	1	2				47.00				
			14	3830				1	1	2				7.66				
			14	2075				1	1	2				4.15				
			16	6075				2	1	2				24.30				
			Axis C		450 9025 12000 6600 4170 10175 14320 6075 6800 11750 13975 450	16	9475			2	1	2					37.90	
16	12000							2	1	2				48.00				
16	6600							2	1	2				26.40				
14	4170							2	1	2				16.68				
16	10625							2	1	2				42.50				
14	14320							2	1	2				57.28				
16	6075							2	1	2				24.30				
16	6800							4	1	2				54.40				
16	11750							3	1	2				70.50				
16	13975							4	1	2				111.80				
0	9475							2	1	2				37.90				



SUPER STRUCTURE--- Beam Bars

Location	Ref No.	Shape	f	Bar Length (mm)	Span Length(mm)	Spacing (mm)	No. of Bar	No Floors	No. of memb.	Diameter of Bars										
										6	8	10	12	14	16	20	24	30		
Axis D		45	9025	16	12000		2	1	2						48.00					
				16	6600		2	1	2					26.40						
				14	4170		2	1	2					16.68						
				16	10175	450	16	10625		2	1	2					42.50			
				14	38250		14	38250		3	1	2				229.50				
				14	11000		14	11000		1	1	2				22.00				
				14	7490		14	7490		2	1	2				29.96				
				16	11050		16	11050		4	1	2					88.40			
Axis 2&8		450	10150	14	2505		2	1	2					10.02						
				14	3230		2	1	2					12.92						
				16	10925		16	10925		4	1	2				87.40				
				20	9200		20	9200		2	1	7					128.80			
Beam E'-D Axis 2-8		450	8300	12	8300		4	1	7			232.40								
				20	2800		20	2800		2	1	7				39.20				
				20	2800		20	2800		1	1	7				19.60				
				20	2800		20	2800		1	1	7				19.60				
Axis 3,6&7		450	10150	16	11050		6	1	2					132.60						
				16	2505		16	2505		6	1	2				30.06				
				16	6125		16	6125		12	1	2				147.00				
				16	4800		16	4800		6	1	2				57.60				
				14	3230		14	3230		6	1	2				38.76				
				14	1500		14	1500		3	1	2				9.00				
				16	1050	450	16	1050		3	1	2				9.00				
Axis 4&5		450	8425	16	8875		4	1	2					71.00						
				16	7575		16	7575		4	1	2				60.60				
				16	6100		16	6100		2	1	2				24.40				
				16	10325		16	10325		4	1	2				82.60				
				14	10325		14	10325		4	1	2				82.60				
				16	2015		16	2015		6	1	2				24.18				
				16	5025	450	16	5025		8	1	2				80.40				
				16	5025		16	5025		8	1	2				80.40				
Total Length (m)													232.40	656.01	1,751.24	187.60				
Unit weight /meter (kg/m)										0.222	0.395	0.617	0.888	1.209	1.579	2.469	3.555	5.55		
Total weight (kg)													206.37	793.12	2,765.21	463.18				

SUPER STRUCTURE---- Beam Bars

Location	Ref No.	Shape	f	Bar Length (mm)	Span Length(mm)	Spacing (mm)	No. of Bar	No Floors	No. of memb.	Diameter of Bars								
										6	8	10	12	14	16	20	24	30
			24	26225			4	2	2								419.60	
Axis D	450	26225	24	9725			5	2	2								194.50	
		9275	24	23230			2	2	2								185.84	
		23230	20	14660			3	2	2							175.92		
		14660	24	10825			3	2	2								129.90	
		10375	16	6175			3	2	2						74.10			
		6175	16	6500			2	2	2						52.00			
		6500	24	2075			2	2	2								16.60	
		1625	16	27775			4	2	2						444.40			
		27775	16	11050			4	2	2						176.80			
		Axis 2&8	450	10150	16	4800			4	2	2							76.80
4800	16			2505			2	2	2							20.04		
2055	14			3230			2	2	2					25.84				
3230	16			6125			6	2	2						147.00			
6125	20			9200			2	2	7								257.60	
Beam E'-D Axis 2-8	450	8300	12	8300			4	2	7			464.80						
		8300	20	2800			2	2	7							78.40		
		2800	20	2800			1	2	7							39.20		
		2800	24	11050			6	2	2								265.20	
Axis 3,6&7	450	10150	20	2455			3	2	2							29.46		
		2005	24	3230			3	2	2								38.76	
		3230	20	6125			12	2	2							294.00		
		6125	20	4800			6	2	2							115.20		
		4800	16	4800			6	2	2						115.20			
		4800	16	1500			3	2	2						18.00			
		1050	24	8875			4	2	2								142.00	
		8425	24	7775			4	2	2								124.40	
Axis 4&5	450	7325	24	3235			4	2	2								51.76	
		3235	20	2865			2	2	2							22.92		
		2865	24	15350			8	2	2								491.20	
		15350	24	2015			6	2	2								48.36	
		1565	20	2000			4	2	2							32.00		
		1550																
		1550																
Total Length (m)													464.80	25.84	1,199.94	1,663.78	3,526.16	
Unit weight /meter (kg/m)										0.222	0.395	0.617	0.888	1.209	1.579	2,469	3,555	5.55
Total weight (kg)													412.74	31.24	1,894.71	4,107.87	12,535.50	

SUPER STRUCTURE--- Beam Bars

Location	Ref No.	Shape	f	Bar Length (mm)	Span Length(mm)	Spacing (mm)	No. of Bar	No Floors	No. of memb.	Diameter of Bars								
										6	8	10	12	14	16	20	24	30
			10	1500	23034	150	155	1	2			465.00						
			8	1500	11517	200	59	1	2		177.00							

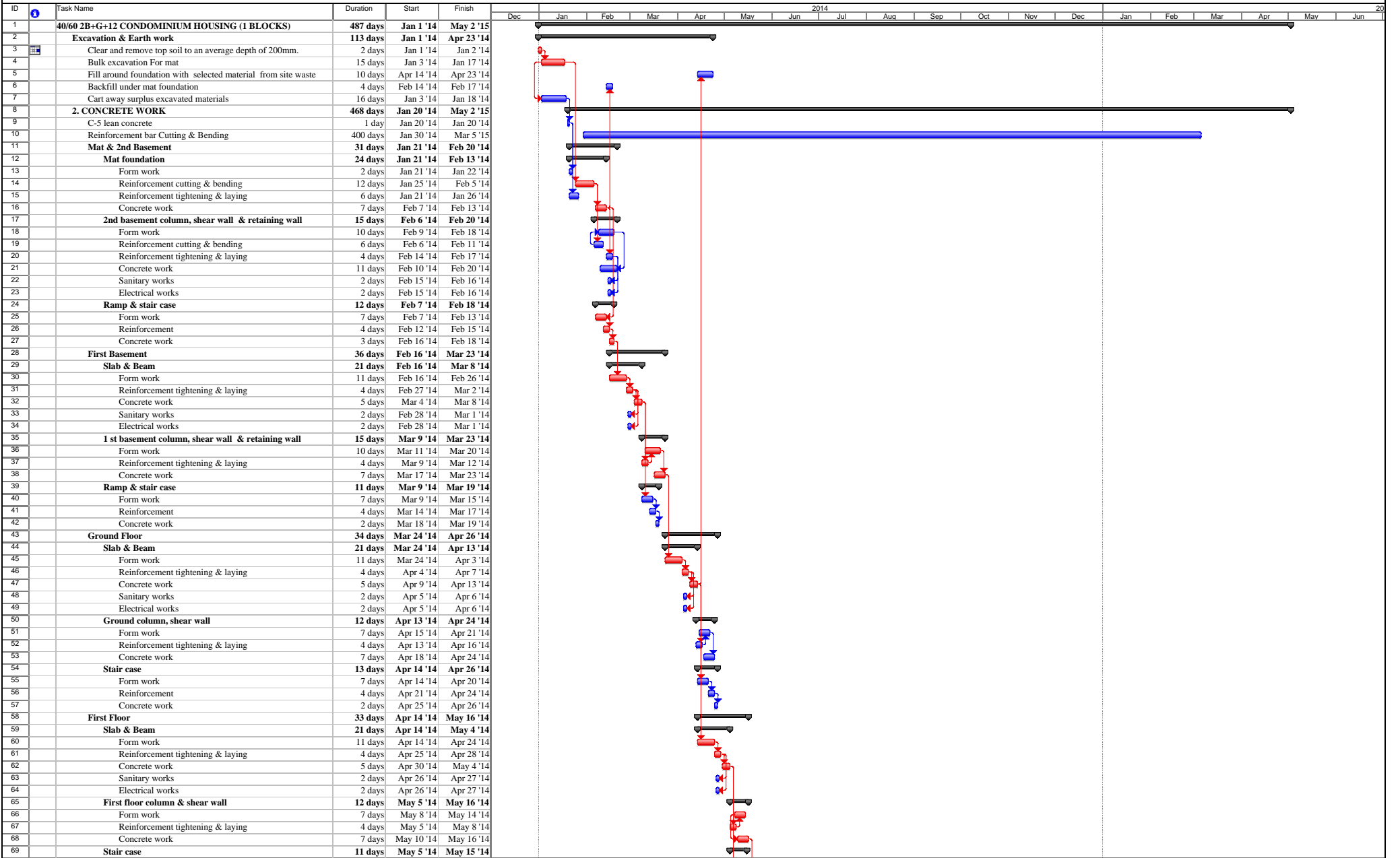
SUPER STRUCTURE--- Beam Bars

Location	Ref No.	Shape	f	Bar Length (mm)	Span Length(mm)	Spacing (mm)	No. of Bar	No Floors	No. of memb.	Diameter of Bars									
										6	8	10	12	14	16	20	24	30	
			10	1400	23034	130	178	12	2			5,980.80							
			20	1400	11517	190	62	12	2							2,083.20			
Axis 2&8 15			20	1400	6402	150	44	12	2							1,478.40			
			8	1400	3201	200	17	12	2		571.20								
Beam E'-D			10	1400	5478	150	38	12	7			4,468.80							
			8	1400	2739	200	15	12	7		1,764.00								
Axis 3,6&7 16			20	1400	4422	130	35	12	6							3,528.00			
			20	1400	2211	150	16	12	6							1,612.80			
Axis 4&5 15			20	1400	4422	120	38	12	4							2,553.60			
			10	1400	2211	150	16	12	4		1,075.20								
												2,335.20	21,033.60				17,740.80		
										0.222	0.395	0.617	0.888	1.209	1.579	2.469	3.555	5.55	
											922.40	12,977.73				43,802.04			

SUPER STRUCTURE--- Stirrups

Location	Ref No.	Shape	f	Bar Length (mm)	Span Length(mm)	Spacing (mm)	No. of Bar	No Floors	No. of memb.	Diameter of Bars									
										6	8	10	12	14	16	20	24	30	
Stirrups for Top-Tie Beam																			
Axis A			10	1400	5874	140	43	12	2			1,444.80							
			8	1400	2937	160	19	12	2		638.40								
Axis B			10	1400	19404	130	150	12	2			5,040.00							
			10	1400	9702	200	50	12	2			1,680.00							
Axis C			20	1400	23034	150	155	12	2							5,208.00			
			10	1400	11517	200	59	12	2			1,982.40							
Axis D			10	1400	23034	130	178	12	2			5,980.80							
			10	1400	11517	190	62	12	2			2,083.20							
Axis 2&8 15			8	1400	6402	150	44	12	2		1,478.40								
			8	1400	3201	200	17	12	2		571.20								
Beam E'-D			20	1400	5478	150	38	12	7							4,468.80			

Table B.11: Time schedule for the 40/60 housing project



Project: 40/60 2B+G+12 CONDOMINIUM HOUSING (1 BLOCKS)
Date: January 2014



ANNEX-C
STRAND SPECIFICATION MANUAL

Common specifications

Specifications after stress relieving

- Low relaxation ensures that no noticeable loss of tension will occur in time, therefore a long lasting compressive force on concrete. ArcelorMittal prestressing steel guarantees a very low relaxation.
- Deviated tension is current in post-tensioned structures, stay-cables and even some prefabrication methods. By strict adherence to manufacturing processes and routine destructive tests, WireSolutions' products exceed standard requirements.
- Stress corrosion and hydrogen embrittlement are known to be a threat for high-tensile structural steels, especially in environments containing chlorides. For several decades, WireSolutions has been producing wire and strands satisfying the most stringent demands in the industry in this field.

Maximum relaxation at 0.7 Rm (20°C) at 1 000 h	Reduction in area at rupture	Minimum elongation at maximum force	Deflected tensile test	Maximum curvature in a free state	Tension corrosion	
					1 test	6 tests
2.5 %	Ductile wire break visible to the naked eye	3.5 %	Post-tension < 28 Stay-cables < 20	< 25 mm/m	Strand < 9.3 mm Minimum 1.5 h	Strand < 9.3 mm Median 3 h
					Wire + strands ≥ 9.3 mm Minimum 2 h	Wire + strands ≥ 9.3 mm Median 5 h

Fatigue behaviour, 2 000 000 cycles for:

- Fatigue triggers quick, unexpected and costly rupture in structural steels. This is especially true with stay-cable strands, submitted to high levels of stress variation. Choice of steel quality, specific production processes and control allow ArcelorMittal steel to reach a high fatigue resistance.

Upper limit of nominal tensile strength	Test range
%	MPa
Stay-cables 45	Stay-cables 300
Strands 70	Strands Plain: 190 Indented: 170
Wire 70	Wire Plain: 200 Indented: 180

Options

Strand indentation					Resistance to low temperature, cryogenics*	Galvanisation Pr EN 10337
∅	a	a±	l	p		
< 12 ≥ 12	0.06 0.07	± 0.03	3.5 ± 0.5	5.5 ± 0.5	-170°C	From 190 to 350 g/m²



*Strands tested to customer's specifications

Technical data

7 wire strands EN 10138 – BS 5896

	Nominal diameter	Tensile strength	Mass	Cross sectional area	Tolerance on mass	Minimum breaking strength	Maximum breaking strength	Yield strength at 0.1% elongation
	mm	MPa	g/m	mm ²	%	kN	kN	kN
	6.85	2060	220.2	28.2	±2	58.1	66.8	51.1
BS	7	2060	234.3	30.0	±2	61.8	71.1	54.4
	8	1860	296.8	38.0	±2	70.7	81.3	60.8
BS	9.3	1860	406.1	52.0	±2	96.7	111.0	83.2
BS	9.6	1960	429.6	55.0	±2	102.0	117.0	87.7
BS	11.3	1860	585.8	75.0	±2	140.0	161.0	120.0
BS	12.5	1860	726.3	93.0	±2	173.0	199.0	149.0
BS	12.9	1860	781.0	100.0	±2	186.0	214.0	160.0
BS	15.2	1770	1086.0	139.0	±2	246.0	283.0	212.0
BS	15.2	1860	1086.0	139.0	±2	259.0	298.0	223.0
	15.3	1770	1093.0	140.0	±2	248.0	285.0	213.0
BS	15.7	1770	1172.0	150.0	±2	266.0	306.0	229.0
	15.7	1860	1172.0	150.0	±2	279.0	321.0	240.0

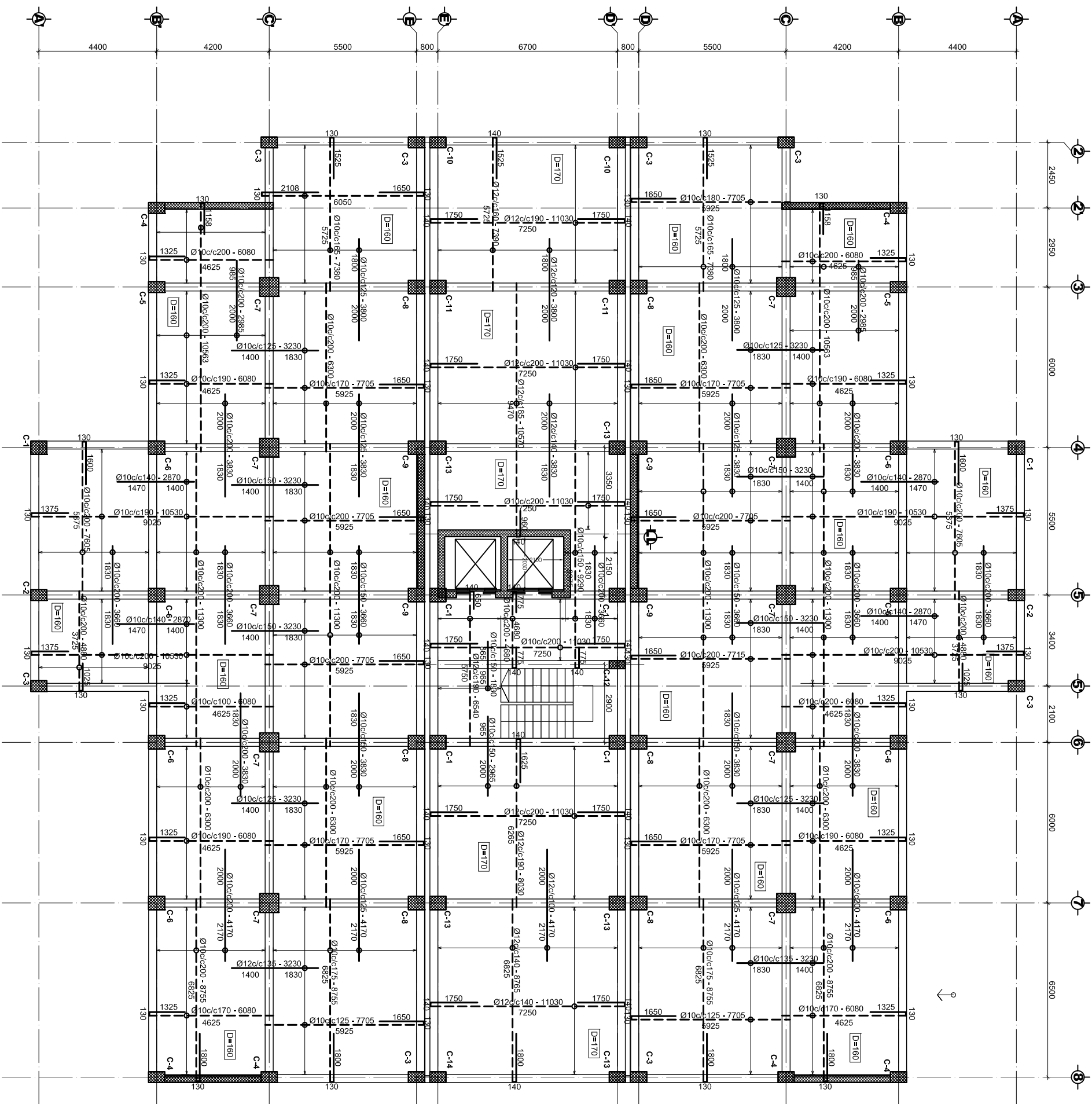
7 wire strands ASTM A 416/A 416 M

ø	Nominal diameter		Diameter tolerance		Grade		Nominal weight		Nominal steel area		Minimum breaking strength		Yield strength minimum load at 1% extension	
	inch	mm	inch	mm	ksi	MPa	lb/1000'	g/m	inch ²	mm ²	lbs	kN	lbs	kN
1/4	0.250	6.40	-0.016/+0.016	-0.40/+0.40	250	1725	122	182	0.036	23.2	9000	40.0	8100	36.0
5/16	0.313	7.90	-0.016/+0.016	-0.40/+0.40	250	1725	197	294	0.058	37.4	14500	64.5	13050	58.1
3/8	0.375	9.53	-0.006/+0.0026	-0.15/+0.65	270	1860	290	432	0.085	54.8	23000	102.3	20700	92.1
7/16	0.438	11.11	-0.006/+0.0026	-0.15/+0.65	270	1860	390	582	0.115	74.2	31000	137.9	27900	124.1
1/2	0.500	12.70	-0.006/+0.0026	-0.15/+0.65	270	1860	520	775	0.153	98.7	41300	183.7	37170	165.3
0.52	0.520	13.20	-0.006/+0.0026	-0.15/+0.65	270	1860	568	844	0.167	107.7	45000	200.2	40500	180.1
0.56	0.563	14.29	-0.006/+0.0026	-0.15/+0.65	270	1860	651	970	0.192	123.9	51700	230.0	46530	207.0
0.6	0.600	15.24	-0.006/+0.0026	-0.15/+0.65	270	1860	740	1102	0.217	140.0	58600	260.7	52740	234.6
0.7	0.700	17.78	-0.006/+0.0026	-0.15/+0.65	270	1860	1000	1487	0.294	189.7	79400	353.2	71500	318.0

3 wire strands EN 10138

	Nominal diameter	Tensile strength	Mass	Cross sectional area	Tolerance on mass	Minimum breaking strength	Maximum breaking strength	Yield strength at 0.1% elongation
	mm	MPa	g/m	mm ²	%	kN	kN	kN
3 x 2.40	5.2	1960	106.2	13.6	±2	26.7	30.7	23.8
3 x 2.40	5.2	2060	106.2	13.6	±2	28.0	32.2	24.9
3 x 2.40	5.2	2160	106.2	13.6	±2	29.4	33.8	26.2
3 x 2.91	6.3	1920	154.6	19.8	±2	38.0	43.7	32.7
3 x 3.00	6.5	1860	165.5	21.2	±2	39.4	45.3	33.9
3 x 3.00	6.5	1960	165.5	21.2	±2	41.6	47.8	37.0
3 x 3.53	7.5	1860	226.5	29.0	±2	53.9	62.0	46.4

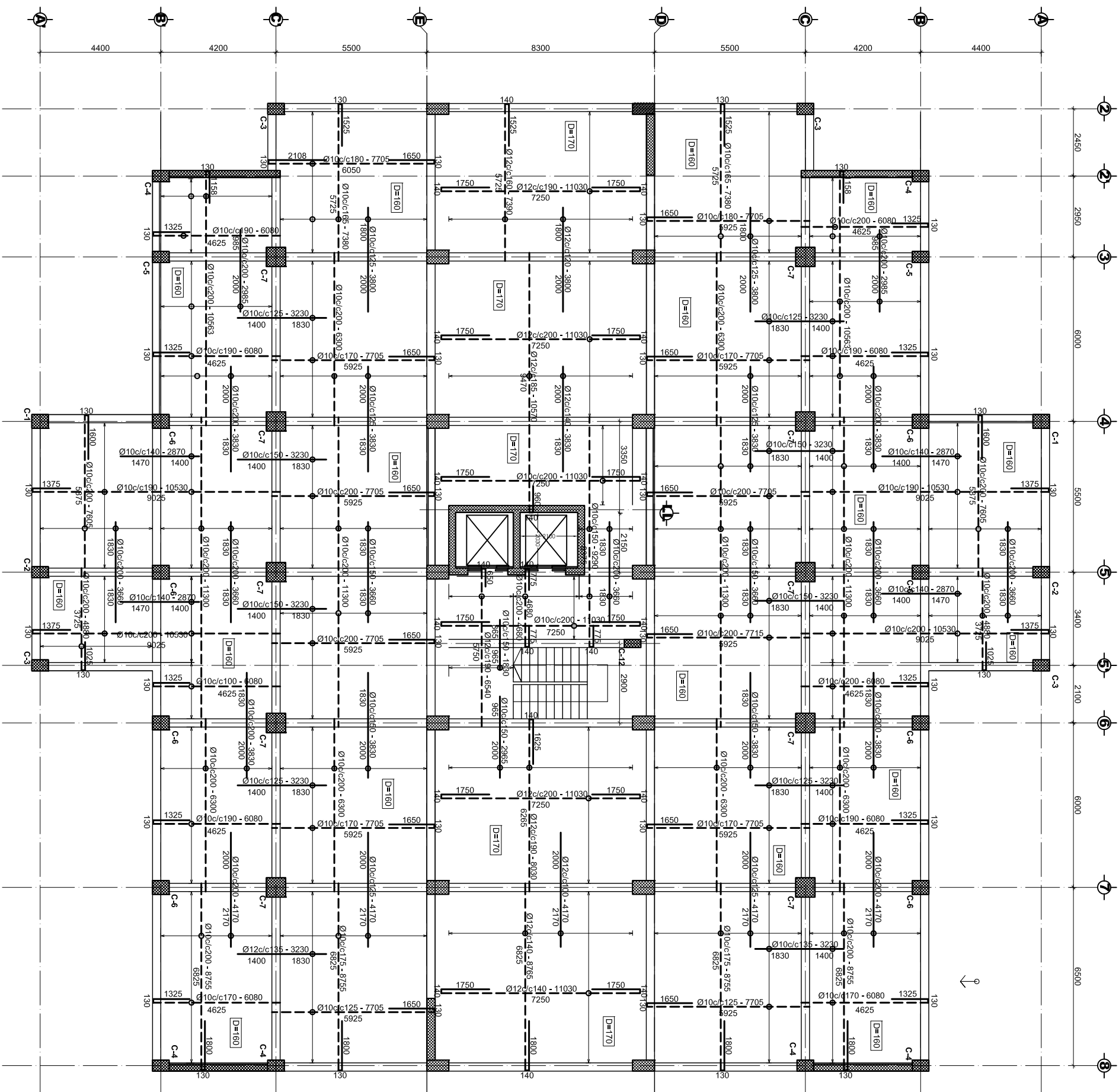
ANNEX-D
DRAWINGS



GROUND FLOOR SLAB REINFORCEMENT & BEAM LAYOUT @ LEVEL ±0.00
SC 1:100

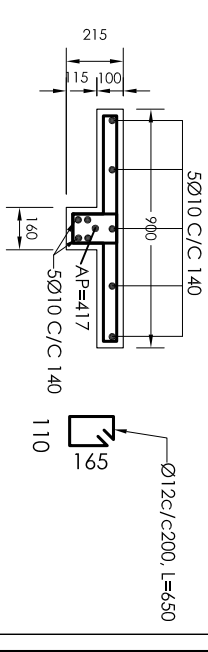
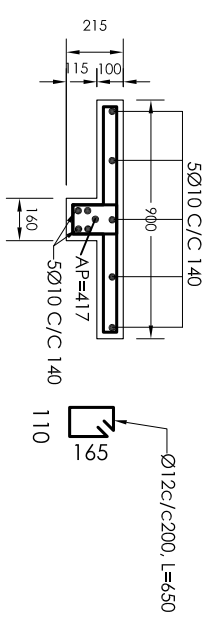
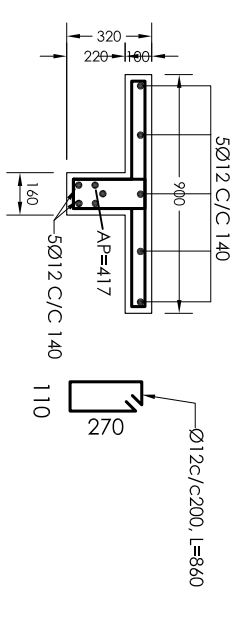
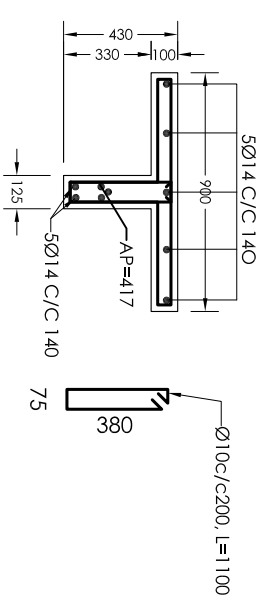
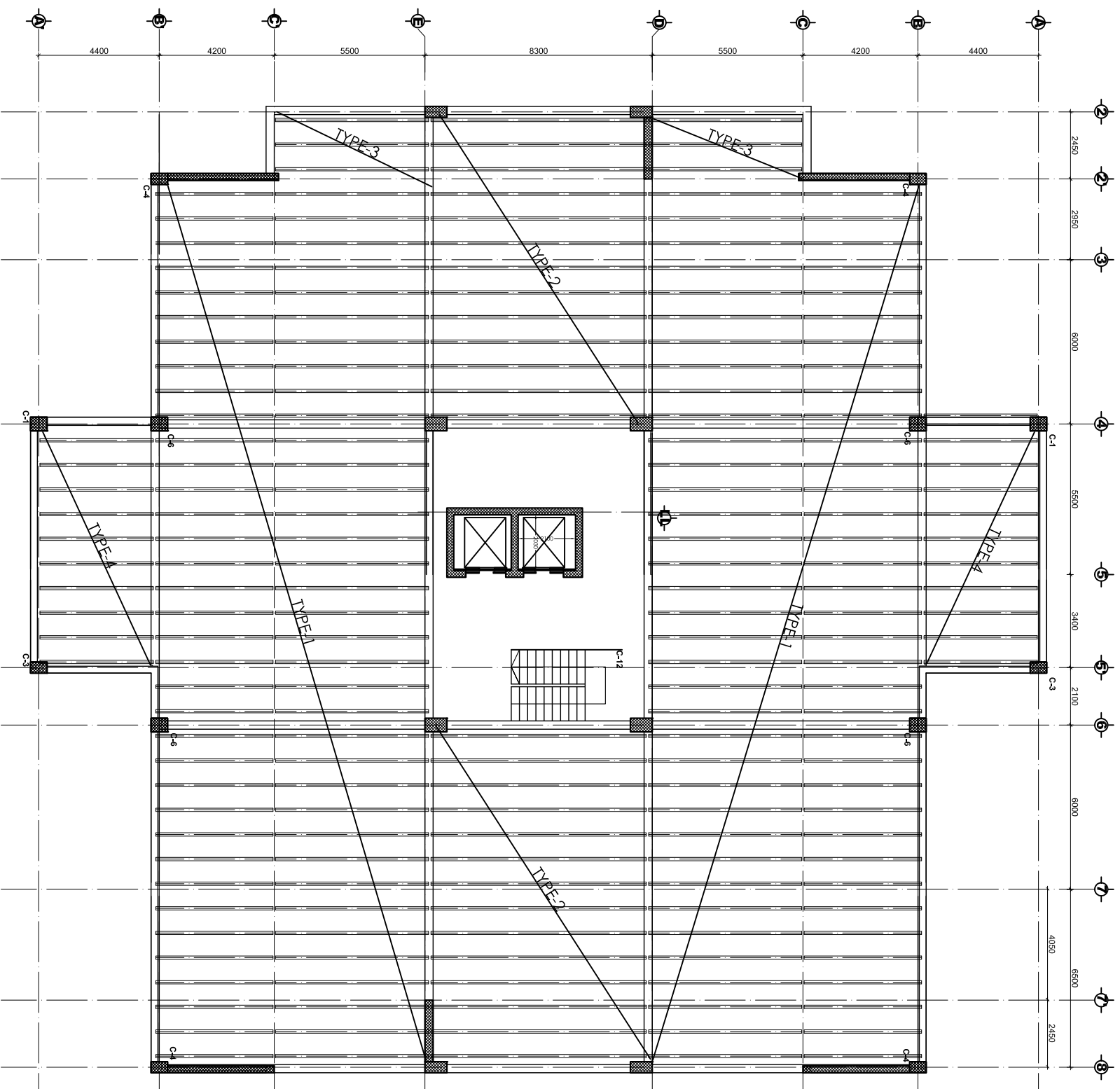


Project:	40/60
PROJECT STATUS:	PRELIMINARY <input type="checkbox"/> APPROVAL <input type="checkbox"/> WORKING <input type="checkbox"/>
Owner:	
Location:	ADDIS ABABA
Title D. No.:	
Dwg Title:	GROUND FLOOR SLAB REINFORCEMENT & BEAM LAYOUT
Design by:	Sign
CAD by:	Selamawit Ashenafi
Scale:	1:50



GROUND FLOOR BEAM LAYOUT PLAN @ LEVEL ±0.00
SC 1:100

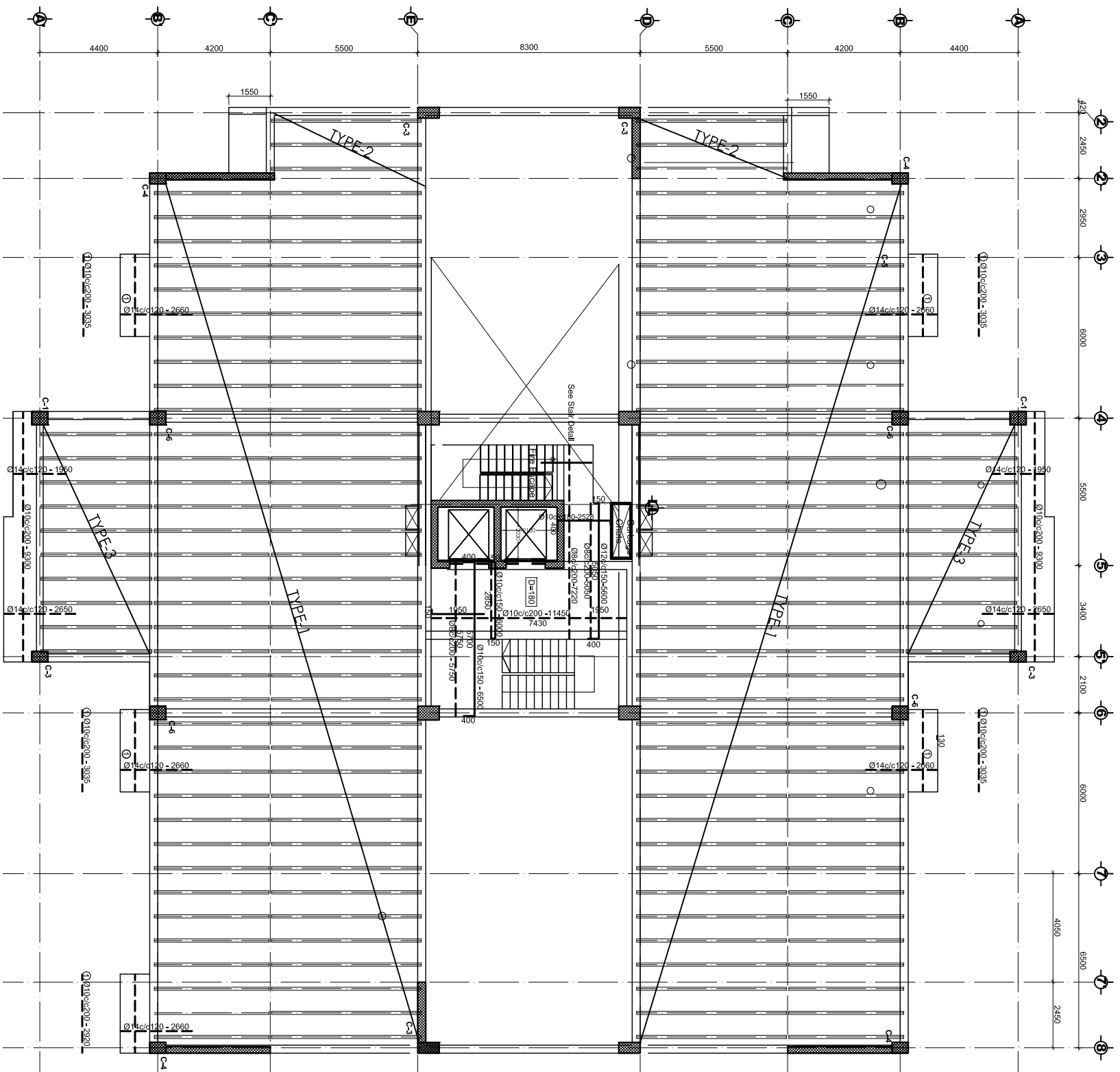
Project: 40/60	
PROJECT STATUS: PRELIMINARY <input type="checkbox"/> APPROVAL <input type="checkbox"/> WORKING <input type="checkbox"/>	
Owner:	
Location: ADDIS ABABA	Title D. No.:
Dwg Title: GROUND FLOOR RIBBED BEAM LAYOUT PLAN	
Design by: Sign	
CAD by: Selamawit Ashenafi	
Scale: 1:50	
Drawing No. ST	2/5



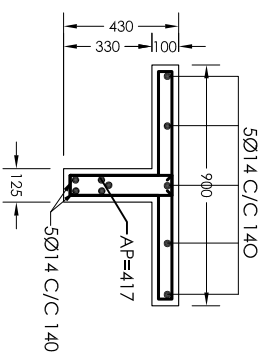
GROUND FLOOR T-BEAM LAYOUT PLAN @ LEVEL ±0.00 SC 1:100

Project:	40/60
PROJECT STATUS:	PRELIMINARY <input type="checkbox"/> APPROVAL <input type="checkbox"/> WORKING <input checked="" type="checkbox"/>
Owner:	
Location:	ADDIS ABABA
Title D. No.:	
Dwg Title:	GROUND FLOOR T-BEAM LAYOUT PLAN & DETAILS
Design by:	Sign
CAD by:	Selamawit Ashenafi
Scale:	1:50

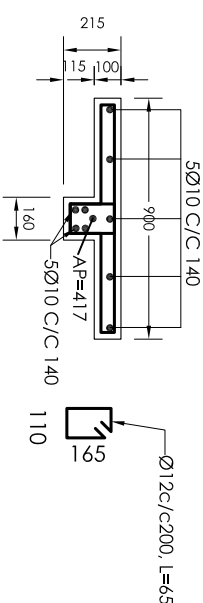
Drawing No.
ST
4/5



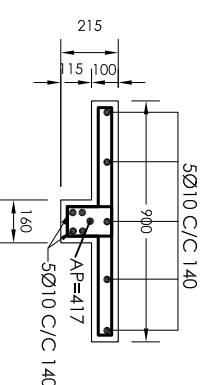
1 TO 12 FLOOR TYPICAL T-BEAM LAYOUT PLAN SC 1:100



TYPE -1 SCALE 1:20



TYPE -2 SCALE 1:20



TYPE -3 SCALE 1:20

Project:	40/60
PROJECT STATUS:	PRELIMINARY <input type="checkbox"/> APPROVAL <input type="checkbox"/> WORKING <input type="checkbox"/>
Owner:	
Location:	ADDIS ABABA
Title D. No.:	
Dwg Title:	1 TO 12 FLOOR TYPICAL T-BEAM LAYOUT PLAN & DETAILS
Design by:	Sign

Drawing No. **ST**
5/5
 CAD by: Selamawit Ashenafi
 Scale: 1:50

ANNEX-E

PRESTRESSED OR PRECAST SITE VIST DATA

SITE: CREC XINYUN Branch Mermersa site

LOCATION: Mermesa, Adama

SERVICE YEARS: 3 Years and a half

INDUSTRY TYPE: Bridge and Railway construction

1. Equipment type

- a) Prestressing equipment
 - Prestressing or stress releasing machines
- b) Transportation equipment type
 - Conveying machines, wheels or rail machines (train)
- c) Transportation equipment capacity regarding load and length
 - Available according to the type of the mold.
- d) Available cranes at site and manufacturing place
- e) crane capacity regarding load and length
 - 10Ton, 25Ton and 80Ton
- f) curing methods used
 - concrete water reducer
- g) available mold shapes
 - sleeper mold, girder mold and electricity poles mold

2. Service years for the equipments

- 10years

3. Maintenance

- varies

4. Capacity

- a) Working hours/day/week
 - 8/day/week
- b) Maximum Prestressing load
 - 60MPa

5. Source of prestressing force

- a) Mechanical b) Hydraulic c) Electrical d) Chemical

6. For what type of construction purpose are the produced units used?

- a) Bridge b) Railway sleepers c) Housing

d) Other (specify): electric poles, sleepers

7. What is location of the prestressing tendon with respect to the concrete section?

a) External prestressing

b) Internal prestressing

8. Which system is usually applied?

a) Pre-tensioning

b) Post-tensioning

9. Amount of prestressing

a) Full prestressing

b) Limited prestressing

c) Partial prestressing

11. Direction of the prestressing member

a) Uniaxial

b) Biaxial

c) multiaxial

12. Concrete casting mechanism

a) Cast in situ

b) Pre-cast members

c) Composite structural members

13. Material data

- a) Concrete grade used: C60
- b) Mix design: depends on the material at hand
- c) Strand type: imported from china
- d) Strand grade: 1570Mpa
- e) Strand sizes available: $\varnothing 5\text{mm}$ - $\varnothing 7.5\text{mm}$
- f) What is the strand cost?

Confidential information

ANNEX-F
PICTURES

PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS



PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS



PRE-CAST PRESTRESSED CONCRETE SYSTEM FOR THE HOUSING PROJECTS

