ASSESSMENT ON THE POTENTIAL USE OF PRECAST LATTICE SLAB FOR THE LOW COST HOUSING PROJECTS IN ADDIS ABABA

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A Thesis submitted to

The School of Civil and Environmental Engineering

Presented in Partial Fulfillment of the Requirements for the Degree of Masters of Science (Structural Engineering)

Addis Ababa University

Addis Ababa, Ethiopia

January, 2021

ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

As members of the examining board of the final M.Sc. Open Defense, we verify that we have read and evaluated the thesis prepared by **Bethel Alebachew** entitled: **Assessment of the Potential Use of Precast Lattice Slab for the Low Cost Housing Projects in Addis Ababa** and recommended for acceptance as a fulfilment of the requirement for the degree of **Master of Science in Civil Engineering (Structural Engineering)**

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DECLARATION

I, the undersigned, declare that the thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for the thesis have been duly acknowledged.

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ACKNOWLEDGEMENTS

I would like to acknowledge the effort of different scholars, Authors, Institutions and so many individuals who directly and indirectly contributed to the successful completion of this research.

I would like to acknowledge my respected advisor Dr. Girma Zerayohannes for his insightful direction and guidance throughout the project. His supports have been invaluable in realizing this project.

Lecturer Asmerom Gezahgn from Civil Engineering department of Addis Ababa Institute of Technology, I truly respect and appreciate his contribution to this project, moreover, it was his idea to incorporate light weight concrete in the research. He was always very welcoming and working with him has been a great privilege.

I would really like to thank all staff members of MH Engineering, Bamacon Engineering, Habtamu International Consulting Architects and Engineers and Ethiopian Building Construction Corporation as they have voluntarily participated in the data collection stage of the research.

I would also like to acknowledge Ato Mesfin, Director of Quality Control and Product Management of Ethiopian Construction Works Corporation and Ato Fetene, Section Head of Contract Administration and Project Coordination of MH Engineering for being volunteers of the interview.

Ato Henok Yitbarek, Department Head of Research Center in Meles Foundation, has contributed insightful comments in the field of research methods; his involvement in this project is very much appreciated.

Last, but not least, I would like to thank my family for supporting me and bearing with me even when I was intolerable. Especially my husband, Ato Tagel Markos, has been very supportive of this work and a good cheer.

ABSTRACT

This research mainly focuses on the development of precast concrete technology in Ethiopia. The research has a quantitative and a qualitative part. In the quantitative part, a new form of slab known as Precast Lattice Slab is adopted and used for a G+9 low cost building. The slab is constructed by using composite method of construction incorporating precast plates and fresh concrete topping. The plates are designed as a series of one way slabs mounted on the beam grid system. All design and analysis calculations in this thesis are as per the European Code. The procedures to be followed in analyzing and designing this precast lattice slab along with cost and weight comparison between the newly advised method and the conventional method used for low cost building projects are presented in this section.

In the qualitative part of the research, market assessment was carried out to determine what caused the low demand for precast technology in Ethiopian construction society (does not include the low cost building projects). Observation, questionnaire, interview and document analysis were used as a data collection tools. Authorities in the area including MH Engineering and Ethiopian Construction Works Corporation have been communicated and relevant inputs were gathered. After a thorough analysis of the data gained from the data collection stage, recommendations are given that enhance the use of precast technology in Ethiopia for all kinds of buildings.

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1. INTRODUCTION

1.1. General Background

The constantly changing requirements of modern architecture and the growing need for rationalization in civil engineering drive the search for constructive systems that satisfy these aspects while ensuring the structure's ability to meet the requisites of load-bearing capacity, service performance and durability.

Prefabrication is among the primary economical building solutions. As the industry strategizes itself to build with less labor and shorter construction time, prefabrication of concrete structures has become a viable alternative to the traditional way of construction. Over the last few years, some industry players have effectively adopted use of precast concrete components to their advantage by combining it with cast in-situ concrete elements. Different mix of precast and cast in-situ elements are used to meet different design requirements for better quality and cost effectiveness. Such combination enables their projects to achieve higher level of productivity than is possible with solely cast in-situ construction. By using precast concrete components predominantly, on-site operations are considerably reduced, providing a safer working environment [1].

Precast concrete buildings previously had an identifiable appearance. However, with advances in technology and technique, precast buildings are now indistinguishable from those constructed using non-precast methods. Designers no longer work within tight constraints. Increasingly, the situation is that precasters are able to accommodate greater variety and complexity, effectively designing their elements to meet design requirements. Continuous investment and innovations have transformed the precast industry so that complex plan layouts and external treatments can be accommodated.

Precast concrete can now be incorporated in every building type. Whether the building has a regular or an irregular shape, the entire structure or elements of that structure, such as frame, floors, walls, stairs or balconies, can all be precast. Precast construction is virtually unlimited in its application and is suitable for single and multi-storey construction. Offices and hotels are commonly constructed in precast, as are hospitals, schools, industrial units and multi-storey car parks, apartments and housing. In fact, precast building elements should be considered as an option on every construction project [2].

The Code of Practice for the Safe Handling, Transportation and Erection of Precast Concrete [3] gives clear and detailed recommendations on the design, manufacture, handling, transportation, erection, propping and rigging procedures concerning precast concrete elements.

According to the code, manufacturing process of precast concrete members has two sub stages known as the Pre-production stage and the Production Stage. In the pre-production stage, contract signing between the manufacturer and the client, schedule for production, identification of builder's preferred product handling methods, inspection of propping and support requirements, estimation of lifting and handling stresses, and check for stability and buckling are to be dealt with. Where as in the production stage, design of moulds, surface finish requirements, assembly and release details, strength requirements for lifting, transport and erection and finally stacking and storage conditions are to be given a due consideration.

Many countries used various precast building systems during the second half of the 20th century to provide low income housing for the growing urban population[4]. They were very popular after the Second World War, especially in eastern European countries and former Soviet Union republics. In the former Soviet Union, different precast building systems are denoted as "seria" where as in Romania they are called "sectiunea."

When the focus is narrowed down to Ethiopia, precast concrete technology is not very popular. The only party which is responsible for the fabrication of precast concrete building elements in Ethiopia is Ethiopian Construction Works Corporation (ECWC). According to the official website of the institution[5] , the Prefabricated Building Elements Manufacturing Institution was first established based on the economy agreement between Ethiopian and Yugoslavian government in 1984EC. Though the institution produced quality products by the time, it was facing financial problems as there was almost no need for the products in the market. As a result, in 1993EC, the institution had to be re-established by the help of the government into Ethiopian Construction Works Corporation. It has been a total of 33 years since the establishment of the institution and within these years the institution has been able to build only 40 buildings. The institution fabricates prestressed building elements such as column, beam, shear wall, slab and footing pads by using C-30 and C-40 concrete grades. Currently, it is involved in 4 projects including the rehabilitation project dedicated for the people who tragically loss their homes in "KOSHE" area. In general, as the institution is the only company which produces precast concrete elements in the country, it can easily be seen that the demand for precast concrete elements is very unsatisfactory.

On the other hand, ever since the low cost housing projects in Ethiopia begun, the use of Composite (combination of precast and cast insitu) reinforced concrete slab specifically that of ribbed slab has developed a great deal. Cost efficiency is one of the most crucial points of low-cost housing. It can mainly be achieved by standardization of building elements and reducing the number of different items needed. Prefabrication and the use of machines and special tools to produce these standardized elements maximize productivity, resulting in lower costs per unit.

Based on the report published by the German company GTZ on the low cost housing projects [6], the Low-cost Housing Project is established, based on a bilateral agreement between the Federal Democratic Republic of Ethiopia and the Federal Republic of Germany. It is implemented by the Ethiopian Ministry of Federal Affairs with the support of GTZ (German Technical Co-operation). While the Ministry of Federal Affairs is the Owner of the Project, the Partners on regional and local level are National Regional States, Regional Bureaus of Works and Urban Development, Regional Construction and Design Authorities, Urban Development Offices and Municipalities.

According to the report, the demand for the services of the Low-cost Housing Project is increasing daily. 85% of the urban population of Ethiopia lives in inhuman, unhygienic and confined conditions. Their housing situation lacks infrastructure and is dominated by “chicka” type of construction (traditional construction method with mud and wood). The population growth of 2.8 % per year and the accelerated migration to urban centers (6 % and more per year) have dramatically increased the demand for affordable, decent housing.

Even though the demand for the low cost housing projects is continually rising, the quality of the buildings is at the same time degrading by the day. Some buildings and building parts have even faced alarming types of failure; the “Jemo Condominium Site” can be taken as an example. The quality loss of the low cost buildings has also negatively influenced other construction sectors not to use composite reinforced concrete slabs for their building projects.

1.2. Statement of the Problem

The demand for Composite Concrete Slab on the low cost housing projects is very high with low competitiveness resulting from its low quality and relatively high prices and therefore the low cost housing projects are no more of low cost. On the contrary, the demand for precast technology in other construction industries of the country, is known to be low as the only precast fabricating company in Ethiopia (Ethiopian Construction Works Corporation) has only been involved in the construction of 40 buildings within 33 years of existence.

1.3. Objective of the research

- **General objective**

The general objective of this research is to adopt and customize the design of a new type of composite reinforced concrete slab (Precast Concrete Lattice Slab) which has a relatively higher quality and lower cost than the conventional method of construction for the low cost housing projects while assessing what caused the low demand of precast concrete building elements on the construction market other than low cost housing projects; and give recommendations so as to expand the new product (precast concrete lattice slab) on the market.

- **Specific objectives**

- Perform analysis and design of precast lattice slab using normal weight concrete.
- Perform analysis and design of precast lattice slab using light weight concrete.
- Execute cost and weight comparison between the conventional method of construction for low cost housing projects and the new one with precast lattice slab.
- Assess why the demand of precast concrete elements for other projects is low on the current construction market of Addis Ababa.
- Based on the result of the market assessment, give recommendations which would help in expanding precast lattice slab for other projects.

1.4. Methodology

The research is of mixed type (both quantitative and qualitative). The quantitative part of the research deals with the analysis and design of precast concrete lattice slab by using both light and normal weight concrete and the qualitative part focuses on the market assessment part.

As mentioned above, the quantitative part of the research covers the analysis and design of the adopted precast concrete lattice slab according to the European Code. The cost and weight comparison between the conventional and the new method is also covered in this section. The software SAP 2000 V14 and ETABS 9.7.4 are used for analysis and design in addition to manual calculation while AutoCAD 2015 is used for drafting.

The qualitative part of the research follows a descriptive design and has three parts named data collection, data presentation and data analysis. In the data collection stage, the data collection tools used are observation, interview, questionnaire and document analysis. These sources of data are generally categorized as primary and secondary sources of data. Under primary sources of data are observation, interview and questionnaire where as under secondary sources of data is literature analysis. The sampling technique used to distribute the questionnaires is availability sampling technique.

1.5. Scope of the Research

- In this study, the analysis and design of the precast lattice slab is done at a theoretical level and laboratory test is not executed.
- Since the analysis and design of ribbed slab is not the focus of this research, for cost and weight comparison, an already designed ribbed slab was used.
- The cost and weight comparison between the conventional and the new method covers only the beam-slab system of a single typical floor; it does not include other building parts such as columns, walls or foundations.
- In this research, the decision for size and placement of the reinforcement is done on the basis of the commonly used arrangement and size of steel bars for the current construction.
- In the data collection process, due to time constraint, the questionnaires were distributed among a limited number of organizations located in Addis Ababa.

1.6. Organization of the Research

This research is organized to have two fundamental parts, Quantitative part and Qualitative part. The first five chapters are under the category of quantitative study and the sixth chapter is under qualitative study.

In the first chapter of the research, it is aimed to provide the reader with a general background regarding the history, development, advantages and disadvantages of precast technology as compared to insitu method of construction at a global level. The progress and gaps observed regarding precast technology in Ethiopia are also stated in this chapter.

In the 2nd chapter, a literature review of important achievements such as inventions, new technologies and researches regarding precast method of construction are mentioned. Theoretical investigations on precast lattice slab including the two types of concrete to be used and its stages of construction are discussed in this chapter.

In the 3rd and 4th chapter, the procedures to be followed when analyzing and designing precast lattice plates along with a design example for both normal weight and light weight concrete is presented and finally, on the last chapter of the quantitative part (chapter 5), cost and weight comparison between ribbed slab and precast lattice slab is performed.

In chapter 6, the market assessment carried out to determine what caused the low demand for precast technology in the current Ethiopian construction society is studied. the different types of data collection tools used and procedures are mentioned here. In addition, the data gathered along with its analysis are presented in this chapter.

Finally, in chapter seven, conclusions and recommendations are given based on the results gained from the previous chapters.

2. LITERATURE REVIEW

2.1. General

Ancient Roman builders made use of concrete and soon poured the material into moulds to build their complex network of aqueducts, culverts, and tunnels. Modern uses for precast technology include a variety of architectural and structural applications including individual parts, or even entire building systems.

In the modern world, precast paneled buildings were pioneered in Liverpool, England, in 1905. The process was invented by city engineer John Alexander Brodie, a creative genius who also invented the idea of the football goal net. The tram stables at Walton in Liverpool followed in 1906. The idea was not taken up extensively in Britain. However, it was adopted all over the world, particularly in Eastern Europe and Scandinavia [7] .

The technology of using lattice girders for precast concrete elements is known to have three stages of development:

a) Development of the "new" industrial precast system (1965 – 1985)

The lattice girder (main part of the so-called half-precast system) was developed at the beginning of the fifties of the last century. In the middle of the 60s, the lattice girder was used for precast products, which was the starting point for the industrial production of precast elements on a broad basis. The production units were very simple at that time, consisting mostly of fixed beds in open areas and in basic production buildings. Tower Cranes in open space or Overhead Cranes in the building were the most important tools for the production.

b) Development of the "new" industrial precast system (1985 – 2005)

The second phase of development was characterized by two inventions. The developments were on the one hand the personal computer in connection with CAD (computer-aided design) systems to generated electronic data of the product and on the other hand the PLC control system (PLC = programmable logic controller) to control automated machines. These developments made it possible to implement automated machines (e. g plotters, concrete spreaders) into the production process and control the process using CAD data. This was also the hour of birth of CAD controlled production. In connection with the pallet carousel systems (invented in the 60s), it was possible to reduce the man hours per m² by two thirds compared to manual production on beds. Simultaneously the quality of the product was improved (surface, edges).

Step by step, automated machines were added to the production process. For example: automated production of reinforcement (mesh welding machines), de-molding and molding robots, laser projection, automated concrete spreaders and master computer systems to guide and control the production process.

c) Development since 2005 and outlook on the upcoming years

Since the year 2000, there has been a number of developments dealing with the digitalization and networking of production. In parts, these methods are already used, but not on a broad scale and not in every country.

A number of researches have been made on the area of precast concrete elements. But more specifically, important researches which are in regard to precast lattice elements are to be discussed below.

By the year 2001, in the research "Lattice Girder Elements in Four Point Bending" by a Swedish Engineer Ingemar Lofgren [8], tests were made to develop an analytical model which would help in the design of lattice girder elements and the tests indicate that the suggested model can be used to calculate flexural strength and stiffness. It was also discovered that the sections undergo three stages when subjected to load. The initial response, for which the section is un-cracked and the load-deformation curve is nearly linear. Then the second phase is initiated by the cracking of the concrete in tension. Here the response becomes non-linear, and the flexural stiffness starts to decrease. Cracking starts at a relatively low load and the total response is to a great extent influenced by this. The last phase of the response is reached when the top chord starts to exhibit buckling. This takes place at rather low stresses, which primarily depend on the slenderness of the top chord. When the stresses in the top chord reach the buckling stress the stiffness of the truss will decreases until the maximum load is reached. The analytical model can predict the behavior up to this point, but not thereafter.

Another remarkable Steel Construction Institute (SCI) publication in the year 2007 is by A.G.J. Way et al, with the title "precast concrete floors in steel framed building" [9]. It provides best practice information on detailing, construction methods and how to satisfy building regulations for the use of precast floor units including hollow core slabs and lattice plates in hot rolled steel framed buildings. It explains how maximized benefits including construction and design advantages can be gained by using precast concrete floor elements in conjunction with structural steel frame.

A statistical research in Brazil entitled "Design, manufacture and construction of buildings with precast lattice-reinforced concrete slabs" by J. R. Figueiredo Filho and A. K. H. Shiramizu in the year 2011 [10], gathered opinions of manufacturers, master builders and designers responsible for calculating, dimensioning and executing precast lattice slabs. The results of the survey showed that the main problems observed were: bulges formed on the soffit of precast lattice-reinforced slabs, which are characteristic of breakage or sagging of the filler element at some point, especially when clay blocks are used; fissures in the slab, concreting pockets and excessive strains were also among the problems most frequently found at these construction sites; and application of counter deflections; it is difficult to reach the specified value, especially if it is high, because the ends of the joists usually become detached from their supports. Finally, the paper suggests for lattice-reinforced slab to become ever more competitive, requires the adoption of measures to improve its design, production and construction processes, as well as the development of studies aimed at improving techniques, procedures and materials. Moreover, all those involved in the process of fabrication, design and construction of lattice reinforced slabs should take the necessary measures to reduce difficulties and prevent errors that lead to pathological problems.

In Ethiopia, a couple of researches were made regarding the precast technology adopted for the low cost housing projects. In the research "Simplified Analysis and Design Considerations of Precast Joist in One Way Ribbed Slab" by Sophonyas Asrat [11], the condition in which buckling of top reinforcement, buckling of stirrups, yielding of bottom reinforcement that occurs in the initial stage and the conditions up to the final stage in the precast joist system were assessed. In his research, he was also able to determine the number of temporary intermediate supports required for the initial stage of construction of the precast joist.

Another paper by Melese Yohannes is on the title "Investigation on the Suitability of Pumice and Scoria Aggregates in Ribbed- Slab Construction" [12]. In this thesis, Experimental and theoretical investigations were conducted to study the strength and deformation characteristics of ribbed slabs with 60 mm topping. The experimental program was dedicated to the study of mix design for the production of all lightweight structural concrete and full size ribbed slab testing. Theoretical investigations were conducted parallel to the experimental investigations. From the experimental investigations, it was concluded that mixtures developed with scoria aggregates have attained required strength and density to be accepted as structural lightweight concrete; however, pumice cannot be used as structural concrete.

Different countries have adopted different codes and guidelines for the analysis and design of precast concrete elements. In the US, precast concrete has evolved as two sub-industries, each represented by a major association. The precast concrete products industry focuses on utility, underground and other non-prestressed products, and is represented primarily by the National Precast Concrete Association (NPCA). Whereas the Precast/Prestressed Concrete Institute (PCI) focuses on prestressed concrete elements and on other precast concrete elements used in above-ground structures such as buildings, parking structures, and bridges.

Since the scope of this research focuses on the European Code, focus will be given to the recent version (EN 13747:2010) of this code concerning precast concrete products (Floor plates for floor systems) [13] . The code defines floor plate with lattice girders as "floor plate in which continuous lattice girders are incorporated generally in the longitudinal direction (i.e. parallel to the span) to provide strength and rigidity for transient situations." It states the material and production requirements for the constituent materials of concrete, the reinforcing steel and the lattice girder element. It provides the appropriate positioning of reinforcement bars, production tolerances and minimum dimensions along with vivid figures. It has also included the requirements for acoustic, thermal and fire resistance performances. Finally in the Annex part of the code, detailing of reinforcement layouts, connections, manufacturing requirements and design procedures are specified.

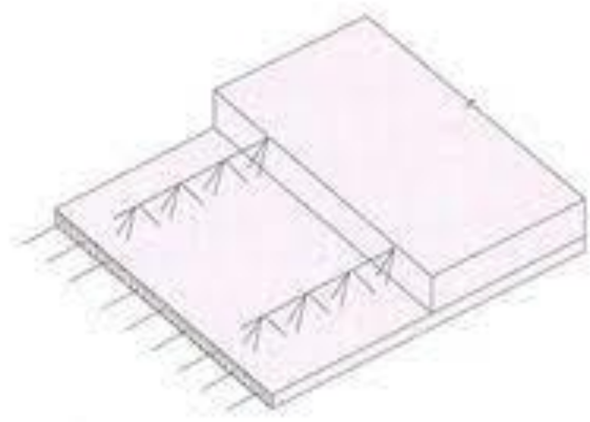
2.2. Theoretical Investigation

As a solution to the problem of low quality and relatively high price observed in the low cost housing projects, a new form of slab known as Precast Lattice Slab is adopted in this research. The European Code[13] defines floor plate with lattice girders as "floor plate in which continuous lattice girders are incorporated generally in the longitudinal direction (i.e. parallel to the span) to provide strength and rigidity for transient situations."

In this research, two types of precast lattice slabs are considered, one made of normal weight concrete (C-25) and another made of light weight concrete (LC25/28). According to the research "Investigation on the suitability of pumice and Scoria Aggregates in Ribbed-Slab Construction" by Melese Yohannes, mixtures developed with scoria aggregates attain required strength and density to be accepted as structural lightweight concrete.

The experimental investigations in the research show that ribbed slabs can be constructed using scoria aggregates with 40 - 47% fine content, 1.6 – 1.8 A/C ratio and 0.4 W/C in range of normal cement content. In the study, the concrete made by using scoria as both coarse and fine aggregate with the following mixing data:

a) Precast lattice plate



b) Arrangement of reinforcements within plates

c) Lifting of precast lattice plate

d) Erection of precast lattice plates

Figure 2.1: Precast lattice slab construction

Fine Aggregate = 47%, PPC = 400kg/m^3, Coarse Aggregate = 424 kg/m^3, Fine Aggregate = 376 kg/m^3, Water = 180.4lit, W/C Ratio = 0.45, A/C ratio = 2 resulted in Average Compressive Strength of 29.33MPa and a density of 1656.2kg/m^3 [12]. By assuming the same concept works for precast lattice slab, the previously mentioned data is adopted for the light weight concrete used in this research.

The analysis and design of the precast lattice slab comprises of three stages. The first stage is the *Production stage* which mainly represents the manufacturing phase of the precast plates. The second stage is the *Construction stage* which comprises of the erection of the precast plates and the concrete pouring phases on the construction site. While the third stage is the *Final stage*, representing the phase after the hardening of concrete and the building is serving its purpose.

For the analysis and design procedure involved in this research, Limit State Design (LSD) approach is used. The method is based on the limit state design philosophy. This design philosophy considers that any structure that has exceeded a limit state for which it was designed is unfit for the intended function or use. The limit state may be reached because the structure is in danger of collapse (ultimate limit state) or because excessive deflection has resulted the structure to be unable to carry out its design functions (serviceability limit state). Other limit states may be reached due to vibration, cracking, durability, fire or various other factors, which mean that the structure can no longer fulfill the purpose for which it was designed. The Ethiopian Building Code Standard (EBCS) has adopted the use of LSD method. Hence, in this research both Ultimate Limit State (ULS) for flexure and shear and Serviceability Limit State (SLS) for deflection are considered for all the three stages of design.

For the modeling of precast concrete slabs in the construction and final stage, ETABS 9.7.4 is used and for the modeling of the latticed reinforcement represented by 3D frame in the production stage, SAP 2000 V14 is used. The research is done according to the European Code.

2.3. Loads on the Precast Lattice Slab at Different Construction Stages

In general, to analyze and design a structure, it is necessary to have a clear picture of the nature and magnitude of the loads applied to the structure. The following discussion covers the primary loads that must be considered for the precast lattice slab and ways of describing and characterizing them.

As discussed previously, the stage of construction of the precast slab can be classified into three. The expected external load is calculated for all the production, construction and final stages of the precast slab. In the production stage, to determine the capacity of the precast plate, the analysis and design output from software is used and for the construction and final stage, analysis output from software followed by manual design calculation according to EN 1992-1-1:2004 is executed.

2.3.1. Production Stage

This phase covers the process of manufacturing of the 0.06m thick precast lattice plate along with the lifting, the transportation and the handling of the cured plate. In this phase the plate is subjected to self weight (dead load) with no additional live load.

2.3.2. Construction Stage

This stage has two sub stages named the Erection Stage and the Concrete Pouring Stage. It is the time of construction which is between the delivery of the produced plates to the construction site and the setting of fresh concrete topping. Temporary supports are used in this stage to support the slab. The number of props used to support a single plate depends on the load to span ratio and diameter of top bar.

2.3.2.1. Erection Stage

In this stage, constructors erect the precast lattice plates in their permanent position. Here the self weight of the plank is considered as dead load and the load of the workers as live load which is taken to be $1kN/m^2$ [11].

2.3.2.2. Concrete Pouring Stage

While estimating the design load in this stage, two alternatives should be considered.

i. The concrete as live load on the poured side. Since the concrete will not be usable for workers before setting, the workers load and the fresh concrete cannot be counted as live load together.

ii. The concrete part could set and the construction workers can use it as a plat form. Therefore, the construction workers as live load and the concrete as dead load is the other alternative.

Among the two alternatives under the concrete pouring stage, the second option governs and hence calculations for the construction stage are as per the governing sub stage.

2.3.3. Final Stage

This stage characterizes the slab after the hardening of the fresh concrete poured in the construction stage. The full depth of the slab is to be considered in this phase. The building considered in this research is residential and hence a live load of $2kN/m^2$ is used [14].

3. ANALYSIS AND DESIGN OF PRECAST SLAB ELEMENTS

3.1. Arrangement and Naming of Precast Plates

A G+9 low cost building with a typical slab is considered for this thesis. The slab was already designed as a ribbed slab spanning in the shorter direction. As shown in the figure below, since the layout of the slab has a symmetric nature, only one side of the slab is considered for design (between Axis A and Axis D; and between Axis 1 and Axis 8). By taking the same design constants, the normal weight and light weight precast lattice slabs are to be designed. In both cases the plates run in the shorter direction. After the analysis and design of both types of precast lattice slabs, they are to be compared with ribbed slab in terms of weight and cost.

Commonly the standard width sizes used for precast lattice slab plates throughout the world are 1.2m and 2.4m. The plates considered in this research are of three sizes based on the architectural design of the building under study. In all the three cases, since a width of 2.4m is difficult to manage, a standard width of 1.2m is used.

The first plate is of size 5.5m × 1.2m × 0.2m (L × W × D) while the second and the third plates have a size of 4.5m × 1.2m × 0.2m (L × W× D). The main difference between Plate-2 and Plate-3 is the size of their effective depth (d). Plate-2 has an effective depth of 0.17m since the bar size used for main reinforcement is of diameter 10mm, whereas Plate-3 has an effective depth of 0.171m since the bar size used for main reinforcement is of diameter 8mm.

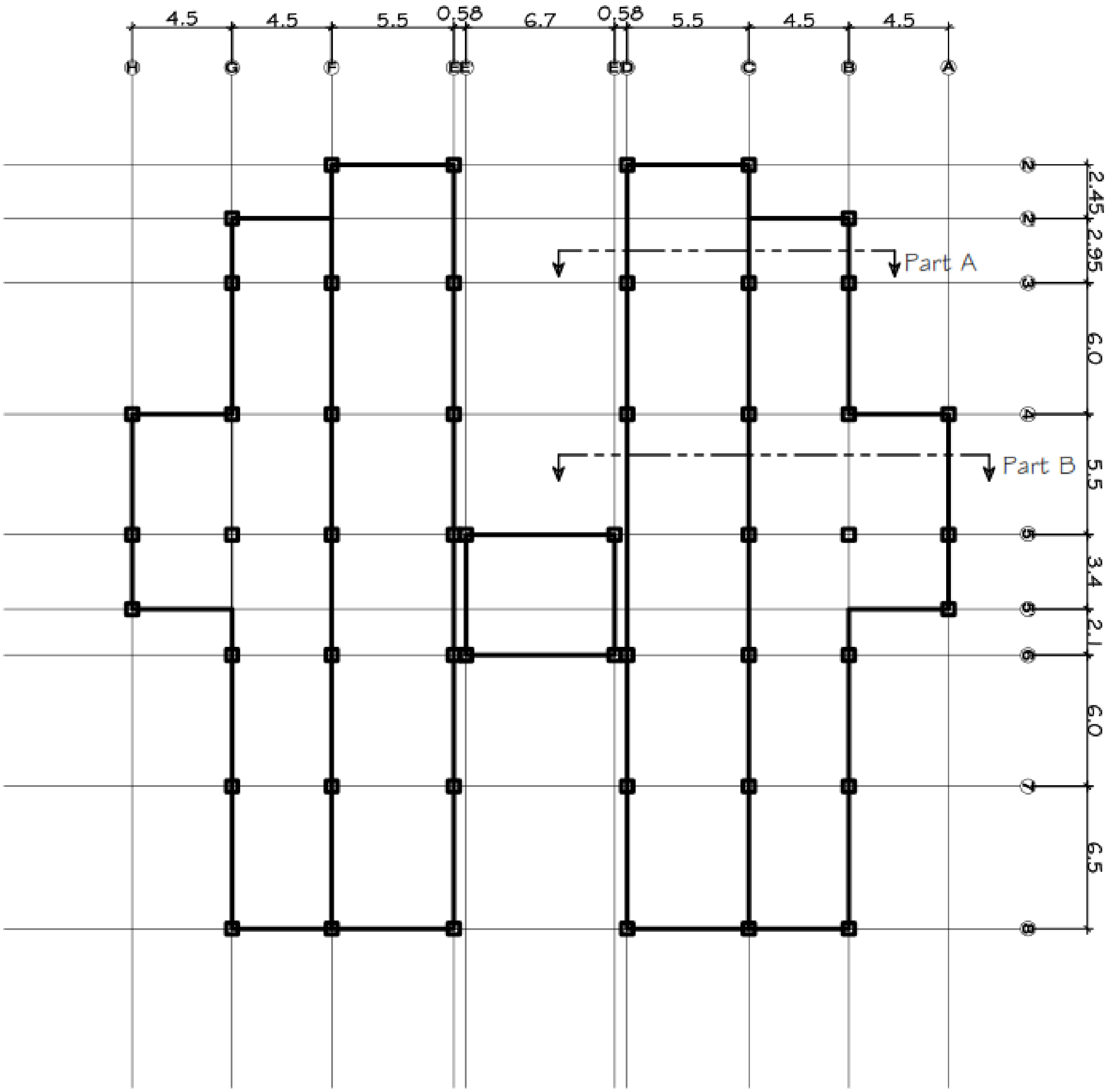
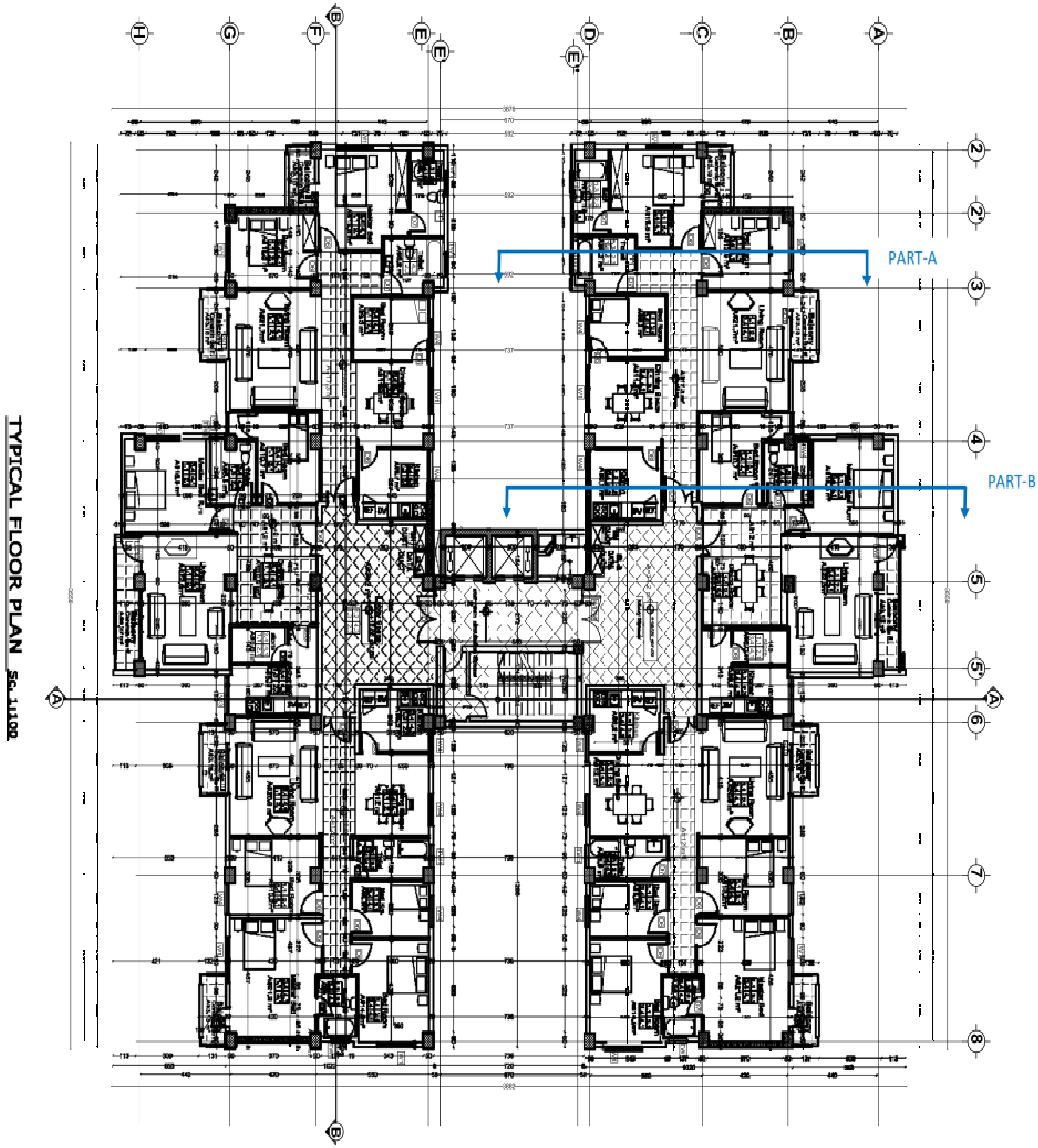


Figure 3.1: Typical floor plan of the designed slab


TYPICAL FLOOR PLAN Sc. 1:100
PART-A
PART-B

3.2. Modeling

In the production stage, while manufacturing, the plate is fully supported on its bottom face but in the process of lifting and transporting, the plate is to be handled only at four points. Hence, for the production stage, the lifting phase is considered for design. Since the main reinforcement and the latticed shear reinforcement are welded at every joint, the latticed reinforcement system in the initial stage is modeled as a 3-D frame system using SAP 2000V14.

In the construction stage, the plate is temporarily supported at a maximum of 2m of spacing. In this stage the plate is modeled on ETABS 9.7.4 as a 0.06m thick concrete plank being pin supported at every temporary support location.

In the final stage, the full section of the plate is considered (0.2m) and is modeled on ETABS 9.7.4 as a continuous beam running in the shorter direction of the slab layout.

3.3. Design Constants and Loads

- **Material Constants**

Based on the procedures on EN 1992-1-1:2004 for concrete and Eurocode 3-1992 for steel, the design strength of the materials is calculated and summarized below. **(*Appendix B*)**

Property	Material		
	Concrete C-25	Concrete LC-25/28	Steel (S-300)
f_{cu}	25 MPa	28 MPa	-
f_{ck}	20 MPa	25 MPa	-
f_{cd}	11.33 MPa	14.17 MPa	-
f_{ctk}	1.55 MPa	1.52 MPa	-
f_{ctd}	1.03 MPa	1.01 MPa	-
E_{cm}	29 GPa	31 GPa	-
Expansion Coefficient	10×10^{-6} K^{-1}.	8×10^{-6} K^{-1}.	-
f_{yk}	-	-	300MPa
f_{yd}	-	-	260.87MPa

Table 3.1: Summary of design constants for different construction materials

- **Unit Weight**

Unit weight of construction materials according to EN 1991-1-1:2001,

Materials	Unit Weight
Normal Concrete	25 kN/m^3
Light Concrete	16.56 kN/m^3
Block	14 kN/m^3
Mortar	23 kN/m^3

Table 3.2: Unit weight of construction materials

- **Design Loads**

Summary of design loads for the different construction stages is presented in the following table and the detailed load calculation for precast lattice slab is presented in ***Appendix C.***

Type of concrete used	Loads acting on the slab	Stages of construction			
		Production stage	Construction stage		Final stage
			Erection stage	Concrete pouring stage	
Normal weight concrete	Ultimate Load (kN/m^2)	2.02	3.52	8.25	12.65
	Service Load (kN/m^2)	-	-	-	9.15
Light weight concrete	Ultimate Load (kN/m^2)	1.34	2.84	5.97	10.37
	Service Load (kN/m^2)	-	-	-	7.46

Table 3.3: Summary of load history

The study of the analysis and design of precast lattice slab includes both ultimate limit state and serviceability limit state conditions.

3.4. Ultimate Limit State (ULS)

Ultimate limit state is concerned with failure by rupture, loss of stability, loss of equilibrium and failure caused by fatigue. In order to satisfy the design requirements of ULS, one needs to apply appropriate safety factors, consider the most critical combinations of loads and avoid brittle failure (ensure ductility). There are three basic assumptions at ULS [15] , these are:

a. Sections perpendicular to the axis of bending that are plane before bending remains plane after bending.
b. The strain in the reinforcement is equal to the strain in the concrete at the same level.
c. The stress in the concrete and reinforcement can be computed from the strains by using stress-strain curves for concrete and steel.

Ultimate limit state includes:

- Limit state for Flexure
- Limit state for Shear
- Limit state for punching
- Limit state for Torsion
- Limit state for Fatigue

Among these, Limit state for Flexure and Shear are the most common ones and are covered in this research.

3.4.1. Limit State for Flexure

Each of the plates used have Ly/Lx ratio greater than 2, as a result, they are categorized as one way rectangular slab. One way slabs may be simply supported or continuous over a number of supports. The bending moments, on which design is to be based, are calculated from elastic analysis in the same way as for beams.

The flexural design of one-way slab sections are treated in the same manner as for singly reinforced rectangular beam sections, considering the slab as strips of beams having a width of 1m (1.2m in this case). The reinforcement bar obtained is distributed uniformly with spacing between bars. The basic rule which needs to be satisfied in limit state of flexure is that the section capacity for flexure should be greater than the existing flexure in the section ($M_{rd} > M_{sd}$)

3.4.1.1. Production Stage

This phase represents the manufacturing, lifting, transportation and handling of precast lattice plates to the construction site. While the plates are being manufactured, they are fully supported on their bottom face therefore the critical phase to be considered in this stage should be the lifting phase. The lifting points support the self weight of the plate. The latticed shear reinforcement is modeled as a 3D frame structure being simply supported at the lifting points with the self weight of the plate being loaded at every joint between the shear reinforcements and the bottom chords.

In this stage, buckling strength of frame members should be checked so that loss of stability due to buckling of the lattice girder or any of its members (the top chord or the diagonals) would not occur.

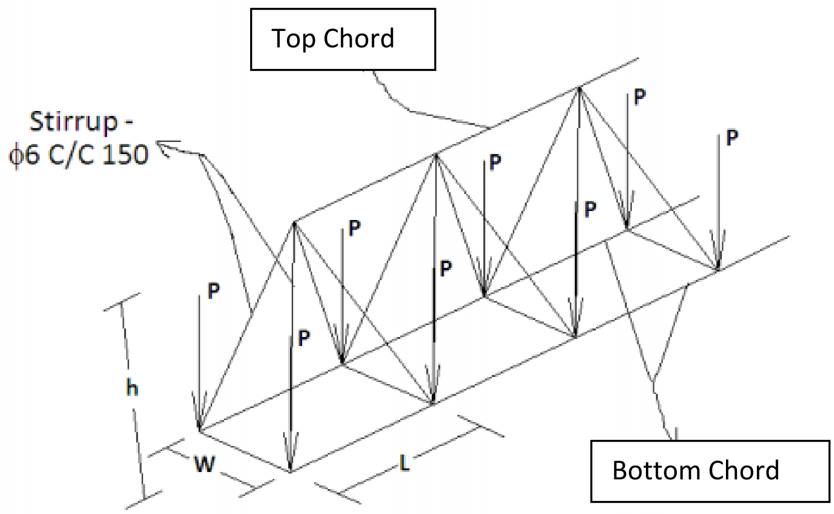


Figure 3.2 : Arrangement and loading of latticed shear reinforcement of plates in the production stage

3.4.1.2. Construction Stage

In this stage, temporary supports are to be used to help the construction process. The span and number of supports to be used is determined based on the design guidelines provided by FUNDIA 1992 [16]. The equations are valid for a maximum construction load of 1.5kN/m^2 . As provided on Annex J of EN 13747, a construction live load of 1kN/m^2 is used for this research. The guideline provides that the maximum allowable span (L_f) for a section with a diameter of 10mm top reinforcement is:

$$L_f \leq \sqrt{\frac{0.4+0.5\,H}{c.H}} \leq 5\left(\sqrt[3]{\frac{h^2}{c.H}}\right)$$

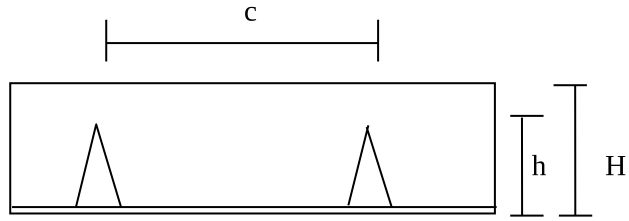


Figure 3.3: Dimensions of the precast plate for the placement of temporary support

Where

c= 0.6m

H = 0.2m, h = [0.2 – (0.05+0.015)] = 0.18m

$$L_f \leq \sqrt{\frac{0.4+0.5\,(0.2)}{0.2*0.6}} = 2.04\text{m} \leq 5\left(\sqrt[3]{\frac{(0.18)^2}{0.2*0.6}}\right) = 3.2m$$

Based on the above calculation, propping should be at maximum, every 2m.

For the analysis of slab in the construction stage, the one way rectangular plates are modeled as continuous beams having a depth of 60mm and a width of 1.2m and loaded with the governing load calculated in the previous chapter for the construction stage for both the normal and light weight concrete. Based on the arrangement of the temporary supports and the length of the plates, there would be two kinds of arrangements, each of them having three spans.

Arrangement-1 (L=5.5m, as shown in *figure 3.1*, Plate-1 runs between Axes 2 and 8)

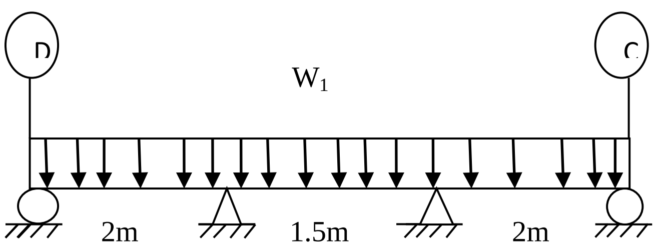


Figure 3.4: Arrangement of temporary support for Plate-1

Arrangement-2 (L=4.5m, as shown in *figure 3.1*, Plate-2 runs between Axes 2' and 8)

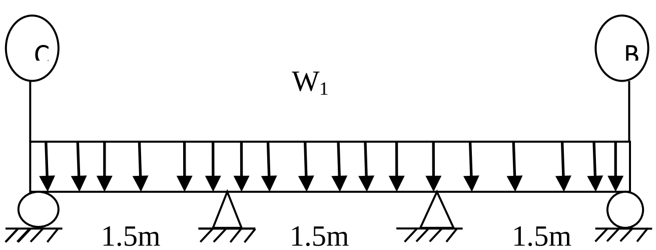


Figure 3.5: Arrangement of temporary support for Plate-2

Arrangement-3 (L=4.5m, as shown in *figure 3.1*, Plate-3 runs between Axes 4 and 5')

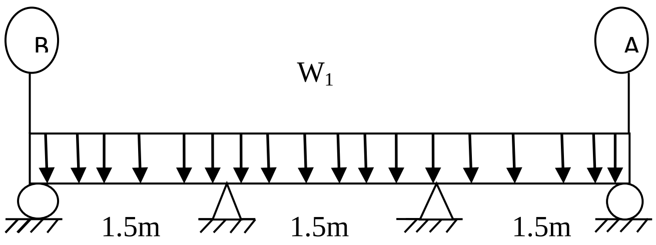


Figure 3.6: Arrangement of temporary support for Plate-3

3.4.1.3. Final Stage

For the analysis of slab in the final stage, the one way rectangular plates are modeled as continuous beams having a depth of 0.2m and a width of 1.2m and loaded with the governing load calculated in the previous chapter for the final stage for both the normal and light weight concrete. Based on their architectural arrangement (Figure 3.1), there are two sections: one is a continuous one way slab with three spans and running between Axes 2' & 4 and Axes 5' & 8 where as the other section has two spans and running between Axes 4 & 5'.

Part-A

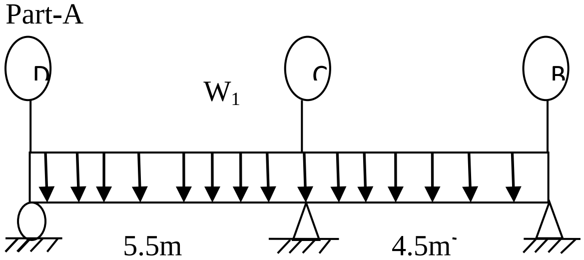


Figure 3.7: Load diagram of Part-A for the final stage

Part-B

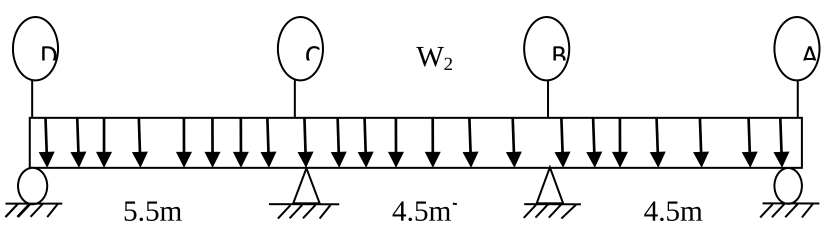
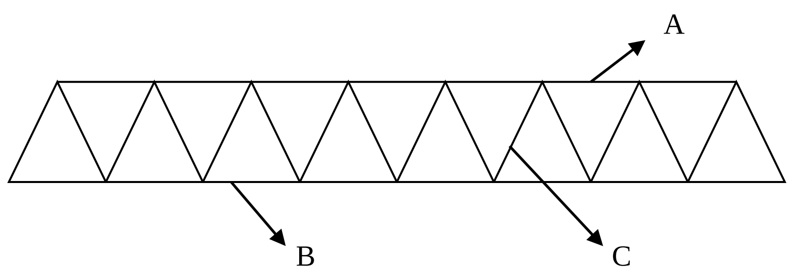


Figure 3.8: Load diagram of Part-B for the final stage

In this stage, in order to estimate the maximum moment which is caused by the variation of live load, pattern loading is used by applying the live load alternatively on the spans and finally moment envelope is developed for both sections. Therefore each of the spans (plates) is designed for maximum moment.

3.4.2. Limit State for Shear

In the design of reinforced concrete members flexure is usually considered first, (i.e. sections are proportioned and areas of longitudinal reinforcement determined for the moment M), because flexural failure is ductile. The members are then designed for shear. Because shear failure is frequently sudden and brittle, the design needs to ensure that shear strength equals or exceeds the flexural strength at all points in the section. In members subjected to an increasing shear stress, formation of diagonal cracks is followed by widening of cracks and brittle compression failure. This type of failure can be suppressed and development of full flexural capacity can be ensured by using shear reinforcement. Inclined stirrups, bent up longitudinal bars or vertical stirrups can be used. In this research the lattice reinforcements of the plates, which are inclined, serve as shear reinforcement.



-

A- Top longitudinal reinforcement

B- Bottom longitudinal reinforcement

C- Inclined shear reinforcement

Figure 3.9: Longitudinal arrangement of lattice reinforcement

3.4.2.1. Construction Stage

As discussed in section 3.1.1.2, for the analysis of slab in the construction stage, the one way rectangular plates are modeled as continuous beams having a depth of 0.06m and a width of 1.2m. Once the slab is analyzed, maximum shear values are used for manual design calculations as per EN 1992-1-1:2004 which is summarized as follows.

$V_{Rd,c}$ - is the design shear resistance of the member without shear reinforcement.

$V_{Rd,s}$ - is the design value of the shear force which can be sustained by the yielding shear reinforcement.

$V_{Rd,max}$ - is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

- Members not requiring shear reinforcement

$V_{Rd,c} = [C_{Rd,c} K (100\rho_1 f_{ck})^{1/3} + K_1 \sigma_{cp}] b_w d$ (EN 1992-1-1:2004: **6.2a**)

$V_{Rd,c} \geq (V_{min} + K_1 \sigma_{cp}) b_w d$ (EN 1992-1-1:2004: **6.2b**)

Where,

f_{ck} - is in MPa

d – effective depth of section given by

$d = D - C_c - \emptyset_{sr} - \emptyset_r/2$

D – total depth of section

C_c – concrete cover, taken to be 15mm for the plates.

$\emptyset_{sr}$ - diameter of secondary reinforcement

$\emptyset_r$ - diameter of main reinforcement

$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with d in mm

$\rho_1 = \frac{A_{s1}}{b_w d} \leq 0.02$

A_{s1} – is the area of the tensile reinforcement

b_w – is the smallest width of the cross-section in the tensile area [mm]

$\sigma_{cp} = \frac{N_{Ed}}{A_c \times d} < 0.2$ fcd

N_{Ed} = is the axial force in the cross-section due to loading or prestressing [in N]

A_c - is the area of concrete cross section [mm^2]

$V_{Rd,c}$ – is in [N]

$C_{Rd,c} = \frac{0.18}{\gamma_c}$ (recommended)

$K_1 = 0.15$ (recommended)

$V_{min} = 0.035\ K^{3/2}\ fck^{1/2}$

- Members requiring shear reinforcement

For members with inclined shear reinforcement, the shear resistance is the smaller value of:

$V_{Rd,s} = \frac{A_{sw}}{s}\ z\ f_{ywd}\ (\cot\theta + \cot\alpha)\sin\alpha$ (EN 1992-1-1:2004: **6.13**)

And

$V_{RD,\,max} = \alpha_{cw}\, b_w\, z\, \nu_1 f_{cd}\ (\frac{\cot\theta + \cot\alpha}{1+cot^2\theta})$ (EN 1992-1-1:2004: **6.14**)

Where,

α - is the angle between shear reinforcement and the beam axis perpendicular to the shear force.

θ – is the angle between the concrete compression strut and the beam axis perpendicular to the shear force. It is recommended to limit the value of the angle ($1 \le \cot\theta \le 2.5$)

fcd - is the design value of the concrete compression force in the direction of the longitudinal member axis.

bw - is the minimum width between tension and compression chords

z - is the inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration. In the shear analysis of reinforced concrete without axial force, the approximate value z = 0.9d may normally be used.

A_{sw} - is the cross-sectional area of the shear reinforcement

s - is the spacing of the stirrups

f_{ywd} - is the design yield strength of the shear reinforcement

ν_1 - is a strength reduction factor for concrete cracked in shear ($\nu_1 = \nu$ recommended)

$$\nu_1 = \nu = 0.6\ [1 - \frac{f_{ck}}{250}]$$

α_{cw} - is a coefficient taking account of the state of the stress in the compression chord. It is recommended to take the value of α_{cw} as 1 for non prestressed structures.

3.4.2.2. Final Stage

For the analysis of slab in the final stage, the same procedure as the construction stage is to be followed except that the one way rectangular plates are modeled as continuous beams having a depth of 0.2m and a width of 1.2m. Once the slab is analyzed, maximum shear values are used for manual design calculations as per EN 1992-1-1:2004 as explained in the previous section. The additional consideration in this stage is the shear stress at the interface between concrete cast at different times. This is to account for the shear stress caused when a fresh concrete topping is casted on the prefabricated concrete plank.

- Shear at the interface between concrete cast at different times

In addition to the previous shear requirements, the shear stress at the interface between concrete cast at different times should also satisfy the following:

$V_{Edi} \leq V_{Rdi}$ (EN 1992-1-1:2004: **6.23**)

V_{Edi} - is the design value of the shear stress in the interface and is given by:

$V_{Edi} = \frac{\beta V_{Ed}}{z\, b_i}$ (EN 1992-1-1:2004: **6.24**)

Where,

β - is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

V_{Ed} - is the transverse shear force

z - is the lever arm of composite section

b_i - is the width of the interface

V_{Rdi} - is the design shear resistance at the interface and is given by:

$V_{Rdi} = C\, fctd + \mu\, \sigma n + \rho\, f_{yd}\, (\mu \sin \alpha + \cos \alpha) \leq 0.5\, \nu\, f_{cd}$ (EN 1992-1-1:2004: **6.25**)

Where,

c and μ - are factors which depend on the roughness of the interface

f_{ctd} - is the design value of the tensile force in the longitudinal reinforcement

σ_n - stress per unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force,

$$\rho = \frac{A_s}{A_i}$$

A_s - is the area of reinforcement crossing the interface, including ordinary shear reinforcement (if any), with adequate anchorage at both sides of the interface.

A_i - is the area of the joint

α - is defined previously, and should be limited by $45° \leq \alpha \leq 90°$

ν - is defined in the previous section.

3.5. Serviceability Limit State (SLS)

It is stated in Section 7.4 of EN 1992-1-1:2004 that the deformation of a member or structure shall not be such that it adversely affects its proper functioning or appearance. Appropriate limiting values of deflection taking into account the nature of the structure, of the finishes, partitions and fixings and upon the function of the structure should be established.

According to EN 1992-1-1:2004, the limit state of deformation may be checked either:

a. by limiting the span/depth ratio, or
b. by comparing a calculated deflection, with a limit value.

It is only reasonable to *check serviceability limit state only for the final stage*. Since the whole depth of the slab is under consideration in the last stage, the first option (limiting the span/depth ratio) is to be used.

3.5.1. Final Stage

Provided that reinforced concrete beams or slabs in buildings are dimensioned so that they comply with the limits of span to depth ratio given below, their deflections may be considered as not exceeding the deflection control limits. The limiting span/depth ratio can be estimated using the expressions below and multiplying this by correction factors to accommodate for the type of reinforcement used and other variables. No allowance has been made for any pre-camber in the derivation of these expressions.

$\frac{l}{d} = K\ [11 + 1.5\sqrt{f_{ck}}\ \frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\ (\frac{\rho_o}{\rho} - 1)^{3/2}]$ if $\rho \leq \rho_o$ (EN 1992-1-1:2004: **7.16a**)

$\frac{l}{d} = K\ [11 + 1.5\sqrt{f_{ck}}\ \frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\ \sqrt{\frac{\rho'}{\rho_o}}\ ]$ if $\rho > \rho_o$ (EN 1992-1-1:2004: **7.16b**)

Where,

l/d - is the limit span/depth

K - is the factor to take into account the different structural systems

ρ_o - is the reference reinforcement ratio = $\sqrt{f_{ck}}\ 10^{-3}$

ρ - is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers), it is given by $\frac{A_{s1}}{bd}$

ρ' - is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers), it is given by $\frac{A_{s2}}{bd}$

f_{ck} - is in MPa units

These expressions have been derived on the assumption that the steel stress, under the appropriate design load at SLS at a cracked section at the mid-span of a beam or slab or at the support of a cantilever, is 310MPa, (corresponding roughly to fyk = 500MPa).Where other stress levels are used, the values obtained using the expressions should be multiplied by $310/\sigma_s$. It will normally be conservative to assume that:

$$\frac{310}{\sigma_s} = \frac{500}{\frac{f_{yk} \times A_{s,req}}{A_{s,prov}}}$$ (EN 1992-1-1:2004: **7.17**)

Where,

σ_s - is the tensile steel stress at mid-span (at support for cantilevers) under the design load at SLS

$A_{s,prov}$ - is the area of steel provided at this section

$A_{s,req}$- is the area of steel required at this section for ultimate limit state

3.6. Design Flow Chart

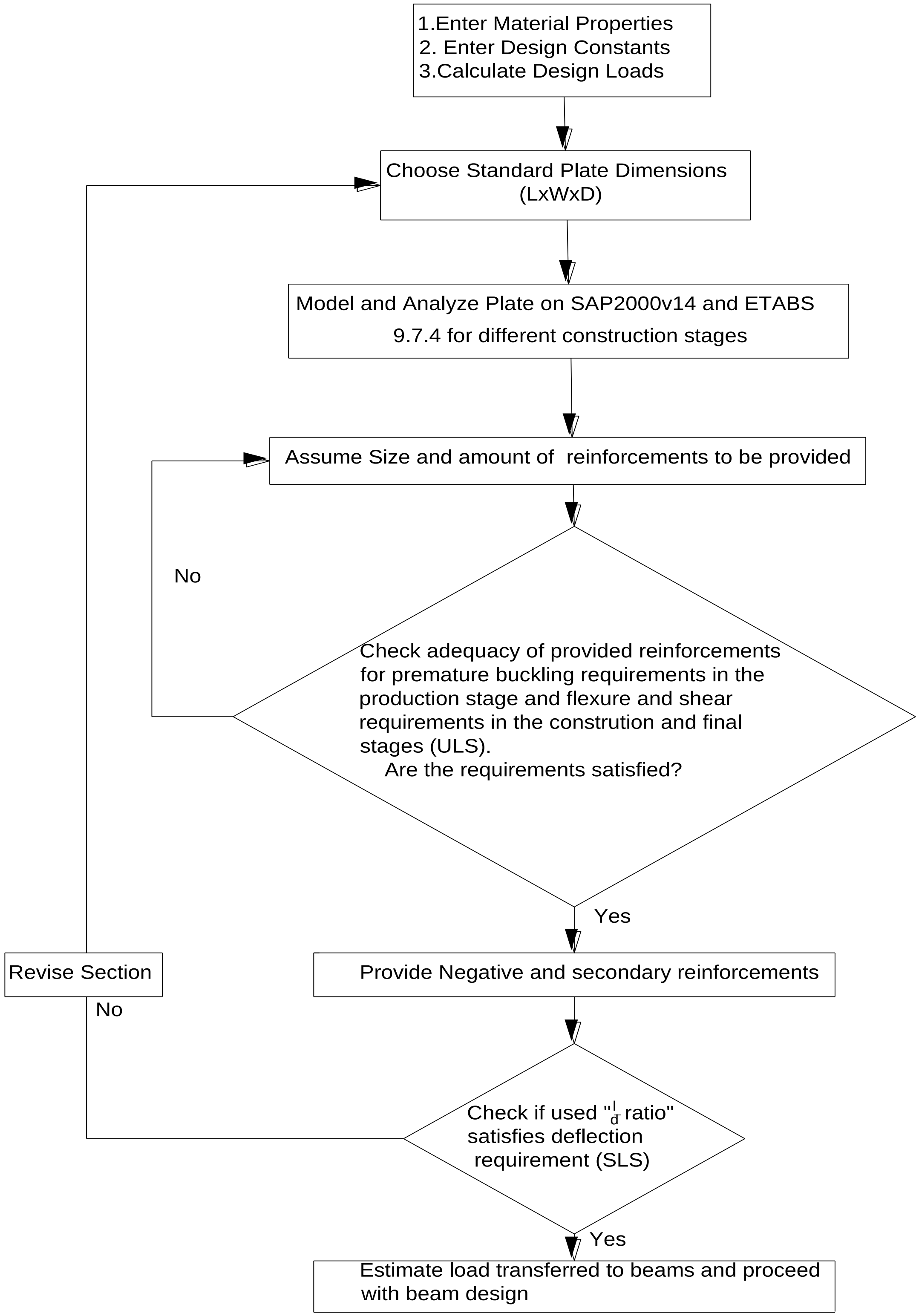


Figure 3.10: Flow chart for plate design

4. DESIGN EXAMPLE

In this section, an example showing the design of composite concrete slabs made of precast lattice plates is presented. The analysis and design of a whole slab-beam system is performed for both normal weight and light weight concrete of a typical slab in G+9 residential building (low cost building) but since the design procedure is similar for all plates, the example covers only the analysis and design of a single plate (Plate-1) through the production, construction and final stage. A detailed design calculation for Plate-1 is presented in ***Appendix D***.

4.1. Design Inputs

4.1.1. Normal Weight Precast Concrete Slab

a) Material Constants

The normal weight concrete is characterized by a concrete grade of C-25/30 with a unit weight of 25kN/m^3 and density of 2500kg/m^3.

f_{cu} = 25MPa	f_{ctk} = 1.55MPa
f_{ck} = 20MPa	f_{ctd} = 1.03MPa
f_{cd} = 11.33MPa	E = 29GPa

b) Construction stage loading on ETABS

The load for the construction stage from 8.25kN/m^2, should be multiplied by the width of the plate (1.2m) to change the loading from areal load to line load.

Load combination on ETABS

Pd = (1.35DL + 1.5LL) × (1.2m) = 1.62DL + 1.8LL

Where, DL = 5 $\frac{kN}{m^2}$ and LL = 1 $\frac{kN}{m^2}$ ***(Appendix C)***

c) Final stage loading on ETABS

The load for the final stage from *Table 3.3 of section 3.3*, which is 12.65kN/m^2, should be multiplied by the width of the plate (1.2m) to change the loading from areal load to line load.

Load combination on ETABS

Pd = (1.35DL + 1.5LL) × (1.2m) = 1.62DL + 1.8LL

Where, DL = 7.15 $\frac{kN}{m^2}$ and LL = 2 $\frac{kN}{m^2}$ ***(Appendix C)***

4.1.2. Light Weight Precast Concrete Slab

a) Material Constants

The light weight concrete is characterized by a concrete grade of LC-25/28 with a unit weight of 16.56KN/m^3 and density of 1656Kg/m^3.

f_{lcu} = 28MPa f_{lctk} = 1.52MPa

f_{lck} = 25MPa f_{lctd} = 1.01MPa

f_{lcd} = 14.17MPa   E = 17.7GPa

b) Construction stage loading ETABS

The load for the construction stage from *Table 3.3 of section 3.3*, which is 5.97kN/m^2, should be multiplied by the width of the plate (1.2m) to change the loading from areal load to line load.

Load combination on ETABS

Pd = (1.35DL + 1.5LL) × (1.2m) = 1.62DL + 1.8LL

Where, DL = 3.31 $\frac{kN}{m^2}$ and LL = 1 $\frac{kN}{m^2}$ ***(Appendix C)***

c) Final stage loading on ETABS

The load for the final stage from *Table 3.3 of section 3.3*, which is 10.37kN/m^2, should be multiplied by the width of the plate (1.2m) to change the loading from areal load to line load.

Load combination on ETABS

Pd = (1.35DL + 1.5LL) × (1.2m) = 1.62DL + 1.8LL

Where, DL = 5.46 $\frac{kN}{m^2}$ and LL = 2 $\frac{kN}{m^2}$ ***(Appendix C)***

4.1.3. Plate Sizes

The different sizes of Plate used are:

- Plate – 1: L=5.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 170mm, D = 200mm
- Plate – 2: L=4.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 170mm, D = 200mm
- Plate – 3: L=4.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 171mm, D = 200mm

4.2. Analysis and Design Parameters for Different Stages of Construction

4.2.1. Ultimate Limit State

a) Check Premature Buckling in the Production Stage/ Lifting

The ultimate limit state requirement of the Production stage is studied by the structural software SAP2000V14 using design preference Eurocode-3, 1993. A sample hand calculation to check the premature buckling of members is presented in ***Appendix D.*** The frame is loaded with a joint load of 0.09kN and 0.06kN in case of normal weight concrete and light weight concrete respectively. The frame has two hundred fifty eight members (stirrups, top and bottom chords) with three lifting points (two at exterior points and another at mid point) acting as a simple support for the 3D model as shown in the figure below.

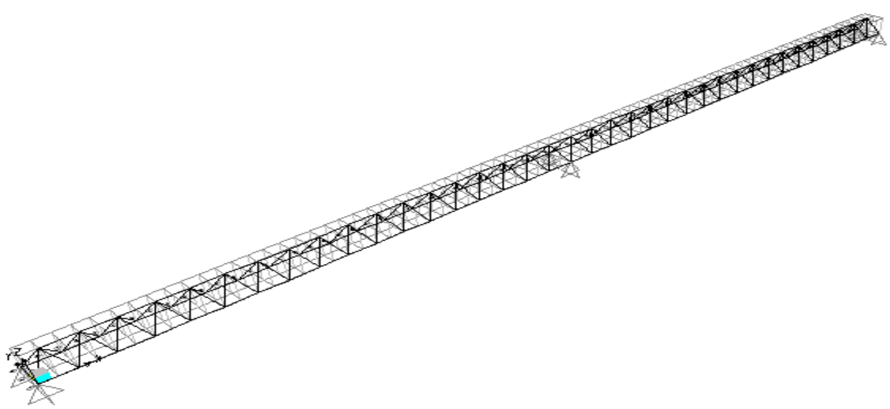


Figure 4.1: 3D modeling of the latticed shear reinforcements of Plate-1 on SAP2000V14

b) Summary of Other Design Parameters for Plate-1

I. Normal Weight Concrete

(use 2Ø10 Top and 11 Ø 10 c/c 110 Bottom Chords with Ø 6 c/c 150 Shear Reinforcement)

Stage of construction	Design Parameters	Action on Plate-1	Capacity of Plate-1
Construction Stage	Flexure	3.16 kNm	6.15 kNm
	Shear	12.12 kN	39.00 kN
Final Stage	Flexure	35.31 kNm	37.40 kNm
	Shear	47.29 kN	153.05 kN

Table 4.1: ULS design parameters in the construction and final stage (Normal Weight)

II. Light Weight Concrete

(use 2Ø10 Top and 10 Ø 10 c/c 110 Bottom Chords with Ø 6 c/c 150 Shear Reinforcement)

Stage of construction	Design Parameters	Action on Plate-1	Capacity of Plate-1
Construction Stage	Flexure	2.26 kNm	6.15 kNm
	Shear	8.74 kN	41.28 kN
Final Stage	Flexure	28.59 kNm	34.20 kNm
	Shear	38.10 kN	157.63 kN

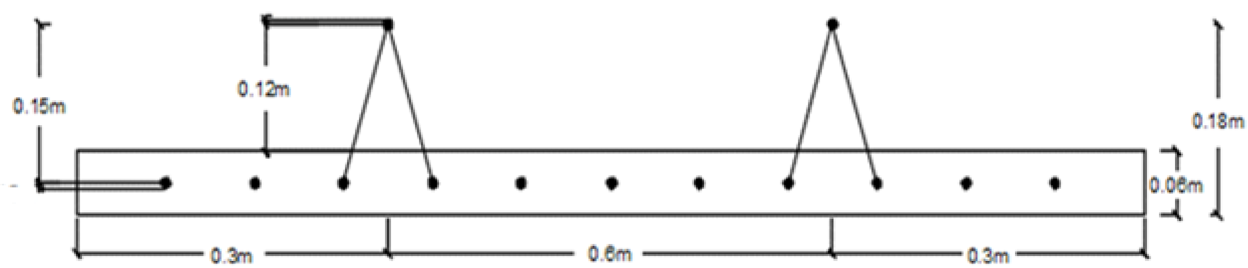
Table 4.2: ULS design parameters in the construction and final stage (Light Weight)

4.2.2. Serviceability Limit State

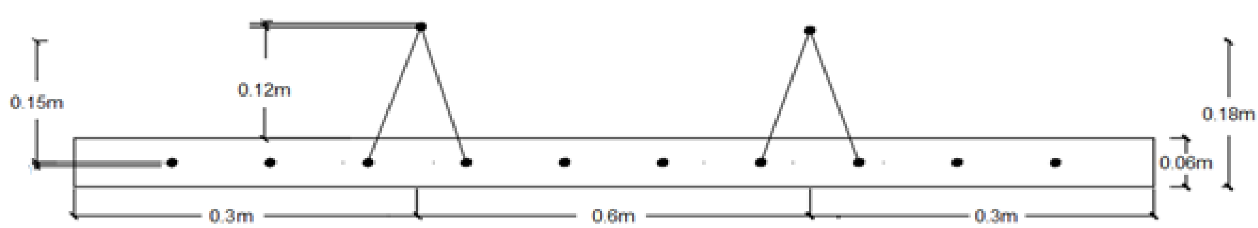
For both light weight and normal weight concrete, the serviceability limit state of deflection is checked for the final stage. The ratio of provided length of the plate (l) to the effective depth (d) is to be compared with the limiting span/depth ratio provided in Eurocode and multiplying it by a correction factor to allow for the type of reinforcement used and other variables.

4.3. Detailing of Plate-1 at Different Stages of Construction

4.3.1. Construction Stage



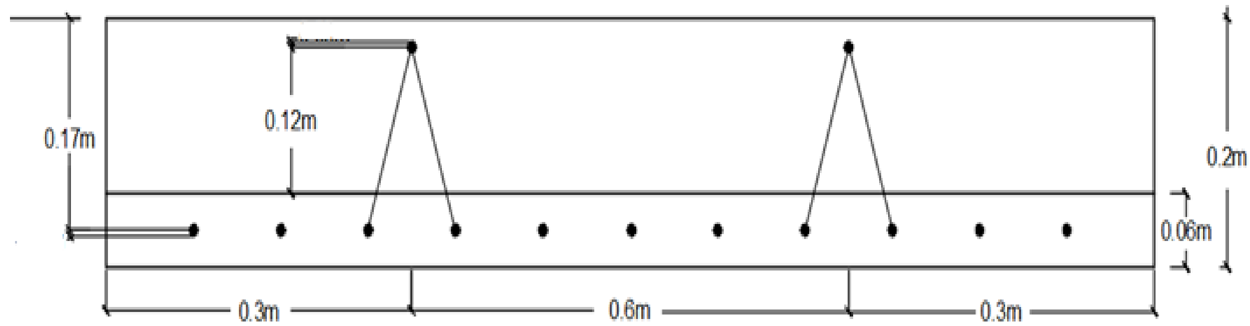
a. Normal weight concrete



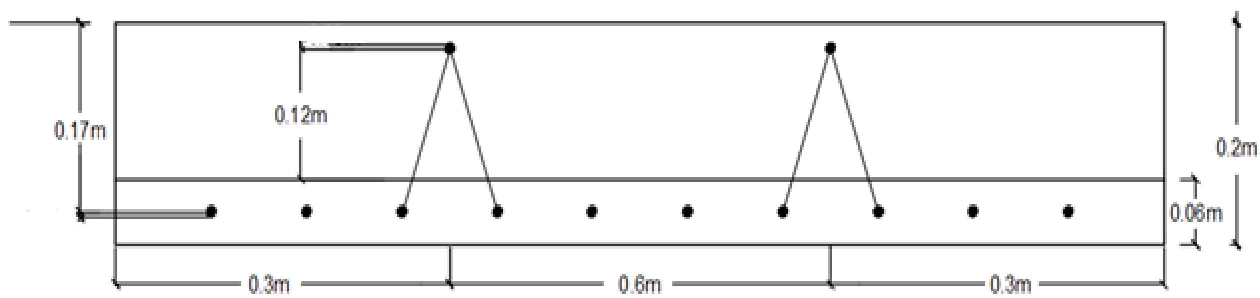
b. Light weight Concrete

Figure 4.2: Section View of Plate-1 at Construction Stage

4.3.2. Final Stage



a. Normal weight concrete



b. Light weight concrete

Figure 4.3: Section view of Plate-1 at Final Stage

4.4. Summary of Provided Reinforcements for All Plate Sizes

- The other two plates, Plate – 2 and Plate – 3, are analyzed and designed in the same way as Plate – 1 and the reinforcement provided for each plate is summarized in the next table.

Plate type	Plate – 1	Plate – 2	Plate – 3
Size of plate (L×W)	5.5m×1.2m	4.5m×1.2m	4.5m×1.2m
Total depth of slab	0.2m	0.2m	0.2m
Effective depth (d) of slab	0.17m	0.17m	0.171m
Section Capacity of plate (Mrd)	37.4KNm	26.7KNm	12.5KNm
Design Moment for plate (Msd)	35.31KNm	23.87KNm	10.25KNm
Main reinforcement of plate	11 Ø 10 c/c 110	8 Ø 10 c/c 150	8 Ø 8 c/c 150
Secondary reinforcement of Plate	13 Ø 6 c/c 450	10 Ø 6 c/c 450	10 Ø 6 c/c 450
Shear reinforcement of plate	Ø 6 c/c 150	Ø 6 c/c 150	Ø 6 c/c 150
Mesh used for crack control (temperature and shrinkage)	Ø 6 c/c 300	Ø 6 c/c 300	Ø 6 c/c 300

Table 4.3: Summary of provided reinforcements for normal weight precast concrete lattice plates

Plate Type	Plate – 1	Plate - 2	Plate – 3
Size of plate (L×W)	5.5m×1.2m	4.5m×1.2m	4.5m×1.2m
Total depth of slab	0.2m	0.2m	0.2m
Effective depth (d) of slab	0.17m	0.17m	0.171m
Section Capacity of plate (Mrd)	34.2KNm	24.1KNm	17.5KNm
Design moment for plate (Msd)	28.59KNm	23.13KNm	9.05KNm
Main reinforcement of plate	10 Ø 10 c/c 120	7 Ø 10 c/c 170	8 Ø 8 c/c 150
Secondary reinforcement of plate	13 Ø 6 c/c 450	10 Ø 6 c/c 450	10 Ø 6 c/c 450
Shear reinforcement of plate	Ø 6 c/c 150	Ø 6 c/c 150	Ø 6 c/c 150
Mesh used for crack control (temperature and shrinkage)	Ø 6 c/c 300	Ø 6 c/c 300	Ø 6 c/c 300

Table 4.4: Summary of provided reinforcements for light weight precast concrete lattice plates

5. COMPARISON BETWEEN THE CONVENTIONAL AND THE NEWLY ADVISED METHOD

5.1. Introduction

According to the report by GTZ, the low cost housing projects have low quality and relatively high price which in turn made this part of the construction sector incompetent. In this research, a new form of slab known as precast concrete lattice slab is introduced. This new method of construction is believed to give solution for the low cost buildings in terms of cost and quality; and this chapter of the research is dedicated to comparing the previous (ribbed slab) method and the new method of construction.

5.2. Cost Comparison

The three types of slab: Ribbed Slab, Precast Concrete Lattice Slab (light weight concrete) and Precast Concrete Lattice Slab (normal weight concrete) were used to design the typical slab (400m^2) of a G+9 low cost building. To identify which of these slabs result in the lowest cost, cost comparison was made for the slab-beam systems representing the three methods and is presented as follows.

5.2.1. Unit Price of Materials

The unit prices for steel, light weight concrete and normal weight concrete are gathered from the website "www.2merkato.com". The unit price for steel includes the material, transportation and labor cost for a kilogram of reinforcement bar where as the unit price for concrete includes the material, labor and vibrator cost for a m^3 of concrete.

Materials	Avg. unit price
Form work	235.00 birr per m^2
Normal weight concrete	2500.00 birr per m^3
Light weight concrete	1870.00 birr per m^3
Reinforcement bar	40.00 birr per kg

Table 5.1: Unit price of different construction materials

- **Cost of Crane**

Since precast plates need a crane to be lifted from the ground and put in to permanent position on the building, the cost of crane has to be included in the cost estimation of precast light and normal weight slabs.

- On average, it has been learned that a tower crane is rented for 2000 birr per hour in the current construction market of Addis Ababa.
- The focus of this thesis is the low cost housing projects and more specifically the 40/60 projects which are known to be high rise buildings ranging from "G + 9" to "2B + 12". As a result, it is known that the crane is also used for lifting other products in addition to precast plates. Based on this fact, it is assumed for the crane to engage in the positioning of plates among other lifting tasks 33% of the time.

crane cost per hour = 0.33 × 2000 birr = 660 birr per hour

(for lifting plates)

If on average the crane lifts 6 plates per hour (2 plates per a single trip),

Crane cost per plate = $\frac{660\ birr/hour}{6\ plates/hour}$ = 110 birr/plate

In order to estimate the cost of crane per 1m^3 of concrete, first one needs to determine the amount of concrete per a single plate.

amount of concrete per plate = 5.5m × 1.2m × 0.2m = 1.32 m^3 per plate

crane cost per m^3 of concrete = $\frac{110\ birr/plate}{1.32\ m^3/plate}$ = 83 birr per plate

Normal Weight Concrete = 2500 + 83 = 2583 birr per m^3

Light Weight Concrete = 1840 + 83 = 1923 birr per m^3

5.2.2. Summary of Quantity of Materials

Materials	Type of Slab		
	Ribbed Slab	Normal Weight Lattice Slab	Light Weight Lattice Slab
Slab Concrete	400 m^2	79.08 m^3	79.08 m^3
Beam Concrete	18.69 m^3	18.69 m^3	17.81 m^3
Rebar	9078.16 kg	7090.45 kg	6473.01 kg
Formwork	585.12 m^2	-	-

Table 5.2: Summary of quantity of materials

Finally, the cost comparison between the three types of slab construction methods is summarized in the following table.

Type of Slab	Cost			Reduction/ Increment of Cost in comparison with ribbed slab (%)
	Cost of Concrete (Slab and Beam)	Cost of Rebar (Slab and Beam)	Total Cost (Slab and Beam)	
Ribbed Slab	454,228.20 birr	363,126.50 birr	817,354.70 birr	-
Precast lattice slab (normal weight concrete)	250,988.64 birr	283,618.20 birr	534,606.85 birr	35% reduction of cost
Precast lattice slab (light weight concrete)	184,841.24 birr	258,920.70 birr	443,761.93 birr	45% reduction of cost

Table 5.3: Cost comparison between ribbed slab and precast lattice slab

5.3. Weight Comparison

In the same way as the cost comparison, weight comparison was also carried out for the slabs. The result of the comparison is summarized and presented in the table below.

Type of Slab	Weight of the Slab-Beam system in KN	Reduction/ Increment of Weight in comparison with ribbed slab (%)
Ribbed Slab	2302.50	-
Precast lattice slab (normal weight concrete)	2544.71	10% increment of weight
Precast lattice slab (light weight concrete)	1674.49	30% reduction of weight

Table 5.4: Weight comparison between ribbed slab and precast lattice slab

5.4. Structural Interpretation

The comparison between ribbed slab and normal weight precast concrete lattice slab shows that there is a 10% increment of weight and a 35% reduction of cost in the lattice slab as compared to the ribbed slab. This comparison shows that the concrete filled in place of the blocks of ribbed slab caused the additional weight in the lattice slab but at the same time, along with the compression chords, it enhances the section capacity of the structural slab leading to reduced amount of reinforcement in the slab and in the beams which finally resulted in 35% cost reduction.

The other comparison between ribbed slab and light weight precast concrete lattice slab shows that there is a 30% reduction of weight and a 45% reduction of cost in the lattice slab as compared to the ribbed slab. Using light weight concrete, which has almost the same unit weight as the blocks used for ribbed slab and at the same time has a comparable strength class as C-25/30, has resulted in a satisfactory reduction of cost and weight per a single slab. Therefore, using light weight concrete precast lattice slab has the following advantages:

- Reduced dead load in the slab
- Reduced transferred load in the beams, columns and foundations
- Reduced reinforcement and cross- sections of beams , columns and foundations
- Reduced base shear for the estimation of earthquake force.
- Reduced cost in building

6. MARKET ASSESSMENT

In our country Ethiopia, the company dedicated to fabricating precast building parts was first established in 1984EC. The company produces prestressed structural building parts including beam, slab, column, foundation and shear wall which are of high quality. These building parts are produced in limited sizes as they are casted in standardized moulds. The fact that ever since the company was established it has only been involved in 40 building projects shows that the demand for precast technology in the country is very low. In addition to this, the fabrication technology has not expanded to other companies and therefore Ethiopian Construction Works Corporation is the only supplier of precast building parts. This shows that the construction society is not interested in precast or composite (combination of cast insitu and precast) method of construction[5].

One finds a different story when referring to the low cost housing projects. The demand for these building types is growing by the day as they have a relatively lower price when compared to other buildings. Rather, the problem with the low cost buildings is that, recently their quality is degrading and their cost is becoming higher and as a result they are no more easily affordable[6].

In the previous chapters of the research, a solution which would increase the quality and at the same time minimize the cost of the buildings was developed by introducing precast concrete lattice slab. In order to expand the technology of precast lattice slab for other building projects other than the low cost housing projects, market assessment was carried out to study what caused the low demand for precast technology. In this chapter, the procedure followed and the results of the market assessment are presented. The outcome of the market assessment would help in identifying the gap and in providing solutions so that the newly adopted type of slap can penetrate the construction market.

6.1. Data Collection

The market assessment part of the research has a descriptive design and the type of survey used is a cross-sectional survey. In the data collection process, the data collection tools used are observation, interview, questionnaire and document analysis. These sources of data are generally categorized as primary and secondary sources of data. Under primary sources of data are observation, interview and questionnaire where as under secondary sources of data is document analysis. .

The sampling technique used to distribute questionnaires is availability sampling technique. The questionnaires were distributed among the offices of MH Engineering, Habtamu International Consulting Architects and Engineers, Bamacon Engineering, Ethiopian Building Construction Corporation and some free lancers. Among the people for whom the questionnaires were distributed, 40 people volunteered to fill in the questionnaires. These people are currently involved in the construction industry and have different job titles and different working experience.

The other data collection tool used is interview with the two key informants. The Product Quality Control and Managing Director of Ethiopian Construction Works Corporation; and the Contract Administration Section Head and Project Coordinator of MH Engineering were interviewed.

6.2. Data Presentation and Analysis

The data gained from the data collection tools (observation, questionnaire, interview and document analysis) are presented and analyzed as follows.

The choice between data presentation methods (Pie Chart and Bar Chart) merely depend on ease of visualization. A pie chart is used when the individual values of each slice is not important and where the data to be presented is easy to visualize when presented as a slice of a circle (25%, 50% , 75% and so on). A bar chart is used otherwise.

Q1. Educational background in Civil Engineering/ Construction related fields.

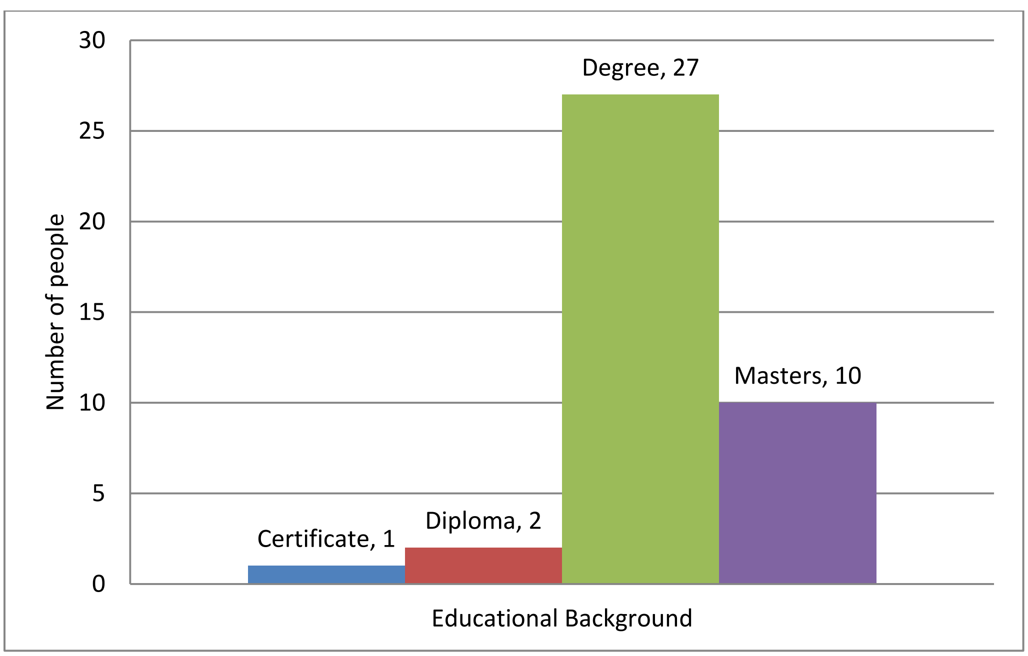


Figure 6.1: Educational background of participants

Q2. Job title

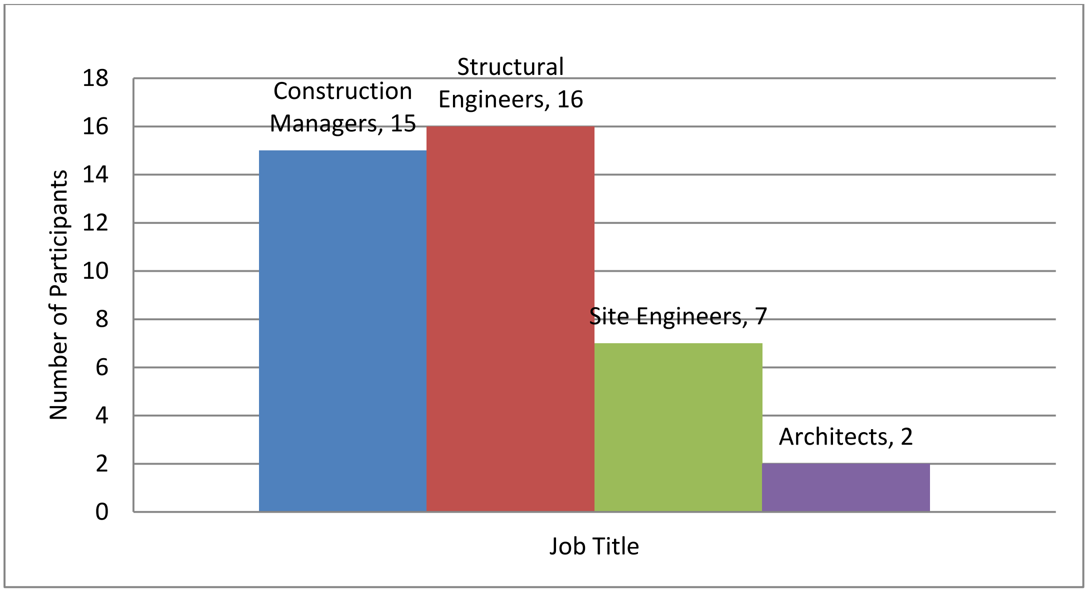


Figure 6.2: Job title of participants

Q3. Total work experience in the field of construction

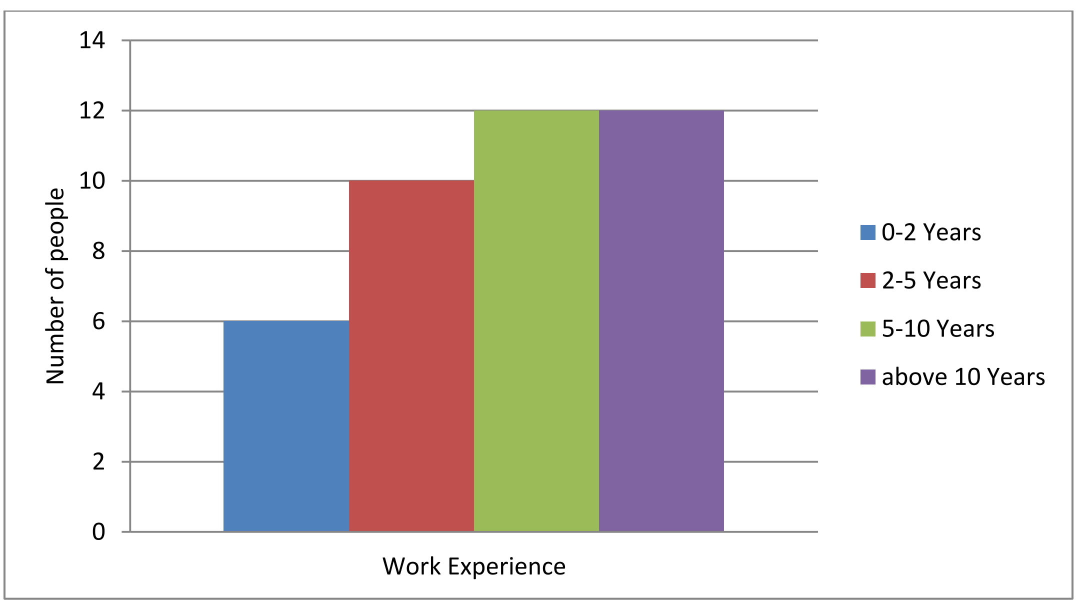


Figure 6.3: Working experience of participants

Q4. Have you ever worked on building projects other than the low cost housing projects which involved precast concrete building elements?

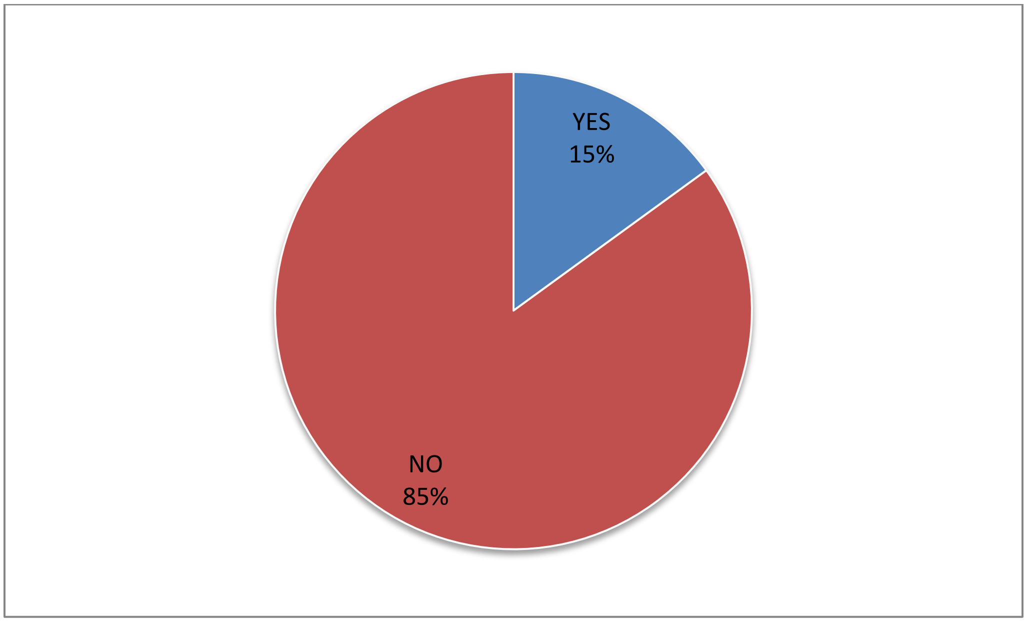


Figure 6.4: Work experience of participants on precast elements

- Among the total number of participants who filled in the questionnaire, 85% of them have never worked on projects involving precast concrete building elements (does not include the low cost housing projects). This shows that, for some reason, most of the buildings in the country are constructed by insitu method of construction.

  The data from interview also supports the previous statement. As the key informant (managing director of Ethiopian Construction Works Corporation) explained, the fact that the company is involved in only 50 projects including the ongoing ones ever since it was established shows that the demand for precast technology is very low in the construction market. It was also mentioned by the informant that there were even times that the company got involved in insitu constructions as a normal contractor to save the company from loss.

Q5. While working on building projects, which of the construction method do you prefer for the project under execution?

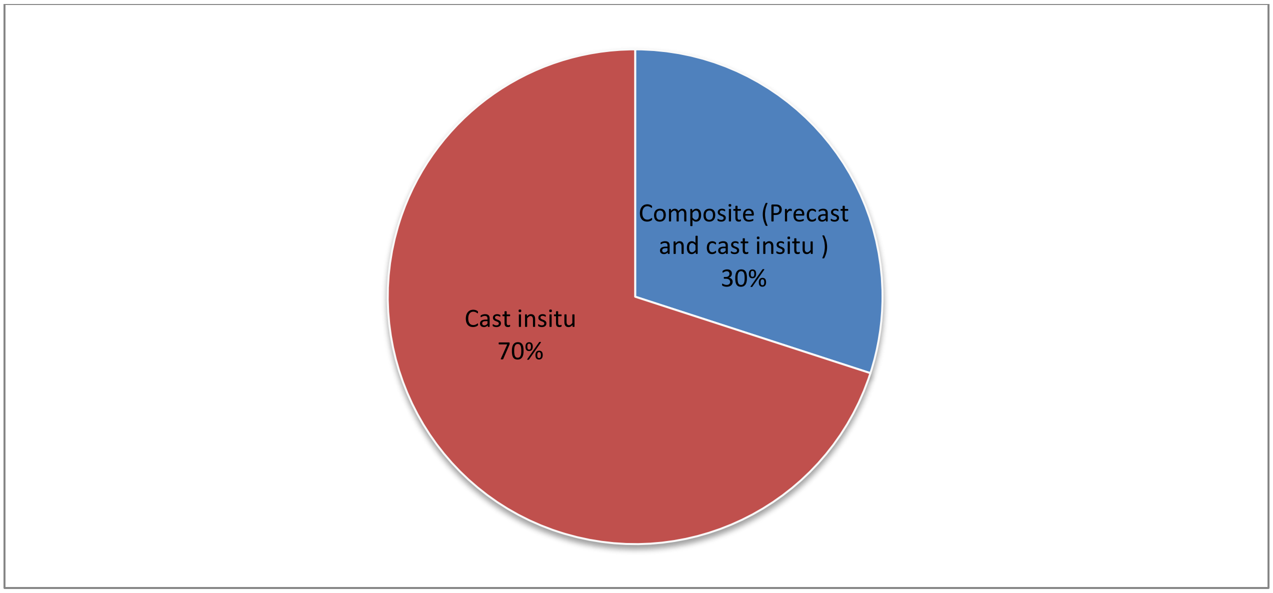


Figure 6.5: Preference on construction methods

- Supporting the idea in the previous question, most of the participants in this study prefer cast insitu method of construction.

Q6. If your answer to question No. 5 is "Cast insitu", why did you avoid using the composite method of construction? (Allowed to thick three)

Reasons	Number of people	% of people
I am not sure of its quality assurance	14/28	50%
It is a relatively time taking method	3/28	11%
I do not have enough knowledge and experience in the area	10/28	36%
I do not think it is cost efficient	12/28	43%
I do not think it allows design freedom	17/28	61%
I doubt its ease of construction	9/28	32%
I doubt its ease of manufacturing	10/28	36%
Other	9/28	32%

Table 6.1: Reasons of not using composite method of construction

- If the reasons are ranked in a descending order based on the results gained from the questionnaires, they would have the following order: the problem of design freedom, the problem of quality, doubt in cost effectiveness, lack of knowledge, doubt of competitiveness in manufacturing and constructing, doubt in time effectiveness and other unmentioned reasons.

- As mentioned previously, the precast elements are of limited sizes since they are casted on standardized moulds. As a result, the design freedom of both the architect and the structural engineer is limited. The data gained from the interview also supports this idea. The Ethiopian Construction Works Corporation produces first and for most, prestressed elements only. Besides that, they produce footings of 3 sizes, columns of 2 sizes, prestressed slab of a single size and shear wall of a single size. It has been mentioned that any building design which is to be built from precast elements has to be according to the available sizes.

- Another reason mentioned by the participants is the issue of quality. The fact that the quality of the low cost housings, which are constructed by composite method of construction, is deteriorating from time to time has made the experts participating in the construction industry lose confidence in precast technology.

- Doubt on the cost effectiveness of composite method of construction is another reason that the participants selected. Most of the participants involved in the data collection process have experience on precast technology in relation to the low cost projects. Therefore their knowledge of precast method of construction is influenced by the experience they have on the projects. And when considering the low cost projects, using composite method of construction (precast ribs and concrete topping) was believed to have reduced the cost of construction. But these days, on the construction site, the amount of wastage covers up to 30% of the construction cost. In addition to that, when one uses precast elements, additional costs of transportation and erection has to be considered. As a result, majority of the people avoid using precast elements.

- Another important issue is lack of knowledge on the area of precast concrete technology. The knowledge can be on the design, production, handling and erection of precast elements. The data from interview also explains that in contrast to cast insitu method of construction, working with precast method of construction requires skilled man power. It was learned that before 30 years, when the company of Ethiopian Construction Works Corporation was first established, 27 people took training on the design, production and erection of concrete precast elements by scholars from Yugoslavia. Ever since then, there has not been any facilitated mass training programs and currently among those 27 people who took the training, only 4 of them are working on precast technology. So this proves that our country lacks people who have enough knowledge on precast concrete technology.

- Coming to ease of construction and manufacturing, unlike cast insitu method of construction, a special care has to be given while designing joints between precast structural elements. And later when erecting on site, attention has to be given for structural connections. The data obtained from interview shows that the machines used to manufacture the precast elements were mounted before 30 years when the company was first established. Since the machineries are outdated, they can only produce structural members which can only go up to a storey of G+10.

Q7. Are you informed of other forms of precast concrete elements other than precast ribs which are used for low cost housing projects in Ethiopian construction market?

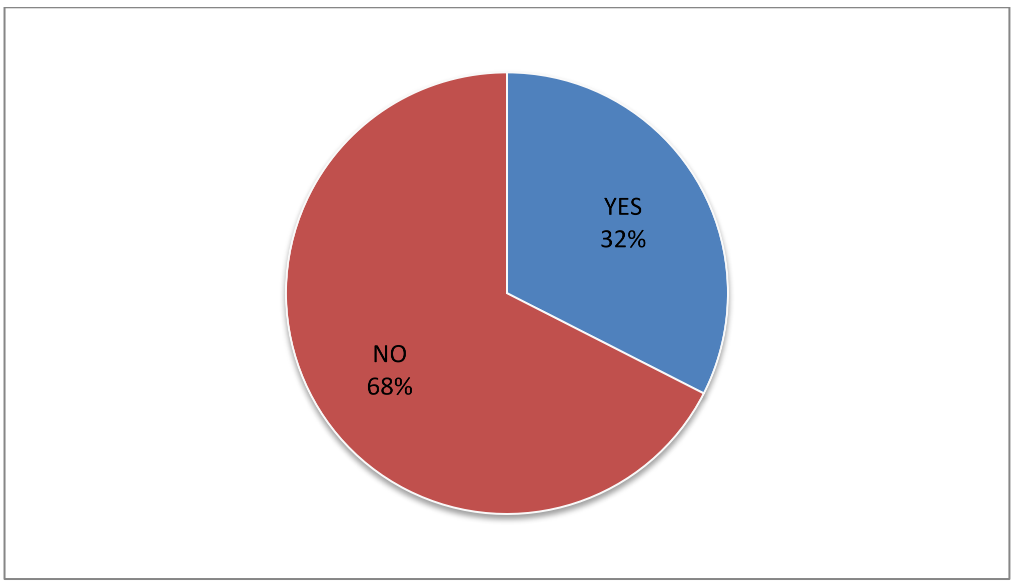


Figure 6.6: Information on precast building elements

- Majority of the participants are unaware of the products of Ethiopian Construction Works Corporation. It is simply assumed that the only form of precast concrete structural member available in the country is the precast rib used for condominium projects. This shows that the promotion work done to introduce the society to these products is unsatisfactory.

Q8. What is your opinion on the development of precast concrete technology in Ethiopia?

- All the participants who filled out the questionnaire pointed out that the development of precast technology in Ethiopia is very poor and have listed their opinion on what should be done to improve the current status of the country. The following are the most repeated ideas and are listed below according to their rank in descending order.
    1. Create awareness among the construction society.
    2. Work on quality improvement of precast building elements.
    3. Produce skilled manpower that is specialized in precast method of construction by facilitating training programs.
    4. Work on promotion of products.
    5. Updating of policies by governmental bodies.
    6. Present a better means of transportation and erection of products
    7. Adopt and introduce new forms of precast technologies by continually doing researches.
    8. Improve the production technology by importing new machineries from abroad.

Q9. Willingness to take trainings on precast concrete technology.

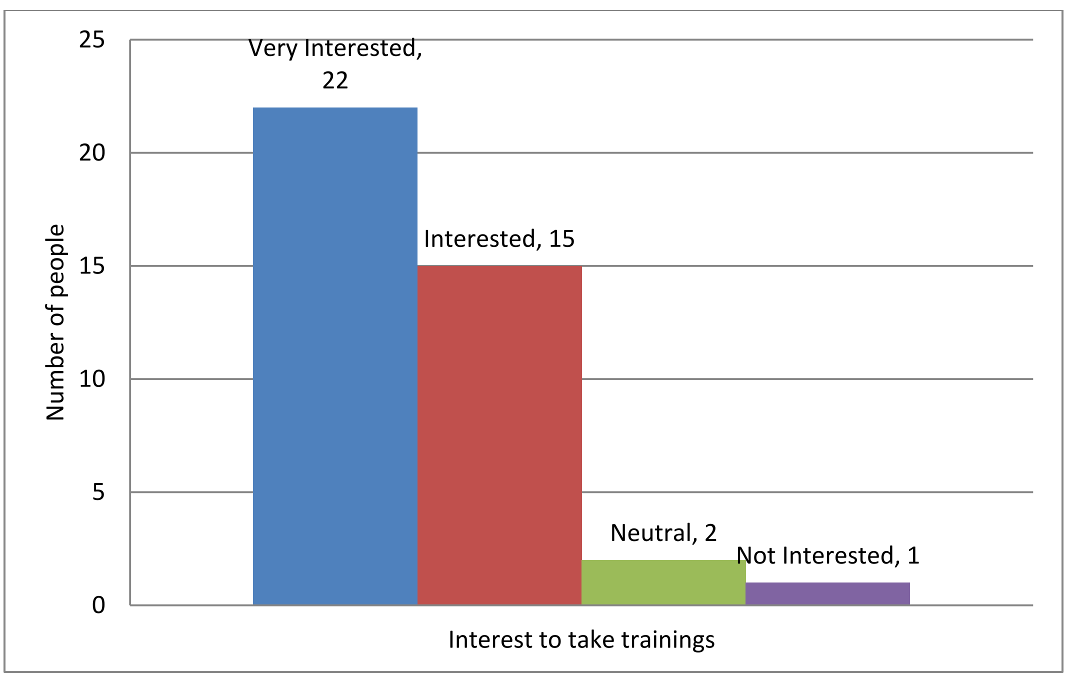


Figure 6.7: Willingness of participants to take trainings

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Conclusions

Summing up the quantitative and qualitative part of the research, the following conclusions have been given.

1. As shown in the quantitative part of the research, the newly adopted precast concrete lattice slab reduces the weight and cost of slab as compared to ribbed slab. Therefore, this new method of construction gives solution for the low cost buildings in terms of cost and quality.
2. The normal weight precast concrete lattice slab reduces the cost of a single slab by 35% where as the light weight precast concrete lattice slab reduces the cost of a single slab by 45% and the weight of a single slab by 30% as compared to the conventional method of slab construction (ribbed slab) for the low cost housing projects.
3. As the aim of the research is to provide recommendations and solutions so as to expand the newly adopted precast lattice slab to other projects, in the qualitative part of the research, market assessment was carried out to determine the reasons that kept the construction society from using the available forms of precast concrete building elements in the market.
4. From the assessment, it was found out that the major reasons that caused the low demand for precast technology include doubt in its quality, allowance of design freedom, doubt in its cost efficiency, lack of knowledge, ease of manufacture and construction, policy of the state and other unmentioned reasons.

7.2. Recommendations

1. While working with precast building elements, the transportation and erection of elements always need a careful attention. In order to minimize the transportation cost, in the same way as the precast ribs are being produced, it is recommended that the precast lattice plates may be produced at the construction site.
2. It is recommended to continuously supervise and monitor the fabrication process of precast plates so as to prevent the quality of the products from deterioration. In addition to that, trial mixes taken from random batch of concrete mixes should be tested in the laboratory. Another thing which can be done to increase the quality of products is importing high tech machineries which facilitate automated production and minimize the manual contribution.

3. Until recent times, the Ethiopian Government had preference on cast insitu method of construction. The reason behind is that cast insitu method of construction is labor intensive and as

a result provides job opportunity for a lot of people and on the contrary, precast method of construction requires few but skilled manpower. But these days, while using a composite method of construction on the low cost housing projects, the additional number of people that would have been employed in the cast insitu construction now gets involved in the production of the precast products. This idea has been practicable by means of Micro and Small Enterprises which fabricate hollow blocks and precast ribs. Therefore it is recommended to further stretch this scheme for the production of precast lattice plates by assisting the small enterprises to get involved in the production process rather than the construction process.

4. Assistance from the government can be facilitating periodic trainings by bringing experts from abroad, promoting their products, providing working place, supplying raw materials and offering loans (financial support). Rather than fabricating these products in a single company, as is the case now, having micro and small enterprises produce them in a decentralized way has the following advantages: the products will be available everywhere minimizing transportation cost, in order to increase competitiveness variety of products will be available and creates job opportunity.
5. Finally it is recommended to do further research on precast lattice slab and other available precast technologies. It is believed by the researcher that using void formers such as polystyrene in precast lattice slab and using other light weight precast elements such as beams, columns, stairs and walls would very much minimize the cost and the weight of construction therefore further studies on these and other new technologies would help grow the construction technology in the country.

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APPENDIX A: General Design Chart

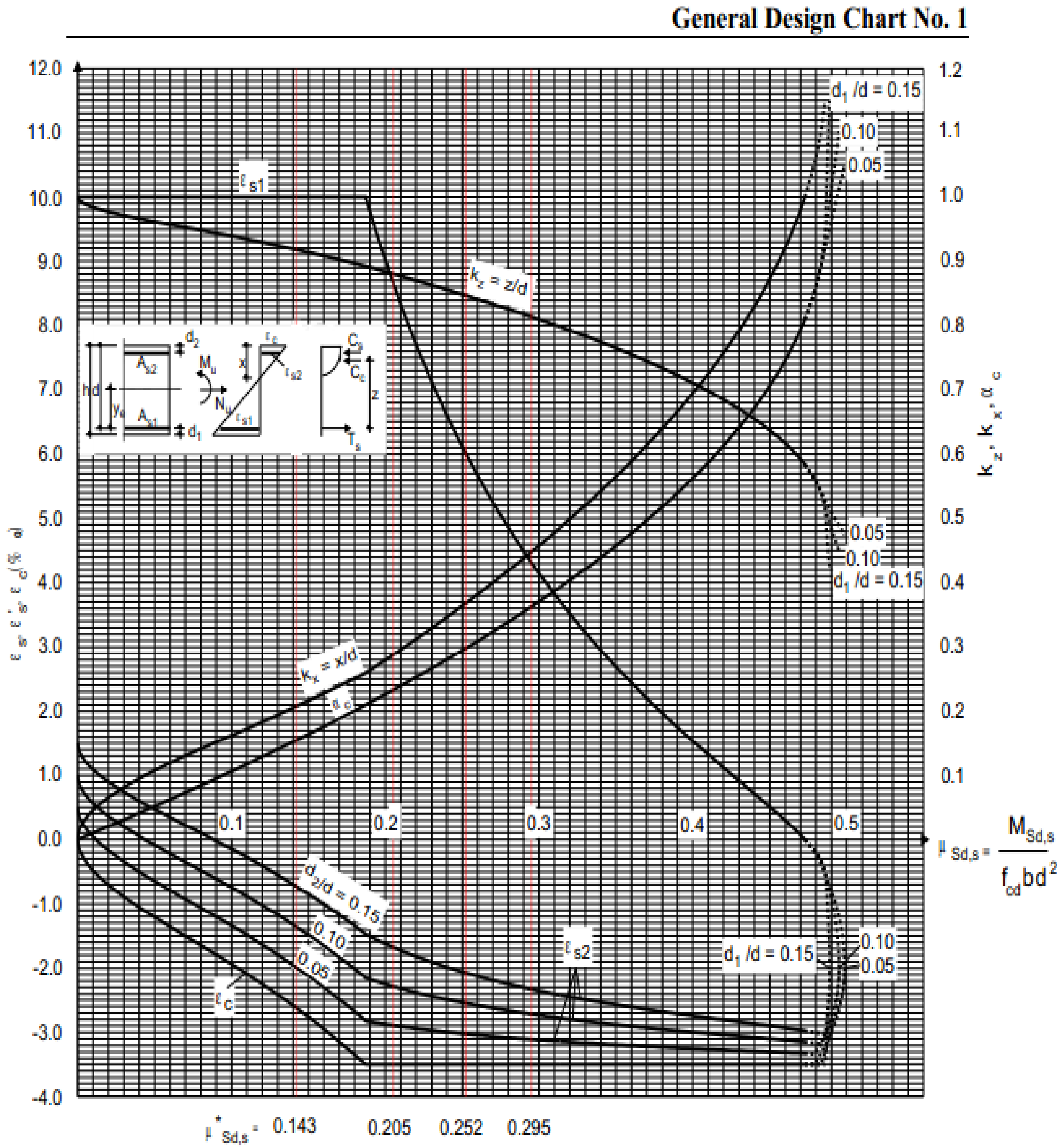


Figure A.1: General Design Chart

APPENDIX B : Calculation Of Design Constants

B.1. MATERIAL CONSTANTS

- **Concrete C-25/30**

In general for normal weight, according to EN 1992-1-1:2004, the design strength of concrete is defined as:

$f_{cd} = \frac{0.85 \times f_{ck}}{\gamma_c}$ in compression, and

$f_{ctd} = \frac{f_{ctk}}{\gamma_c}$ in tension

Where,

f_{ck} and f_{ctk} - are the characteristic standard cylinder compressive and tensile strength of concrete, respectively.

γ_c – is partial safety factor for concrete which is taken as 1.5 for class I works.

For C-25/30,

$f_{ck} = 0.8 \times f_{cu} = 0.8 \times 25$ MPa=20 MPa

Where f_{cu} is the characteristic standard compressive cube strength

$f_{ctk} = 0.21 \times [(f_{ck})^{(2/3)}] = 0.21 \times [(20)^{(2/3)}] = 1.55$ MPa

Therefore, from above,

$$f_{cd} = \frac{0.85 \times 20}{1.5} = 11.33 \text{ MPa}$$

$$f_{ctd} = \frac{1.55}{1.5} = 1.03 \text{ MPa}$$

For C-25 the mean value of secant modulus of E_{cm} can be estimated as 29 GPa.

The linear coefficient of thermal expansion of normal weight concrete is taken to be equal to 10×10^{-6} K^{-1}.

- **Concrete LC-25/28**

LC-25/28 is a class of light weight concrete used in this research. In the same way as normal weight concrete, the design strength of light weight concrete according to EN 1992-1-1:2004 is given by

$f_{lcd} = \frac{0.85 \times f_{lck}}{\gamma_c}$ in compression, and

$f_{lctd} = \frac{f_{lctk}}{\gamma_c} = \frac{\eta_1 \times f_{ctk}}{\gamma_c}$ in tension

Where,

f_{lck} and f_{lctk} - are the characteristic standard cylinder compressive and tensile strength of light weight concrete respectively.

γ_c – is partial safety factor for concrete which is taken as 1.5 for class I works.

η_1 – is a coefficient for determining tensile strength which is defined as:

$\eta_1 = 0.4 + 0.6\ (\frac{\rho}{2200})$ and ρ is the oven-dry density of light weight concrete in kg/m^3 which is given by 1656.2 kg/m^3

$\eta_1 = 0.4 + 0.6\ (\frac{1656.2}{2200}) = 0.85$

For LC-25/28,

f_{lck} is read from table 11.3.1 of EN 1992-1-1:2004 as 25 MPa.

$f_{ctk} = 0.21 \times [(f_{lck})^{(2/3)}] = 0.21 \times [(25)^{(2/3)}] = 1.79$ MPa

$f_{lctk} = \eta_1 \times f_{ctk} = 0.85 \times 1.79 = 1.52$ MPa

Therefore, from above,

$$f_{lcd} = \frac{0.85 \times 25}{1.5} = 14.17 \text{ MPa}$$

$$f_{lctd} = \frac{1.52}{1.5} = 1.01 \text{ Mpa}$$

The mean value of secant modulus (E_{lcm}) of light weight concrete is given by

$E_{lcm} = E_{cm} \times \eta_E$

Where E_{cm} is the mean value of secant modulus of normal weight concrete with a comparable strength to the light weight concrete, it is read as 31 GPa from table 3.1 of EN 1992-1-1:2004.

η_E is a conversion factor for calculating the modulus of elasticity and it is calculated as:

$$\eta_E = [\frac{\rho}{2200}]^2$$

$$\eta_E = [\frac{1656.2}{2200}]^2 = 0.57$$

Hence, $E_{lcm} = (31 \times 0.57) = 17.7$ GPa

The linear coefficient of thermal expansion of light weight concrete is taken to be equal to $8 * 10^{-6}$ K^{-1}.

- **Steel (S-300)**

According to Eurocode 3-1992, the design strength of steel is given by:

$$f_{yd} = \frac{f_{yk}}{\gamma_s}$$

Where f_{yk}= the characteristic yield strength of reinforcing steel

γ_S = Partial safety factor for reinforcing steel

For class I works, $\gamma_S = 1.15$

For S-300, f_{yk} = 300 MPa

Hence, $f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{300}{1.15} = 260.87$ MPa

B.2. UNIT WEIGHT

According to EN 1991-1-1:2001,

$\gamma_{\text{normal concrete}} = 25$ kN/m^3

$\gamma_{\text{light concrete}} = 16.56$ kN/m^3

$\gamma_{\text{block}} = 14$ kN/m^3

$\gamma_{\text{mortar}} = 23$ kN/m^3

APPENDIX C : Design Load Calculation

C.1. Production Stage

This phase covers the process of manufacturing of the 0.06m thick precast lattice plate along with the lifting, the transportation and the handling of the cured plate. In this phase the plate is subjected to self weight (dead load) with no additional live load.

- For normal weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m)\times(25\ kN/m^3) = 1.5\ kN/m^2$

Live load:

$LL = 0\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times 1.5\ kN/m^2] + [1.5 \times 0\ kN/m^2] =$ ***2.02 kN/m²***

- For light weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m)\times(16.56\ kN/m^3) = 0.99\ kN/m^2$

Live load:

$LL = 0\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times 0.99\ kN/m^2] + [1.5 \times 0\ kN/m^2] =$ ***1.34 kN/m²***

Service load = $DL + LL = (0.99 + 0)\ kN/m^2 = 0.99\ kN/m^2$

C.2. Construction Stage

This stage has two sub stages named the Erection Stage and the Concrete Pouring Stage. It is the time of construction which is between the delivery of the produced plates to the construction site and the setting of fresh concrete topping. Temporary supports are used in this stage to support the slab. The number of props used to support a single plate depends on the load to span ratio and diameter of top bar.

C.2.1. Erection Stage

In this stage, constructors erect the precast lattice plates in their permanent position. Here the self weight of the plank is considered as dead load and the load of the workers as live load which is taken to be $1 kN/m^2$ according to EN 13747: Annex J.

- For normal weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m)\times(25\ kN/m^3) = 1.5\ kN/m^2$

Live load:

$P_2 = 1\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times 1.5\ kN/m^2] + [1.5 \times 1\ kN/m^2] =$ ***3.52 kN/m²***

- For light weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m)\times(16.56\ kN/m^3) = 0.99\ kN/m^2$

Live load:

$P_2 = 1\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times 0.99\ kN/m^2] + [1.5 \times 1\ kN/m^2] =$ ***2.84 kN/m²***

C.2.2 Concrete Pouring Stage

While estimating the design load in this stage, two alternatives should be considered.

i. The concrete as live load on the poured side. Since the concrete will not be usable for workers before setting, the workers load and the fresh concrete cannot be counted as live load together.

- For normal weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m)\times(25\ kN/m^3) = 1.5\ kN/m^2$

Live load:

$P_3 = P_{fresh\ concrete} = (D - h_p) \times \gamma_c = (0.2 - 0.06) \times (25\ kN/m^3) = 3.5\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times 1.5\ kN/m^2] + [1.5 \times 3.5\ kN/m^2] =$ ***7.28 kN/m²***

- For light weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m) \times (16.56\ kN/m^3) = 0.99\ kN/m^2$

Live load:

$P_3 = P_{fresh\ concrete} = (D - h_p) \times \gamma_c = (0.2 - 0.06) \times (16.56\ kN/m^3) = 2.32\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times 0.99\ kN/m^2] + [1.5 \times 2.32\ kN/m^2] =$ ***4.82 kN/m²***

ii. The concrete part could set and the construction workers can use it as a plat form. Therefore, the construction workers as live load and the concrete as dead load is the other alternative.

- For normal weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m) \times (25\ kN/m^3) = 1.5\ kN/m^2$

$P_3 = P_{fresh\ concrete} = (D - h_p) \times \gamma_c = (0.2 - 0.06) \times (25\ kN/m^3) = 3.5\ kN/m^2$

Live load:

$P_2 = 1\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times (1.5 + 3.5)\ kN/m^2] + [1.5 \times 1\ kN/m^2] =$ **8.25 kN/m²**

- For light weight concrete

Dead load:

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m) \times (16.56\ kN/m^3) = 0.99\ kN/m^2$

$P_3 = P_{fresh\ concrete} = (D - h_p) \times \gamma_c = (0.2 - 0.06) \times (16.56\ kN/m^3) = 2.32\ kN/m^2$

Live load:

$P_2 = 1\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times (0.99 + 2.32)\ kN/m^2] + [1.5 \times 1\ kN/m^2] =$ **5.97 kN/m²**

Among the two alternatives under the concrete pouring stage, the second option governs and hence calculations for the construction stage are as per the governing sub stage.

C.3. Final Stage

This stage characterizes the slab after the hardening of the fresh concrete poured in the construction stage. The full depth of the slab is to be considered in this phase. The building considered in this research is residential and hence a live load of $2kN/m^2$ is assumed as per EN 1991-1-1:2001.

- For normal weight concrete

Dead load: Taking 30mm cement screed and 20 mm plaster

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m)\times(25\ kN/m^3) = 1.5\ kN/m^2$

$P_3 = P_{fresh\ concrete} = (D - h_p) \times \gamma_c = (0.2 - 0.06)\times (25\ kN/m^3) = 3.5\ kN/m^2$

$P_4 = P_{cement\ screed} = t \times \gamma_{screed} = (0.03m) \times (23\ kN/m^3) = 0.69\ kN/m^2$

$P_5 = P_{plaster} = t \times \gamma_{plaster} = (0.02m) \times (23\ kN/m^3) = 0.46\ kN/m^2$

$P_6 = P_{partition} = 1\ kN/m^2$

Live load:

$P_2 = 2\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times (1.5 + 3.5 + 0.69 + 0.46 + 1)\ kN/m^2] + [1.5 \times 2\ kN/m^2]$

$= \boldsymbol{12.65\ kN/m^2}$

Service load = $DL + LL = (1.5 + 3.5 + 0.69 + 0.46 + 1 + 2)\ kN/m^2 = \boldsymbol{9.15\ kN/m^2}$

- For light weight concrete

Dead load: 30mm cement screed and 20 mm plaster

$P_1 = P_{plate} = h_p \times \gamma_c = (0.06m)\times(16.56\ kN/m^3) = 0.99\ kN/m^2$

$P_3 = P_{\text{fresh concrete}} = (D - h_p)\times\gamma_c = (0.2 - 0.06)\times (16.56\ kN/m^3) = 2.32\ kN/m^2$

$P_4 = P_{\text{cement screed}} = t \times \gamma_{screed} = (0.03m) \times (23\ kN/m^3) = 0.69\ kN/m^2$

$P_5 = P_{plaster} = t \times \gamma_{plaster} = (0.02m) \times (23\ kN/m^3) = 0.46\ kN/m^2$

$P_6 = P_{partition} = 1\ kN/m^2$

Live load:

$P_2 = 2\ kN/m^2$

Design load = $P_d = 1.35\ DL + 1.5\ LL = [1.35 \times (0.99 + 2.32 + 0.69 + 0.46 + 1)\ kN/m^2] + [1.5 \times 2kN/m^2]$

$= \mathbf{10.37\ kN/m^2}$

Service load = DL + LL = $(0.99 + 2.32 + 0.69 + 0.46 + 1 + 2)\ KN/m^2 = \mathbf{7.46\ kN/m^2}$

Type of concrete used	Loads acting on the slab	Stages of construction			
		Production stage	Construction stage		Final stage
			Erection stage	Concrete pouring stage	
Normal weight concrete	Ultimate Load (kN/m^2)	2.02	3.52	8.25	12.65
	Service Load (kN/m^2)	-	-	-	9.15
Light weight concrete	Ultimate Load (kN/m^2)	1.34	2.84	5.97	10.37
	Service Load (kN/m^2)	-	-	-	7.46

Table C.1: Summary of load history

APPENDIX D : Design Example

In this section, an example showing how the design of composite concrete slabs made of precast lattice plates is presented. The analysis and design of a whole slab-beam system is performed for both normal weight and light weight concrete of a typical slab in G+9 residential building (low cost building) but since the design procedure is similar for all plates, the example covers only the analysis and design of a single plate through the production, construction and final stage.

D.1. Plate Design: Normal Weight Concrete

The normal weight concrete is characterized by a concrete grade of C-25 with a unit weight of 25kN/m^3 and density of 2500kg/m^3.

- f_{cu} = 25MPa f_{ctk} = 1.55MPa
  f_{ck} = 20MPa f_{ctd} = 1.03MPa
  f_{cd} = 11.33MPa   E = 29GPa

Plate sizes to be used are:

- Plate – 1: L=5.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 170mm, D = 200mm
- Plate – 2: L=4.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 170mm, D = 200mm
- Plate – 3: L=4.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 171mm, D = 200mm

D.1.1. Ultimate Limit State: Flexure Design

The design example will only cover the analysis and design of Plate – 1.

- *Use 11 Ø 10 c/c 110 (tension), 2 Ø 10 (compression) and Ø 6 c/c 150 shear reinforcement*

D.1.1.1. Production Stage (Premature Buckling of Members)

The ultimate limit state requirement of the Production stage is studied by the structural software SAP2000V14 using design preference Eurocode-3, 1993. A sample hand calculation to check the premature buckling of members is presented in the following section. Two hundred fifty eight members (stirrups, top and bottom chords) exist in this model, and in this section one specific member (stirrup) is selected from the model shown below, for a joint load of 0.09kN with three lifting points (two at exterior points and another at mid point) acting as a simple support for the 3D model.

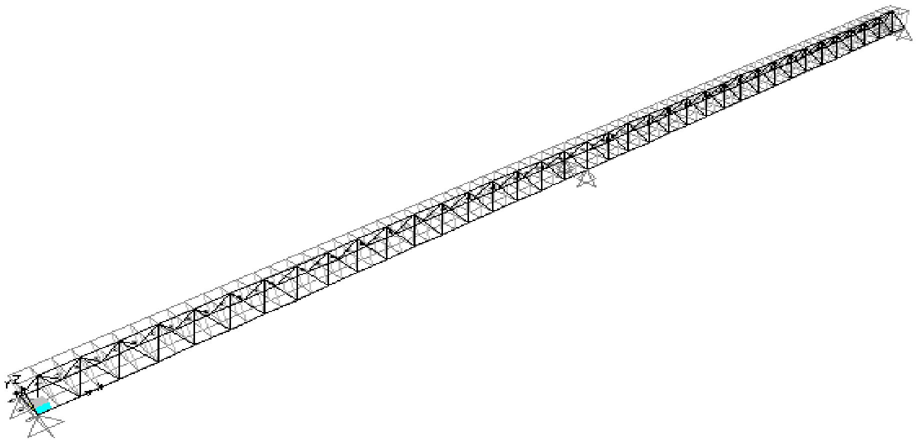


Figure D.1: 3D modeling of the latticed shear reinforcements of Plate-1 on SAP2000V14

$P_d = 2.02\,\frac{kN}{m^2}$ (from Table C.1)

change to joint load

$P_d = \frac{1}{2}\,(2.02\frac{kN}{m^2})*\,(s)*(\frac{L}{2}) = \frac{1}{2}\,(2.02)\,(0.15m)\,(\frac{1.2m}{2}) = 0.18kN$

Label of the member: 430 and *D – C ratio*: 0.8

Analysis output from SAP2000V14:

P = 0.772kN (compression), L = 0.18371 m

End Moment : Along the axis Y – Y: -2.6×10^{-3} kNm and 6.36×10^{-4} kNm

Along the axis Z – Z: 2.0×10^{-3} kNm and -3.59×10^{-4} kNm

The buckling resistance of compression members is given by:

$$\frac{N_{sd}}{\chi_{min}\frac{Af_y}{\gamma_{m1}}} + \frac{K_y M_{y,sd}}{W_{pl,y}\frac{f_y}{\gamma_{m1}}} + \frac{K_z M_{z,sd}}{W_{pl,z}\frac{f_y}{\gamma_{m1}}} \leq 1 \qquad \text{(from Sec 5.5.4, Eurocode-3,1993)}$$

$\lambda_1 = 93.9\,(\frac{235}{f_y})^{0.5} = 93.9\,(\frac{235}{300})^{0.5} = 83.11$

$i = \frac{r}{2} = \frac{3}{2} = 1.5mm$ and $l = 0.18371$

According to Eurocode-3, 1993, for members resisting loads other than wind load, $\lambda \leq 180$

$\lambda_y = \lambda_z = \frac{l}{i} = \frac{183.71mm}{1.5mm} = 122.47$, $\beta_A = 1$ for Class 1

$\overline{\lambda_y} = \overline{\lambda_z} = \frac{\lambda_y}{\lambda_1}\sqrt{\beta_A} = = \frac{122.47}{83.11}\sqrt{1} = 1.47$

α = 0.49, (from table 5.5.3 and table 5.5.1, Eurocode-3, 1993)

For buckling curve c, $\bar{\lambda}$ = 1.47 and using interpolation

$\chi_y = \chi_z = 0.3257$ (from table 5.5.2 Eurocode-3, 1993)

$\chi_{min} = 0.3257$

$\psi_y = \frac{6.36\times10^{-4}}{-2.6\times10^{-3}} = -0.245$

$\beta_{M,Y} = 1.8 - 0.7\,\psi = 1.8 - (0.7 \times -0.245) = 1.628$

$\psi_z = \frac{-3.59\times10^{-4}}{2\times10^{-3}} = -0.18$

$\beta_{M,z} = 1.8 - 0.7\,\psi = 1.8 - (0.7 \times -0.18) = 1.926$

$w_{pl,y} = w_{pl,z} = Z = \frac{d^3}{6} = \frac{6^3}{6} = 36\ mm^3$

$w_{el,y} = w_{el,z} = s = \frac{\pi d^3}{32} = \frac{\pi 6^3}{6} = 21.2\ mm^3$

$\mu_y = \overline{\lambda_y}[2\beta_{M,Y} - 4] + [\frac{w_{pl,y}-w_{el,y}}{w_{el,y}}] \leq 0.9$

$\mu_y = 1.47\times[2\times1.628 - 4] + [\frac{36-21.2}{21.2}] = -0.396 \leq 0.9$

$\mu_z = \overline{\lambda_z}[2\beta_{M,z} - 4] + [\frac{w_{pl,z}-w_{el,z}}{w_{el,z}}] \leq 0.9$

$\mu_z = 1.47\times[2\times1.926 - 4] + [\frac{36-21.2}{21.2}] = 0.481 \leq 0.9$

$k_y = 1- \frac{\mu_y\times N_{sd}}{\chi_y\times A\times f_y} \leq 1.5$

$k_y = 1- \frac{-0.396\times0.772\times10^3}{0.3257\times28.27\times10^{-6}\times260.87\times10^6} = 1.127$

$k_z = 1- \frac{\mu_z\times N_{sd}}{\chi_z\times A\times f_y} \leq 1.5$

$k_z = 1- \frac{0.481\times0.772\times10^3}{0.3257\times28.27\times10^{-6}\times260.87\times10^6} = 0.845$

Demand to capacity ratio

$$\frac{N_{sd}}{\chi_{\min}(\frac{A\times f_y}{\gamma_{m1}})} + \frac{K_y M_{y,sd}}{W_{pl,y}(\frac{f_y}{\gamma_{m1}})} + \frac{K_z M_{z,sd}}{W_{pl,z}(\frac{f_y}{\gamma_{m1}})} \leq 1$$

$$\frac{1.1\times0.772\times10^3}{0.3257\times28.27\times10^{-6}\times260.87\times10^6} + \frac{1.1\times1.127\times0.0026\times10^3}{3.6\times10^{-8}\times260.87\times10^6} + \frac{1.1\times0.886\times0.002\times10^3}{3.6\times10^{-8}\times260.87\times10^6} = 0.644$$

SAP result = 0.615

ratio = $\frac{0.615}{0.644}$ = 95.5%, difference is less than 5%

D.1.1.2. Construction Stage

a) Design Moment

The load for the construction stage from Table C.1 (8.25kN/m^2) should be multiplied by the width of the plate (1.2m) to change the loading from areal load to line load.

The analysis of the slab strip is executed on ETABS v 9.7.4 and the maximum internal moment is found to be ***3.16kNm.***

b) Section Capacity

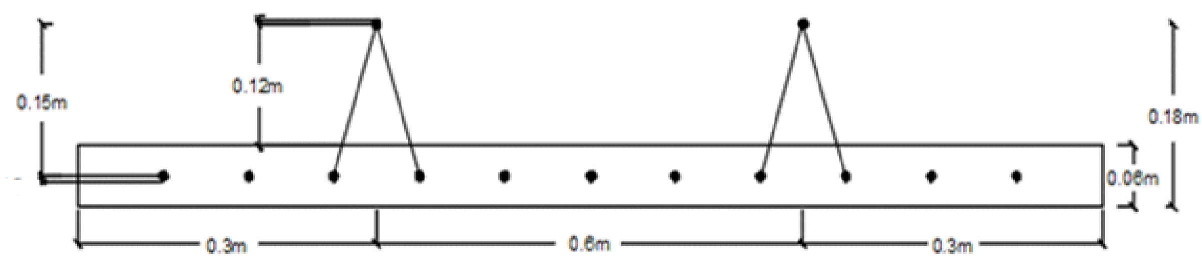


Figure D.2: Section view of plate-1 for the construction stage

The following assumptions are made in order to estimate the section capacity of the plate in the production and construction stage.

- ✓ The neutral axis lies in the concrete area.
- ✓ All reinforcement bars have yielded.
- ✓ The concrete has crushed.
- ✓ The section is under balanced failure.

$d_p = D_p – C_c - Ø_{sr} - Ø_r/2$

Where,

d_p - is effective depth of plank.

D_p - is total depth of plank

Cc - Concrete cover, 15mm

$\emptyset_{sr}$ - diameter of shear reinforcement, 10mm

$\emptyset_{r}$ - diameter of longitudinal reinforcement, 10mm

d_p = (60 – 15 – 10 – 5) mm = 30mm

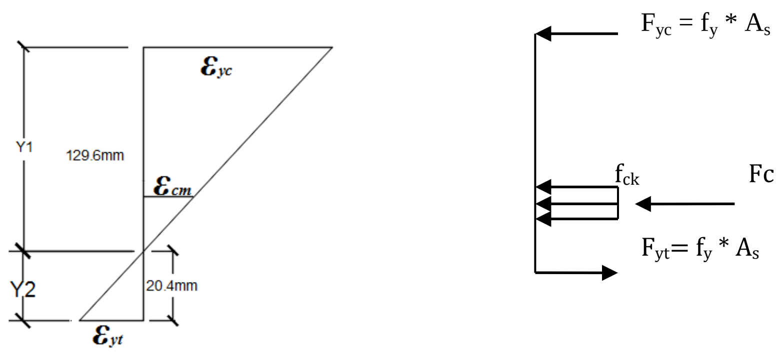


Figure D.3: Stress and Strain diagram

- From equilibrium equation

$$F_{yc}+F_c=F_{yt} ,$$

Where $F_{yc} = 260.87\frac{N}{mm^2} \times \frac{2 \times \pi \times 10^2}{4} = 41kN$, $F_{yt} = 260.87\frac{N}{mm^2} \times \frac{11 \times \pi \times 10^2}{4} = 225.3kN$

$$F_c = F_{yt} - F_{yc} = 225.3kN - 41kN = 184.3kN$$

And from the section capacity of the concrete (rectangular stress block),

$$F_c = (0.8x) \times f_{ck} \times b_w \Rightarrow \quad x = \frac{F_c}{0.8 \times f_{ck} \times b_w} = \frac{184.3 \times 10^3 kN}{0.8 \times 20\frac{N}{mm^2} \times 1200mm} = 9.6mm$$

- Check whether the steel has yielded and the N.A. lies in the concrete.

From similarity of triangles,

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{y_1}{y_2} = \frac{120 + 9.6}{20.4} = 6.35$$

$$\Rightarrow \frac{\varepsilon_{yc}}{6.35} = \varepsilon_{yt}$$

⇒Therefore the top and bottom reinforcements would not reach the yield point simultaneously.

$$\varepsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{260.87MPa}{200GPa} = 1.3\ {}^{0}/_{00}$$

- Determine the exact strain in the concrete

$$\frac{\varepsilon_{yc}}{\varepsilon_{cm}} = \frac{y_1}{y_1 - 120} = \frac{120+9.6}{9.6} = 13.5$$

$$\varepsilon_{cm} = \frac{\varepsilon_{yc}}{13.5} = \frac{1.3‰}{13.5} = 0.1‰ < 3.5\ ‰$$

Therefore if there is no premature buckling of bars, the top reinforcement would reach the yield point first and next determine the exact strain conditions.

From above,

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{1.3‰}{\varepsilon_{yt}} = \frac{129.6}{20.4}$$

$$\varepsilon_{yt} = 0.21‰$$

$$f_{st} = \varepsilon_{yt} \times E_s = \frac{0.21}{1000} \times\ 200\ GPa\ = 42\ MPa$$

$$F_{yT} = 42 \frac{N}{mm2} \times \frac{11 \times \pi \times\ 10^2}{4} = 36.3kN$$

From similarity of triangles

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{a}{160-a} \Rightarrow \frac{1.3‰}{0.21‰} = 6.2 = \frac{a}{160-a}$$ Solving this

$$a = 137.8\ mm$$

x = a – 120 = 17.8mm (Neutral axis depth within the concrete)

And from equilibrium equation

$F_{yc} + F_c = F_{yt}$ Where $F_{yc} = 260.87\frac{N}{mm^2} \times \frac{2\times \pi\times10^2}{4}$, $F_{yt} = 36.3kN$

$= 41kN$

F_c = (36.3 – 41) kN = -4.7kN $\Rightarrow$ the concrete section must resist this tension force (not possible!)

$\Rightarrow$ N.A. is not in the concrete section.

Assumption: the N.A. lies in between the concrete and the top reinforcement.

$$F_{yc} = F_{yt} \quad \Rightarrow \quad 260.87\frac{N}{mm^2} \times \frac{2\times\pi\times10^2}{4} = f_{yt} \times \frac{11\times\pi\times10^2}{4}$$

$$f_{yt} = 47.4\frac{N}{mm^2} = 47.4MPa$$

$$\Rightarrow F_{yt} = 47.4\frac{N}{mm^2} \times \frac{11\times\pi\times10^2}{4} = 40.93kN$$

From linear stress and strain relationship

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{47.4MPa}{200GPa} = 0.24{}^{0}/_{00} < 1.3{}^{0}/_{00}$$

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{a}{160-a} \Rightarrow \frac{1.3‰}{0.24‰} = 5.42 = \frac{a}{160-a}$$ Solving this

a = 126.6 mm≈ 125 mm (negligible concrete area under tension)

From the above conditions the flexural capacity of concrete would be

$M_{rd} = F_{yc}$ x 0.15m = 41kN x 0.15m

= 6.15 kNm > 3.16kNm ...... Safe!

D.1.1.3. Final Stage

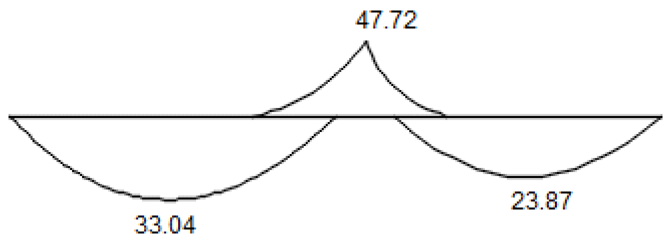
a) Design Moment

As per Table C.1, the design load (P_d) for the final stage is 12.65kN/m^2. When loading the slab strip, the design areal load should be changed into line load by multiplying it with the width of the strip (1.2m).

Pattern loading for the final stage shall first be performed and the moment envelope be developed. The slab strip having the same width as the individual plates is modeled on ETABS 9.7.4 in order to develop the moment envelope.

- **Moment Envelope**

PART – A



PART – B

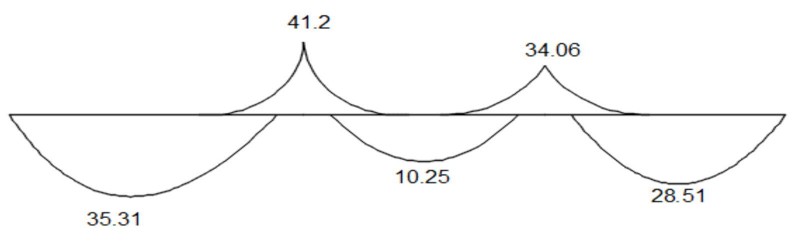


Figure D.4: Moment envelope developed on ETABS for both Part A and Part B

From the moment envelope, there are three groups of *span moment* values:

1. M_{sd1} = 35.31kNm, M_{sd2} = 33.04kNm, M_{sd3} = 28.51kNm (Plate - 1)
2. M_{sd1} = 23.87kNm (Plate - 2)
3. M_{sd1}= 10.25kNm (Plate - 3)

Hence the design moment of plate-1 for the final stage is **35.31kNm.**

b) Section Capacity

The amount of longitudinal reinforcement is first assigned and then the section capacity of the plate is calculated. But before that, the minimum reinforcement area has to be determined so that the area of reinforcement provided should be greater than the minimum reinforcement area.

According to EN 1992-1-1:2004, Section 9.2.1.1, the area of longitudinal tension reinforcement should not be taken as less than $A_{s,min}$ which is defined as follows:

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d > 0.0013 b_t d$$ (EN 1992-1-1:2004: **9.1**)

Where,

b_t - denotes the mean width of the tension zone,

f_{ctm} - should be determined with respect to the relevant strength class according to Table 3.1 of EN 1992-1-1:2004, for C-25 f_{ctm} is read as 2.2MPa,

f_{yk} And d – defined previously

$A_{smin} = \frac{0.26 \times 2.2MPa \times 1200mm \times 170mm}{300MPa}$ = 389mm² > 0.0013b$_t$ d = 0.0013×1200mm× 170mm = 265mm²

Therefore the area of main reinforcement assigned should be equal to or greater than **$A_{s,min}$ = 389mm².**

- Use 11 Ø 10 c/c 110 (tension), 2 Ø 10 (compression), Msd = 35.31kNm (take maximum)

  d= D – C_c - $\emptyset_{sr}$ - $\emptyset_r$/2 = (200 – 10 - 15 – 5) mm = 170mm, d_2 = 20mm,

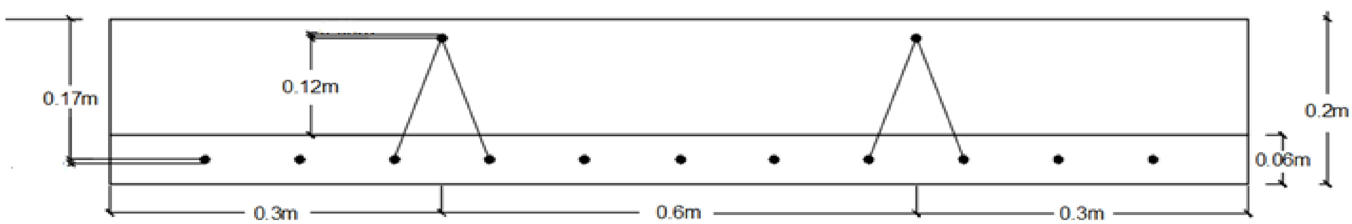


Figure D.5: Section view of plate-1 for the final stage

- .A_{s1} (tension) = n ($\frac{\pi \times d^2}{4}$), where n is number of bars and d is diameter of bar.

  $A_{s1} = 11(\frac{\pi \times 10^2}{4})$ = 863.5mm².

- A_{s2} (Compression) = $2(\frac{\pi \times 10^2}{4})$ = 157mm²

The plate is considered as *a doubly reinforced section* and the following assumptions are used in order to estimate the section capacity of the plate.

- ✓ The neutral axis is within section.
- ✓ All reinforcements have yielded.

$\varepsilon_{yd} = \frac{f_{yd}}{E_s}$ where ε_{yd} is yield strain in the reinforcement.

$\varepsilon_{yd} = \frac{260.87MPa}{200GPa} = 1.3\,{}^0/_{00}$

$\alpha_c = \frac{(A_{s1} - A_{s2})\, f_{yd}}{f_{cd}\, bd}$ (EN 1992-1-1:2004)

Where α_c is a constant used in the calculation of moment and all the other terms are defined previously.

$\alpha_c = \frac{(863.5 - 157)260.87}{11.3 \times 1200 \times 170} = 0.08$

εs_1 and εs_2 which are defined as strain in the top and bottom reinforcement respectively are then read from the general design chart.

$\varepsilon s_1 = 10\,{}^0/_{00}$ and $\varepsilon s_2 = -0.6{}^0/_{00} < \varepsilon y$d…………assumption not ok!

Trial – 1

$f_s = \varepsilon_s E$ where f_s is stress and ε_s strain in the reinforcement bar.

$f_s = -0.6 \times 10^{-3} \times 200$GPa = -120MPa

$\alpha_c = \frac{(A_{s1} f_{yd} - A_{s2} f_s)}{f_{cd}\, bd} = \frac{(863.5 * 260.87) + (157 * 120)}{11.3 * 1200 * 170} = 0.11$

Read $\varepsilon s_1 = 10\,{}^0/_{00}$ and $\varepsilon s_2 = -0.8{}^0/_{00}$

Trial – 2

$f_s = \varepsilon_s E = -0.8 \times 10^{-3} \times 200$GPa = -160MPa

$\alpha_c = \frac{(A_{s1} f_{yd} - A_{s2} f_s)}{f_{cd}\, bd} = \frac{(863.5 \times 260.87) + (157 \times 160)}{11.3 \times 1200 \times 170} = 0.11$

Read $\varepsilon s_1 = 10\,{}^0/_{00}$ and $\varepsilon s_2 = -0.8{}^0/_{00}$ (the iteration converges)

Read $\mu_{sd} = 0.105$ (from the general design chart)

Section Capacity

$M_{rd} = \mu_{sd}\, f_{cd}\, bd^2 + A_{s2}\, f_s\, (d - d_2)$

M_{rd} = [0.105×11.3MPa×1200mm×(170mm)^2] + [157mm^2×(-160MPa)×150mm]

M_{rd} = 37.4kNm > M_{sd} = 35.31kNm (maximum span moment for Plate-1)……safe!

c) Design of Secondary Reinforcement

Plate – 1

According to EN 1992-1-1:2004, section 9.3.1,

- minimum reinforcement for secondary reinforcement is given by

$A_{s,secondary} = 0.2 \times A_{s,main}$

$A_{s,main}$ = (863.5 + 157)mm^2= 1020.5mm^2

$A_{s,secondary} = (0.2 \times 1020.5)mm^2$ = **$\underline{204.1mm^2}$**

- maximum spacing limit for secondary reinforcement is given by:

S_{max} = mini of $\begin{cases} 3.5h = 3.5 \times 200 = 700mm \\ and \\ 450mm \end{cases}$ take S_{max} = **<u>450mm</u>**

Use Ø 6 bars, a_s = 28.26mm^2

S = $\frac{b \times a_s}{A_s} = \frac{5500mm \times 28.26mm^2}{201.1mm^2}$ = 1350mm > S_{max} = 450mm......not ok!

Calculate the amount of reinforcement to be provided using maximum spacing limit.

$A_s = \frac{b \times a_s}{s} = \frac{5500mm \times 28.26mm^2}{450mm}$ = 345.4mm^2 > $A_{s,min}$ = 204.1mm^2.........ok!

$No. of\ bars = \frac{A_s}{a_s} = \frac{345.4mm^2}{28.26mm^2} \approx 13$

- use 13Ø 6 c/c 450mm bars

d) Design of Negative Reinforcement

The design negative moment developed according to the moment envelope diagram (Figure D.4) is ***47.72kNm*** for PART-A (joins Plate – 1 and Plate - 2).

Check section capacity (singly or doubly reinforced)

$$\mu_{sd} = \frac{M_{sd}}{f_{cd}bd^2} \Rightarrow \mu_{sd} = \frac{47.72 \times 10^6}{11.33 \times 1200 \times (170^2)} = 0.12 < \mu_{sd}^* = 0.295$$ ....SRS (singly reinforced section)

Read k_z = 0.93 from the general design chart for μ_{sd} = 0.12 and calculate area of reinforcement to be provided.

$A_{s,tot} = \frac{M_{sd}}{z\, f_{yd}} = \frac{M_{sd}}{Kz\, d\, f_{yd}} = \frac{47.72 \times 10^6}{0.93 \times 170 \times 260.87} = 1157.03mm^2$

But the plates have two compression chords of '∅ 10' which serve as tension chord for the negative moment. Therefore,

A_s = [1157.03 – (2 × 78.5)] mm^2 = 1000mm^2

$No. of\ bars = \frac{A_s}{a_s} = \frac{1000mm^2}{78.5mm^2}$ ≈ 13∅ 10 bars in addition to the 2 ∅ 10 chords.

D.1.2. Ultimate Limit State: Shear Design

D.1.2.1. Construction Stage

a. **Design Shear**

As discussed previously, for the analysis of slab in the construction stage, the one way rectangular plates are modeled as continuous beams having a depth of 0.06m and a width of 1.2m.

Plate - 1 has a length of 5.5m and hence it is temporarily supported at two different points in the construction stage. Once the slab is analyzed, maximum shear values are used for manual design calculations as per EN 1992-1-1:2004 which is summarized as follows.

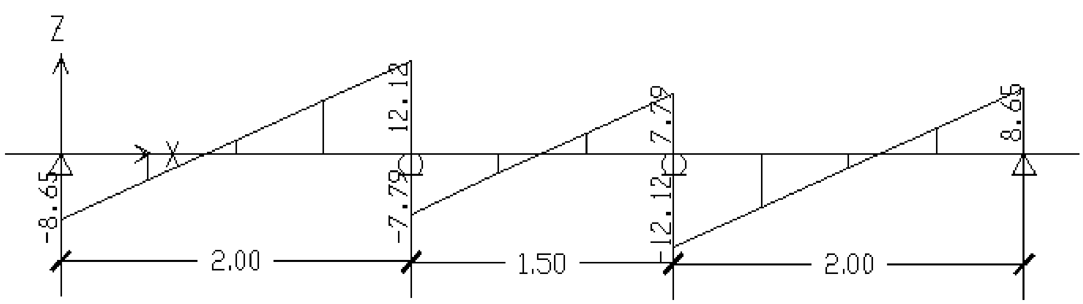


Figure D.6: Shear diagram of Plate – 1 for the construction stage (shear values are in kN)

The maximum shear force from the shear diagram is ***12.12kN***. Taking d distance from the face of the support is almost equal with 12.12kN.

b. **Section Capacity**

Use ∅ 6 c/c 150 shear reinforcement

- Design constants

f_{cd} = 11.33MPa, f_{ctd} = 1.03MPa, f_{yd} = 260.87MPa

- Concrete capacity

$V_{Rd,c} = [C_{Rd,c} K (100\rho_1 f_{ck})^{1/3} + K_1 \sigma_{cp}] b_w d \geq (V_{min} + K_1 \sigma_{cp}) b_w d$

Where,

f_{ck} = 20MPa

$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with d = 30mm

$K = 1 + \sqrt{\frac{200}{30}} = 3.58 > 2.0$ take K = 2

$\rho_1 = \frac{A_{s1}}{b_w d} = \frac{11\times78.5}{1200\times30} = 0.024 > 0.02$ take $\rho_1 = 0.02$

$\sigma_{cp} = \frac{N_{Ed}}{A_c d} < 0.2$ fcd, but $N_{Ed} = 0$ therefore $\sigma_{cp} = 0$

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

$K_1 = 0.15$

$V_{min} = 0.035 K^{3/2} fck^{1/2} = (0.035)(2)^{3/2} (20)^{1/2} = 0.44$

$(V_{min} + K_1 \sigma_{cp}) b_w d = (0.44 + 0)(1200 \times 30) = 15.84$kN

$V_{Rd,c} = [0.12\times2(100 \times 0.02 \times 20)^{1/3} + (0.15\times 0)] (1200 \times 30) =$ **29.55kN** >15.84kN

- Stirrup capacity (inclined)

$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} (\cot\theta + \cot\alpha) \sin\alpha$

Since the stirrup has two inclination angles, we shall classify the stirrup capacity into two.

$V_{Rd,s} = V_{Rd,s1} + V_{Rd,s2}$

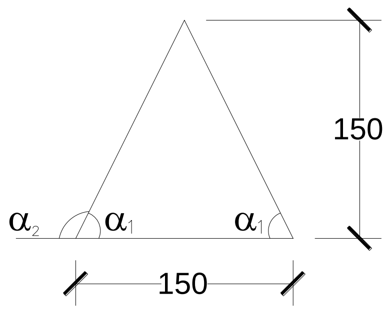


Figure D.7: Stirrup arrangement

$\tan \alpha_1 = \frac{150}{75} = 2$

$\boldsymbol{\alpha_1 = 63.4^0}$

$\alpha_2 = 180 - \alpha_1 = 116.6^0$

$V_{Rd,s1} = \frac{A_{sw}}{s}$ z f_{ywd} $(\cot \theta + \cot \alpha_1) \sin \alpha_1$

$A_{sw} = 2(2 \times a_s) = 113.04\text{mm}^2$

$Z = 0.9d = (0.9 \times 30) = 27\text{mm}$

$f_{ywd} = 260.87\text{MPa}$

$\theta = 45^0$, $\cot \theta = 1$

$\cot \alpha_1 = \cot (63.4^0) = 0.5$

$\sin \alpha_1 = 0.89$

$V_{Rd,s1} = [113.04 \times 27 \times 260.87 (1 + 0.5) \times 0.89]$

$= 7.09\text{kN}$

$\cot \alpha_2 = \cot (116.6^0) = -0.5$

$V_{Rd,s2} = [113.04 \times 27 \times 260.87 (1 - 0.5) \times 0.89]$

$= 2.36\text{kN}$

$V_{Rd,s} = (7.09 + 2.36)\text{kN} =$ **9.45kN**

- $V_{rd,max}$

$V_{rd,max} = V_{rd,max1} + V_{rd,max2}$

$V_{rd,max1} = \alpha_{cw} b_w z \, \nu_1 f_{cd} \left(\frac{\cot\theta + \cot \alpha 1}{1 + cot^2 \theta} \right)$

$V_{rd,max2} = \alpha_{cw}\, b_w\, z\, \nu_1 f_{cd} \left(\frac{\cot\theta + \cot\alpha 2}{1+\cot^2\theta} \right)$

Where,

α_{cw} is 1 for non prestressed structures.

Z = 0.9d = 27mm

$\nu_1 = \nu = 0.6\,[1 - \frac{f_{ck}}{250}]$ $\quad f_{ck}$ = 20MPa solving this $\nu_1 = 0.552$

$\theta = 45^0$, $\cot\theta = 1$

$\cot\alpha_1 = \cot(63.4^0) = 0.5$ and $\cot\alpha_2 = \cot(116.6^0) = -0.5$

$V_{rd,max1} = \frac{[1 \times 1200 \times 27 \times 0.552 \times 11.33 \times (1+0.5)]}{1+1} = 151.57\text{kN}$

$V_{rd,max2} = \frac{[1 \times 1200 \times 27 \times 0.552 \times 11.33 \times (1-0.5)]}{1+1} = 50.66\text{kN}$

$V_{rd,max}$ = (151.57 + 50.66)KN = **202.23kN**

- Shear capacity of member

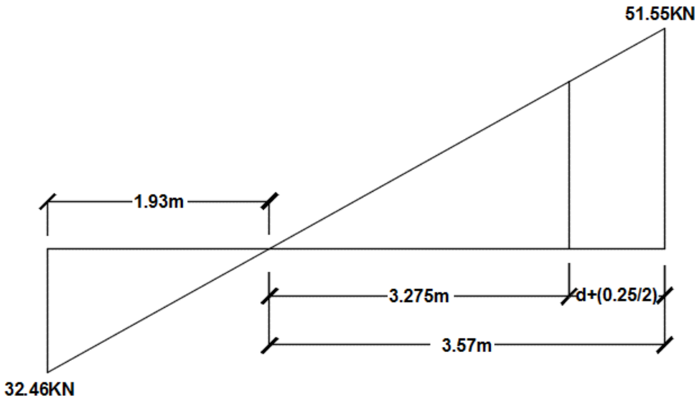
$$V_{Rd} = \text{Min of} \begin{cases} V_{Rd,c} + V_{Rd,s} \\ and \\ V_{rd,max} \end{cases} = \text{min of} \begin{cases} 29.55 + 9.45 = 39kN \\ and \\ 202.23kN \end{cases}$$

V_{Rd} = 39kN > V_{sd} = 12.12kN…………safe!

D.1.2.2. Final Stage

a) Design Shear

Shear design of this stage follows the procedure used in the construction stage, the only difference being the depth of section considered (d = 170mm) and span length of plates.



Assuming b_w of beam to be 250mm.

3.57-(0.17+0.25/2) = 3.275m

Figure D.8: Maximum design shear of plate – 1for the final stage (from ***ETABS***)

$$\frac{3.57m}{3.275m} = \frac{51.55kN}{Vsd}$$

$\boldsymbol{v_{sd}}$ **= 47.29kN**

b) Section Capacity

- Concrete capacity

$V_{Rd,c} = [C_{Rd,c} K (100\rho_1 f_{ck})^{1/3} + K_1 \sigma_{cp}] b_w d \geq (V_{min} + K_1 \sigma_{cp}) b_w d$

Where,

f_{ck} = 20MPa

$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with d = 170mm

$K = 1 + \sqrt{\frac{200}{170}} = 2.08 > 2.0$ take K = 2

$\rho_1 = \frac{A_{s1}}{b_w d} = \frac{11 \times 78.5}{1200 \times 170} = 0.0042 < 0.02$ take $\rho_1 = 0.0042$

$\sigma_{cp} = \frac{N_{Ed}}{A_c d} < 0.2$ fcd,   but $N_{Ed} = 0$ therefore $\sigma_{cp} = 0$

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

$K_1 = 0.15$

$V_{min} = 0.035 K^{3/2} fck^{1/2} = (0.035)(2)^{3/2} (20)^{1/2} = 0.44$

$(V_{min} + K_1 \sigma_{cp}) b_w d$ = (0.44 + 0) (1200 × 170) = 89.76kN

$V_{Rd,c}$ = [0.12×2(100 × 0.0042 × 20)$^{1/3}$ + (0.15× 0)] (1200 × 170) = **99.52kN** > 89.76kN

- Stirrup capacity (inclined)

$V_{Rd,s} = \frac{A_{sw}}{s}\ z\ f_{ywd}\ (\cot\theta + \cot\alpha)\sin\alpha$

Since the stirrup has two inclination angles, we shall classify the stirrup capacity into two (calculated in the previous section)

$V_{Rd,s} = V_{Rd,s1} + V_{Rd,s2}$

A_{sw} = 2(2 × a_s) = 113.04mm^2

Z = 0.9d = (0.9 × 170) = 153mm

f_{ywd} = 260.87MPa

$\theta = 45^0$, $\cot\theta = 1$

$\cot\alpha_1 = \cot(63.4^0) = 0.5$

$\sin\alpha_1 = 0.89$

$V_{Rd,s1}$ = [113.04 × 153 × 260.87 (1 + 0.5) × 0.89]

= 40.15kN

$\cot\alpha_2 = \cot(116.6^0) = -0.5$

$V_{Rd,s2}$ = [113.04 × 153 × 260.87 (1 - 0.5) × 0.89]

= 13.38kN

$V_{Rd,s}$ = (40.15 + 13.38)KN = **53.53kN**

- $V_{rd,max}$

$V_{rd,max} = V_{rd,max1} + V_{rd,max2}$

$V_{rd,max1} = \alpha_{cw}\, b_w\, z\, \nu_1 f_{cd} \left(\frac{\cot\theta + \cot\alpha 1}{1+cot^2\theta}\right)$

$V_{rd,max2} = \alpha_{cw} b_w z \nu_1 f_{cd} \left(\frac{\cot\theta + \cot\alpha 2}{1+cot^2\theta} \right)$

Where,

α_{cw} is 1 for non prestressed structures.

Z = 0.9 × 170 = 153mm

$\nu_1 = \nu = 0.6 [1 - \frac{f_{ck}}{250}]$ f_{ck} = 20MPa solving this $\nu_1 = 0.552$

$\theta = 45^0$, $\cot\theta = 1$

$\cot\alpha_1 = \cot(63.4^0) = 0.5$ and $\cot\alpha_2 = \cot(116.6^0) = -0.5$

$V_{rd,max1} = \frac{[1 \times 1200 \times 153 \times 0.552 \times 11.33 \times (1+0.5)]}{1+1}$ = 859kN

$V_{rd,max2} = \frac{[1 \times 1200 \times 153 \times 0.552 \times 11.33 \times (1-0.5)]}{1+1}$ = 286.3kN

$V_{rd,max}$ = (859 + 286.3)kN = **1145.3kN**

- Shear capacity of member

$$V_{Rd} = \text{Min of} \begin{cases} V_{Rd,c} + V_{Rd,s} \\ and \\ V_{rd,max} \end{cases} = \text{min of} \begin{cases} 99.52 + 53.53 = 153.05kN \\ and \\ 1145.3kN \end{cases}$$

V_{Rd} = 153.05kN > V_{sd} = 47.29kN…………safe!

c) Shear at the Interface Between Concrete Cast at Different Times

In addition to the previous shear requirements, the shear stress at the interface between concrete cast at different times should also satisfy the following:

$V_{Edi} \leq V_{Rdi}$ (EN 1992-1-1:2004: **6.23**)

$V_{Edi} = \frac{\beta V_{Ed}}{z\, b_i}$ (EN 1992-1-1:2004: **6.24**)

$V_{Rdi} = C\, fctd + \mu\, \sigma n + \rho\, f_{yd} (\mu \sin\alpha + \cos\alpha) \leq 0.5\, \nu\, f_{cd}$ (EN 1992-1-1:2004: **6.25**)

- Shear stress at the interface (V_{Edi})

The constants are previously defined in *section 3.4.2.2*.

In the flexure design of the final stage, it is learned that the neutral axis lies nearly on the top reinforcement and therefore all the tension force in the section is resisted by the compression force in the concrete which lies above the neutral axis.

For μ_{sd} = 0.105 (from flexure design), read k_x = 0.16 from the general design chart

$k_x = \frac{x}{d} = \frac{x}{170}$ = 0.16, x = 27mm ≈ 25mm depth of the top reinforcement

From equilibrium,

$c_c = f_{yt1} + f_{t2}$ where, Cc is compression force in the concrete above the neutral axis and

f_{yt1} is tension force in the yielded bottom reinforcement

f_{t2} is tension force in the top reinforcement

$c_c = A_{s1} f_{yd} + A_{s2} f_s$

c_c = (863.5×260.87) + (157 ×160) = 250.4kN

Since the concrete below the neutral axis do not contribute to tension resistance,

$$\beta = \frac{\text{longitudinal force in the new concrete area}}{\text{longitudinal force either in the compression or tension zone}} = \frac{250.4kN}{250.4kN} = 1$$

V_{Ed} = maximum design shear for the final stage = 47.29kN

z = 0.9d = 0.9 × 170 = 153mm

b_i = 1200mm

$$V_{Edi} = \frac{\beta V_{Ed}}{z\, b_i} = \frac{(1 \times 47.29 \times 10^3)N}{(153 \times 1200)\text{mm}^2} = \mathbf{0.26MPa}$$

- Design shear resistance at the interface (V_{Rdi})

According to section 6.2.5 (2) of EN 1992-1-1:2004, the roughness coefficients c and μ are taken to be 0.35 and 0.6 respectively for smooth surface or a free surface left without further treatment after vibration.

$$\rho = \frac{A_s}{A_i} = \frac{n \times a_s}{A_c} = \frac{(296 \times 28.26)\text{mm}^2}{1200mm \times 5500\text{mm}} = 0.00127$$

f_{ctd} = 1.03Mpa, f_{yd} = 260.87MPa, $\sigma_n = 0$ and $\alpha = 63.4^0$

$V_{Rdi} = C\ fctd + \mu\ \sigma n + \rho\ f_{yd}\ (\mu \sin\alpha + \cos\alpha) \leq 0.5\ \nu\ f_{cd}$

$V_{Rdi} = (0.35 \times 1.03) + (0) + (0.00127 \times 260.87\ [(0.6 \times 0.89) + 0.45]) < (0.5 \times 0.552 \times 11.3)$

$= 0.69MPa < 3.12MPa$

Take V_{Rdi} = 0.69MPa > V_{Edi} = 0.26MPa……….safe!

D.1.3. Serviceability Limit State: Deflection Control

D.1.3.1. Final Stage

For the final stage, the limiting span/depth ratio can be estimated using the provided expressions in Eurocode and multiplying it by a correction factor to allow for the type of reinforcement used and other variables.

$\frac{l}{d} = K\ [11 + 1.5\sqrt{f_{ck}}\ \frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\ (\frac{\rho_o}{\rho} - 1)^{3/2}]$ if $\rho \leq \rho_o$ (EN 1992-1-1:2004: **7.16a**)

$\frac{l}{d} = K\ [11 + 1.5\sqrt{f_{ck}}\ \frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}]$ if $\rho > \rho_o$ (EN 1992-1-1:2004: **7.16b**)

Where,

ρ_o - is the reference reinforcement ratio = $\sqrt{f_{ck}}\ 10^{-3}$

$\rho_o = \sqrt{20MPa} \times 10^{-3} = 0.00447$

ρ - is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers), it is given by $\frac{A_{s1}}{bd}$

$\rho = \frac{A_{s1}}{bd} = \frac{11 \times 78.5}{1200 \times 170} = 0.00423 < \rho_o = 0.00447$ hence use the first formula.

Summarized value of $\frac{l}{d}$ calculated from the above formula is given on table 7.4N of EN 1992-1-1:2004 based on the value of 'ρ' and position of member.

ρ = 0.00423 therefore take ρ = 0.5% (lightly reinforced)

Position of member: End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side.

Based on these data, read $\frac{l}{d}$ = 26 from table

Correction factor = $\frac{310}{\sigma_s}$ (EN 1992-1-1:2004: **7.17**)

$$= \frac{500}{\frac{f_{yk} \times A_{s,req}}{A_{s,prov}}} = \frac{500}{300 \times 1} = 1.67 \qquad \text{assuming } \frac{A_{s,req}}{A_{s,prov}} = 1$$

$(\frac{l}{d})_{limit} = 26 \times 1.67 = 43.42$

$(\frac{l}{d})_{actual} = \frac{maximum\ span\ length\ between\ supports}{effective\ depth\ of\ section} = \frac{5500}{170}$ = **32.35** < $(\frac{l}{d})_{limit}$ = 43.42……..safe!

D.2. Plate Design: Light Weight Concrete

The light weight concrete is characterized by a concrete grade of LC-25/28 with a unit weight of 16.56kN/m^3 and density of 1656kg/m^3.

- f_{lcu} = 28MPa f_{lctk} = 1.52MPa
  f_{lck} = 25MPa f_{lctd} = 1.01MPa
  f_{lcd} = 14.17MPa E = 17.7GPa

Plate sizes to be used are:

- Plate – 1: L=5.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 170mm, D = 200mm
- Plate – 2: L=4.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 170mm, D = 200mm
- Plate – 3: L=4.5m, W = 1.2m, d_{plank} = 60mm, $d_{efective}$ = 171mm, D = 200mm

D.2.1. Ultimate Limit State: Flexure Design

The design example will only cover the analysis and design of Plate – 1.

- *Use 10 Ø 10 c/c 110 (tension), 2 Ø 10 (compression) and Ø 6 c/c 150 shear reinforcement*

D.2.1.1. Production Stage (Premature Buckling of Members)

The ultimate limit state requirement of the Production stage is studied by the structural software SAP2000V14 using design preference Eurocode-3, 1993. A sample hand calculation to check the premature buckling of members is presented in the following section. Two hundred fifty eight members (stirrups, top and bottom chords) exist in this model, and in this section one specific member (stirrup) is selected from the model shown below, for a joint load of 0.06kN with three lifting points (two supports at exterior points and one at the middle) acting as a simple support for the 3D model.

P_d=1.34 $\frac{kN}{m^2}$ (from table C.1)

change to joint load

$P_d = \frac{1}{2}(1.34\frac{kN}{m^2})*(s)*(\frac{L}{2}) = \frac{1}{2}(1.34)(0.15m)(\frac{1.2m}{2}) = 0.06kN$

Label of the member:430 with D – C ratio of 0.8

Analysis output from SAP2000V14:

P = 0.515kN (compression)   and   L = 0.18371 m

End Moment : Along the axis Y – Y: -1.7×10^{-3} kNm and 4.24×10^{-4} kNm

Along the axis Z – Z: 1.3×10^{-3} kNm and -2.39×10^{-4} kNm

The buckling resistance of compression members is:

$$\frac{N_{sd}}{\chi_{min}\frac{Af_y}{\gamma_{m1}}} + \frac{K_y M_{y,sd}}{W_{pl,y}\frac{f_y}{\gamma_{m1}}} + \frac{K_z M_{z,sd}}{W_{pl,z}\frac{f_y}{\gamma_{m1}}} \leq 1$$ (Sec. 5.5.4, Eurocode-3, 1993)

$\lambda_1 = 93.9\,(\frac{235}{f_y})^{0.5} = 93.9\,(\frac{235}{300})^{0.5} = 83.11$

$i = \frac{r}{2} = \frac{3}{2} = 1.5mm$ and l = 0.18371

According to Eurocode-3, 1993, for members resisting loads other than wind load, $\lambda \leq 180$

$\lambda_y = \lambda_z = \frac{l}{i} = \frac{183.71mm}{1.5mm} = 122.47$, $\beta_A = 1$ for Class 1

$\overline{\lambda_y} = \overline{\lambda_z} = \frac{\lambda_y}{\lambda_1}\sqrt{\beta_A} = = \frac{122.47}{83.11}\sqrt{1} = 1.47$

$\alpha = 0.49$,   (table 5.5.3 and table 5.5.1, Eurocode-3, 1993)

For buckling curve c, $\bar{\lambda} = 1.47$ and using interpolation

$\chi_y = \chi_z = 0.3257$ (table 5.5.2, Eurocode-3, 1993)

$\chi_{min} = 0.3257$

$\psi_y = \frac{4.24\times10^{-4}}{-1.7\times10^{-3}} = -0.249$

$\beta_{M,Y} = 1.8 - 0.7\,\psi = 1.8 - (0.7 \times -0.249) = 1.974$

$\psi_z = \frac{-2.39\times10^{-4}}{1.3\times10^{-3}}$ = -0.184

$\beta_{M,z}$ = 1.8 - 0.7 ψ = 1.8 - (0.7 × -0.184) = 1.929

$w_{pl,y} = w_{pl,z}$ = Z = $\frac{d^3}{6} = \frac{6^3}{6}$ = 36 mm^3

$w_{el,y} = w_{el,z}$ = s = $\frac{\pi d^3}{32} = \frac{\pi 6^3}{6}$ = 21.2 mm^3

$\mu_y = \overline{\lambda_y}[2\beta_{M,Y} - 4] + [\frac{w_{pl,y}-w_{el,y}}{w_{el,y}}] \leq 0.9$

μ_y = 1.47×[2×1.974 - 4] + $[\frac{36-21.2}{21.2}]$ = 0.622 ≤ 0.9

$\mu_z = \overline{\lambda_z}[2\beta_{M,z} - 4] + [\frac{w_{pl,z}-w_{el,z}}{w_{el,z}}] \leq 0.9$

μ_z = 1.47×[2×1.929 - 4] + $[\frac{36-21.2}{21.2}]$ = 0.489 ≤ 0.9

$k_y = 1- \frac{\mu_y\times N_{sd}}{\chi_y\times A\times f_y} \leq 1.5$

$k_y = 1- \frac{0.622\times0.515\times10^3}{0.3257\times28.27\times10^{-6}\times260.87\times10^6}$ = 0.867

$k_z = 1- \frac{\mu_z\times N_{sd}}{\chi_z\times A\times f_y} \leq 1.5$

$k_z = 1- \frac{0.489\times0.515\times10^3}{0.3257\times28.27\times10^{-6}\times260.87\times10^6}$ = 0.895

Demand to capacity ratio

$$\frac{N_{sd}}{\chi_{\min(\frac{A\times f_y}{\gamma_{m1}})}} + \frac{K_y M_y{,}sd}{W_{\text{pl,y}(\frac{f_y}{\gamma_{m1}})}} + \frac{K_z M_z{,}sd}{W_{\text{pl,z}(\frac{f_y}{\gamma_{m1}})}} \leq 1$$

$$\frac{1.1\times0.515\times10^3}{0.3257\times28.27\times10^{-6}\times260.87\times10^6} + \frac{1.1\times0.867\times0.0017\times10^3}{3.6\times10^{-8}\times260.87\times10^6} + \frac{1.1\times0.895\times0.0013\times10^3}{3.6\times10^{-8}\times260.87\times10^6} = 0.415$$

SAP result = 0.417

ratio = $\frac{0.415}{0.417}$ = 99%, difference is less than 5%.

D.2.1.2. Construction Stage

a) Design Moment

The load for the construction stage from Table C.1 (5.97kN/m^2) should be multiplied by the width of the plate (1.2m) to change the loading from areal load to line load.

The analysis of the slab strip is executed on ETABS v 9.7.4 and the maximum internal moment is found to be ***2.26kNm***.

b) Section Capacity

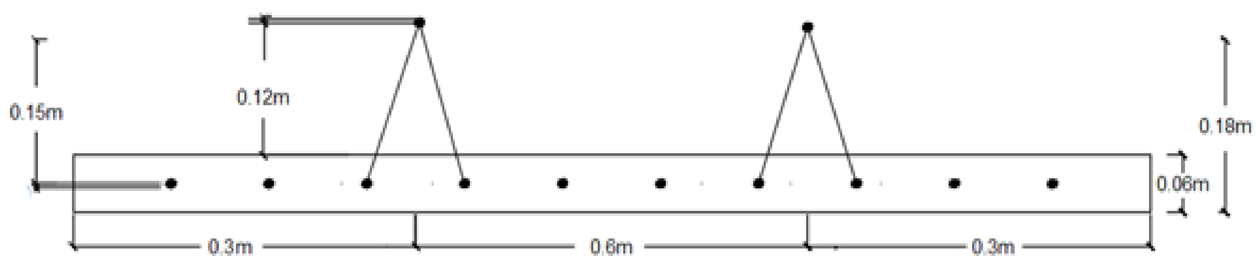


Figure D.9: Section view of plate-1 for the construction stage (Light Weight)

The following assumptions are made in order to estimate the section capacity of the plate in the production and construction stage.

- ✓ The neutral axis lies in the concrete area.
- ✓ All reinforcement bars have yielded.
- ✓ The concrete has crushed.
- ✓ The section is under balanced failure.

$d_p = D_p - C_c - \emptyset_{sr} - \emptyset_r/2$

Where, d_p - is effective depth of plank.

D_p - is total depth of plank

Cc - Concrete cover, 15mm

$\emptyset_{sr}$ - diameter of shear reinforcement, 10mm

$\emptyset_r$ - diameter of longitudinal reinforcement, 10mm

d_p = (60 – 15 – 10 – 5) mm = 30mm

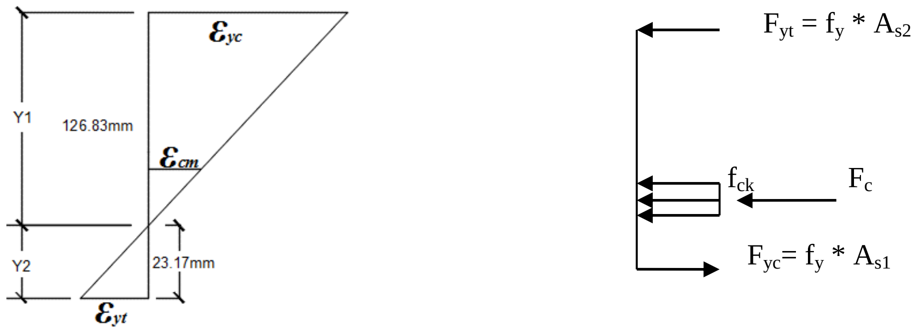


Figure D.10: Stress and Strain diagram (Light Weight)

- From equilibrium equation

$F_{yc}+F_c=F_{yt}$,

Where $F_{yc} = 260.87\frac{N}{mm^2}\times\frac{2\times\pi\times 10^2}{4}$, $F_{yt} = 260.87\frac{N}{mm^2}\times\frac{10\times\pi\times 10^2}{4}$

$= 41kN$ $= 204.78kN$

$$F_c = F_{yt} - F_{yc} = 204.78kN - 41kN = 163.82kN$$

And from the section capacity of the concrete

$$F_c = (0.8x)\times f_{ck}\times b_w \Rightarrow \quad x = \frac{F_c}{0.8\times f_{ck}\times b_w} = \frac{163.82\times 10^3 kN}{0.8\times 25\frac{N}{mm^2}\times 1200mm} = 6.83mm$$

- Check whether the steel has yielded and the N.A. lies in the concrete.

From similarity of triangles,

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{y_1}{y_2} = \frac{120+6.83}{23.17} = 5.47$$

$$\Rightarrow \frac{\varepsilon_{yc}}{5.47} = \varepsilon_{yt}$$

$\Rightarrow$Therefore the top and bottom reinforcements would not reach the yield point simultaneously.

$$\varepsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{260.87MPa}{200GPa} = 1.3\ {}^{0}/_{00}$$

- Determine the exact strain in the concrete

$$\frac{\varepsilon_{yc}}{\varepsilon_{cm}} = \frac{y_1}{y_1 - 120} = \frac{120+6.83}{6.83} = 18.6$$

$$\varepsilon_{cm} = \frac{\varepsilon_{yc}}{18.6} = \frac{1.3‰}{18.6} = 0.07‰ < 3.5\ ‰$$

Therefore if there is no premature buckling of bars the top reinforcement would reach the yield point first and next determine the exact strains conditions.

From above,

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{1.3‰}{\varepsilon_{yt}} = \frac{126.83}{23.17}$$

$$\varepsilon_{yt} = 0.238‰$$

$$f_{st} = \varepsilon_{yt} \times E_s = \frac{0.238}{1000} \times 200\ \text{GPa} = 47.6\ \text{MPa}$$

$$F_{yT} = 47.6 \frac{N}{mm2} \times \frac{10 \times \pi \times 10^2}{4} = 37.4\text{kN}$$

From similarity of triangles

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{a}{160-a} \Rightarrow \frac{1.3‰}{0.238‰} = 5.46 = \frac{a}{160-a}$$ Solving this

$$a = 135.2\ \text{mm}$$

$x = a - 120 = 15.2$mm (Neutral axis depth within the concrete)

And from equilibrium equation

$$F_{yc} + F_c = F_{yt} \qquad \text{Where} \quad F_{yc} = 260.87 \frac{N}{mm^2} \times \frac{2\times \pi \times 10^2}{4} \quad , \quad F_{yt} = 37.4\text{kN}$$

$$= 41\text{kN}$$

$F_c = (37.4 - 41)$ kN = -4.7kN $\Rightarrow$ the concrete section must resist this tension force (not possible!)

$\Rightarrow$ N.A. will not be in the concrete section.

Assumption: the N.A. lies in between the concrete and the top reinforcement.

$$F_{yc} = F_{yt} \qquad \Rightarrow \quad 260.87 \frac{N}{mm^2} \times \frac{2 \times \pi \times 10^2}{4} = f_{yt} \times \frac{10 \times \pi \times 10^2}{4}$$

$$f_{yt} = 52.8\frac{N}{mm^2} = 52.8MPa$$

$$\Rightarrow F_{yt} = 52.8\frac{N}{mm^2} \times \frac{10\times\pi\times10^2}{4} = 40.1kN$$

From linear stress and strain relationship

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{52.8MPa}{200GPa} = 0.264^0/_{00} < 1.3^0/_{00}$$

$$\frac{\varepsilon_{yc}}{\varepsilon_{yt}} = \frac{a}{160-a} \Rightarrow \frac{1.3‰}{0.264‰} = 4.9 = \frac{a}{160-a} \text{ Solving this}$$

a = 132 mm (negligible concrete area under tension)

From the above conditions the flexural capacity of concrete would be

$M_{rd} = F_{yc}$ x 0.15m = 41kN x 0.15m

= 6.15 kNm > 2.26kNm ...... Safe!

D.2.1.3. Final Stage

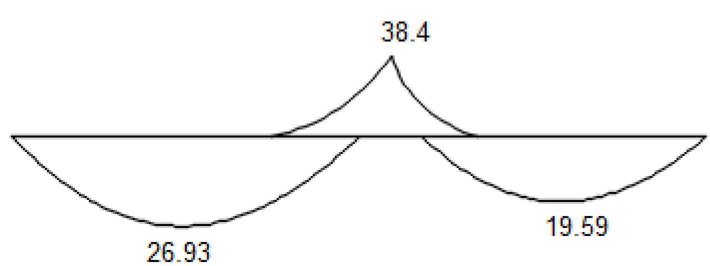
a) Design Moment

As per Table C.1, the design load (P_d) for the final stage is 10.37kN/m^2. When loading the slab strip, the design areal load should be changed into line load by multiplying it with the width of the strip.

Pattern loading for the final stage shall first be performed and the moment envelope be developed. The slab strip having the same width as the individual plates is modeled on ETABS 9.7.4 in order to develop the moment envelope.

- **Moment Envelope**

PART – A



PART – B

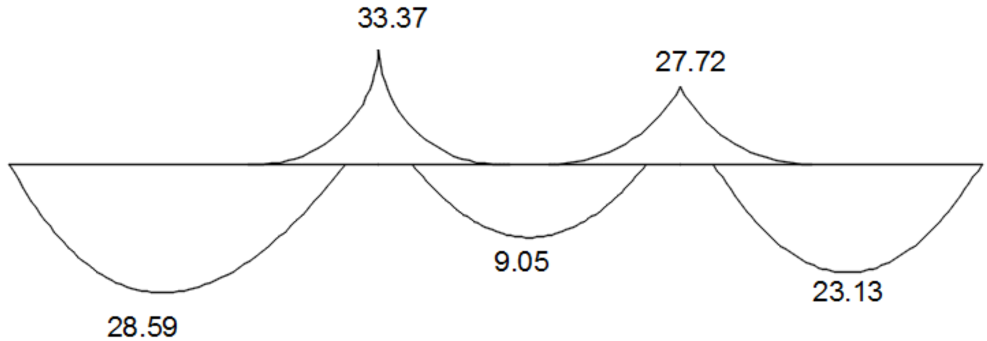


Figure D.11: Moment envelope developed on ETABS for both Part A and Part-B (Light Weight)

From the moment envelope, there are three groups of *span moment* values:

1. M_{sd1} = 28.59kNm, M_{sd2} = 26.93kNm (Plate - 1)
2. M_{sd1} = 23.13kNm, M_{sd3} = 19.59kNm (Plate - 2)
3. M_{sd1}= 9.05kNm (Plate - 3)

Hence the design moment of plate-1 for the final stage is **28.59kNm.**

b) Section Capacity

The amount of longitudinal reinforcement is first assigned and then the section capacity of the plate is calculated. But before that, the minimum reinforcement area has to be determined so that the area of reinforcement provided should be greater than the minimum reinforcement area.

According to EN 1992-1-1:2004, Section 9.2.1.1, the area of longitudinal tension reinforcement should not be taken as less than $A_{s,min}$ which is defined as follows:

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d > 0.0013 b_t d$$

Where,

b_t - denotes the mean width of the tension zone,

f_{lctm} - should be determined with respect to the relevant strength class according to Table 11.3.1 of

EN 1992-1-1:2004, for LC-25/28, $f_{lctm} = f_{ctm} \times \eta_1$ = 2.6MPa × 0.85 = 2.21MPa

f_{yk} and d – defined previously

$A_{smin} = \frac{0.26 \times 2.21MPa \times 1200mm \times 170mm}{300MPa}$ = <u>391mm²</u> > $0.0013 b_t d = 0.0013 \times 1200mm \times 170mm = 265mm^2$

Therefore the area of main reinforcement assigned should be equal or greater than $A_{s,min} = 391mm^2$.

- Use 10 Ø 10 c/c 120 (tension), 2 Ø 10 (compression), Msd = 28.59kNm (take maximum)

  $d = D - C_c - \emptyset_{sr} - \emptyset_r/2 = (200 - 10 - 15 - 5)$ mm = 170mm, d_2 = 20mm,

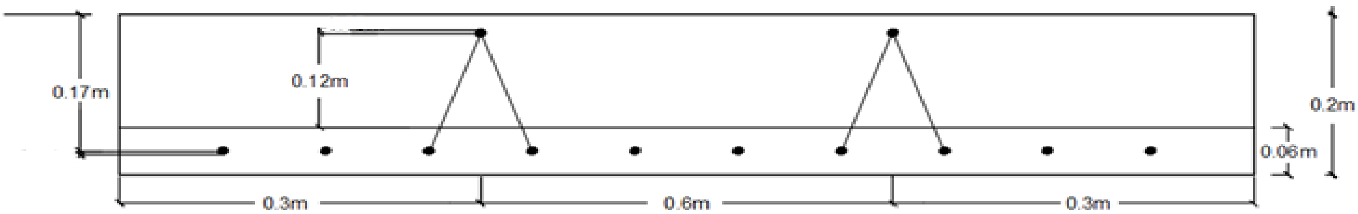


Figure D.12: Section view of plate-1 for the final stage (Light Weight)

- .A_{s1} (tension) = n ($\frac{\pi \times d^2}{4}$), where n is number of bars and d is diameter of bar.

  $A_{s1} = 10 (\frac{\pi \times 10^2}{4}) = 785mm^2$.

- A_{s2} (Compression) = $2(\frac{\pi \times 10^2}{4}) = 157mm^2$

The plate is considered as *a doubly reinforced section* and the following assumptions are used in order to estimate the section capacity of the plate.

- ✓ The neutral axis is within section.
- ✓ All reinforcements have yielded.

$\varepsilon_{yd} = \frac{f_{yd}}{E_s}$ where ε_{yd} is yield strain in the reinforcement.

$\varepsilon_{yd} = \frac{260.87MPa}{200GPa} = 1.3\ ^0/_{00}$

$\alpha_c = \frac{(A_{s1} - A_{s2}) f_{yd}}{f_{cd}\, bd}$ where α_c is a constant used in the calculation of moment and the other terms are defined previously.

$\alpha_c = \frac{(785-157)260.87}{14.17 \times 1200 \times 170} = 0.057$

εs_1 and εs_2 which are defined as strain in the top and bottom reinforcement respectively are then read from the general design chart.

$\varepsilon s_1 = 10\,{}^0/_{00}$ and $\varepsilon s_2 = -0.2{}^0/_{00} < \varepsilon yd$…………assumption not ok!

Trial – 1

$f_s = \varepsilon_s E$ where f_s is stress and ε_s strain in the reinforcement bar.

$f_s = -0.2 \times 10^{-3} \times 200GPa = -40MPa$

$$\alpha_c = \frac{(A_{s1} f_{yd} - A_{s2} f_s)}{f_{cd}\, bd} = \frac{(785*260.87) + (157*40)}{14.17 * 1200 * 170} = 0.073$$

Read $\varepsilon s_1 = 10\,{}^0/_{00}$ and $\varepsilon s_2 = -0.35{}^0/_{00}$

Trial – 2

$f_s = \varepsilon_s E = -0.35 \times 10^{-3} \times 200GPa = -70MPa$

$$\alpha_c = \frac{(A_{s1} f_{yd} - A_{s2} f_s)}{f_{cd}\, bd} = \frac{(785 \times 260.87) + (157 \times 70)}{14.17 \times 1200 \times 170} = 0.075$$

Read $\varepsilon s_1 = 10\,{}^0/_{00}$ and $\varepsilon s_2 = -0.35\,{}^0/_{00}$ (the iteration converges)

Read $\mu_{sd} = 0.073$ (from the general design chart)

Section Capacity

$M_{rd} = \mu_{sd}\, f_{cd}\, bd^2 + A_{s2}\, f_s\, (d-d_2)$

$M_{rd} = [0.073 \times 14.17MPa \times 1200mm \times (170mm)^2] + [157mm^2 \times (-70MPa) \times 150mm]$

$M_{rd} = 34.2kNm > M_{sd} = 28.59kNm$ ……safe!

c) Design of Secondary Reinforcement

Plate – 1

According to EN 1992-1-1:2004, section 9.3.1,

- minimum reinforcement for secondary reinforcement is given by

$A_{s,secondary} = 0.2 \times A_{s,main}$

$A_{s,main}$ = (785 + 157)mm^2= 942mm^2

$A_{s,secondary} = (0.2 \times 942)mm^2$ = **$188.4mm^2$**

- maximum spacing limit for secondary reinforcement is given by:

$$S_{max} = \text{mini of} \begin{cases} 3.5h = 3.5 \times 200 = 700mm \\ and \\ 450mm \end{cases}$$ take S_{max} = **450mm**

Use Ø 6 bars, a_s = 28.26mm^2

S = $\frac{b \times a_s}{A_s} = \frac{5500mm \times 28.26mm^2}{188.4mm^2}$ = 825mm > S_{max} = 450mm……not ok!

Calculate the amount of reinforcement to be provided using maximum spacing limit.

$A_s = \frac{b \times a_s}{s} = \frac{5500mm \times 28.26mm^2}{450mm}$ = 345.4mm^2 > $A_{s,min}$ = 188.4mm^2………ok!

$No. of\ bars = \frac{A_s}{a_s} = \frac{345.4mm^2}{28.26mm^2}$ ≈ 13Ø 6 bars

d) Design of Negative Reinforcement

The design negative moment developed according to the moment envelope diagram (figure D.11) is ***38.4KNm*** for PART-A (joins Plate – 1 and Plate - 2).

Check section capacity (singly or doubly reinforced)

$$\mu_{sd} = \frac{M_{sd}}{f_{cd}bd^2} \Rightarrow \mu_{sd} = \frac{38.4 \times 10^6}{14.17 \times 1200 \times (170^2)} = 0.08 < \mu_{sd}^* = 0.295....\text{SRS (singly reinforced section)}$$

Read k_z = 0.95 from the general design chart for μ_{sd} = 0.08 and calculate area of reinforcement to be provided.

$$A_{s,tot} = \frac{M_{sd}}{z\, f_{yd}} = \frac{M_{sd}}{Kz\, d\, f_{yd}} = \frac{38.4 \times 10^6}{0.95 \times 170 \times 260.87} = 911.45mm^2$$

But the plates have two compression chords of 'Ø 10' which serve as tension chord for the negative moment. Therefore,

A_s = [911.45 – (2 × 78.5)] mm^2 = 754.45mm^2

$No. of\ bars = \frac{A_s}{a_s} = \frac{754.45mm^2}{78.5mm^2}$ ≈ 10Ø 10 bars in addition to the 2 Ø 10 chords.

D.2.2. Ultimate Limit State: Shear Design

D.2.2.1. Construction Stage

a. Design Shear

As discussed earlier, for the analysis of slab in the construction stage, the one way rectangular plates are modeled as continuous beams having a depth of 0.06m and a width of 1.2m.

Plate - 1 has a length of 5.5m and hence it is temporarily supported at two different points in the construction stage. Once the slab is analyzed, maximum shear values are used for manual design calculations as per EN 1992-1-1:2004 which is summarized as follows.

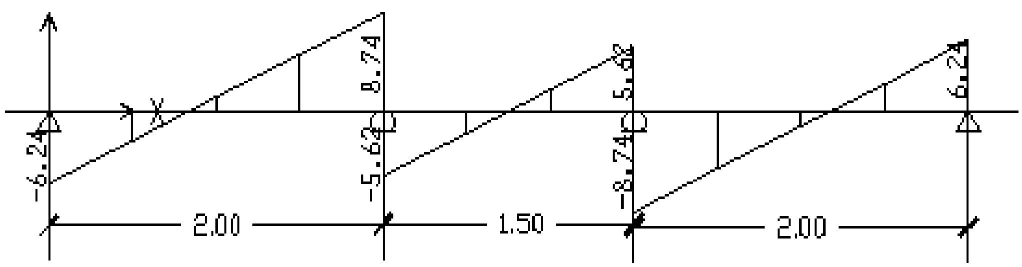


Figure D.13: Shear diagram of Plate – 1 for the construction stage (Light Weight)

The maximum shear force from the shear diagram is ***8.74kN***. Taking "d" distance from the face of the support is almost equal with 8.74kN.

b. Section Capacity

Use ∅ 6 c/c 150 shear reinforcement

- Design constants

f_{cd} = 14.17MPa, f_{ctd} = 1.01MPa, f_{yd} = 260.87MPa

- Concrete capacity

$V_{Rd,c} = [C_{Rd,c} K (100\rho_1 f_{ck})^{1/3} + K_1 \sigma_{cp}] b_w d \geq (V_{min} + K_1 \sigma_{cp}) b_w d$

Where,

f_{ck} = 25MPa

$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with d = 30mm

$K = 1 + \sqrt{\frac{200}{30}} = 3.58 > 2.0$ take K = 2

$\rho_1 = \frac{A_{s1}}{b_w d} = \frac{10\times78.5}{1200\times30} = 0.0218 > 0.02$ take $\rho_1 = 0.02$

$\sigma_{cp} = \frac{N_{Ed}}{A_c d} < 0.2$ fcd, but $N_{Ed} = 0$ therefore $\sigma_{cp} = 0$

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

$K_1 = 0.15$

$V_{min} = 0.035\, K^{3/2}\, fck^{1/2} = (0.035)(2)^{3/2}\,(25)^{1/2} = 0.495$

$(V_{min} + K_1\, \sigma_{cp})\, b_w d = (0.495 + 0)\,(1200 \times 30) = 17.82kN$

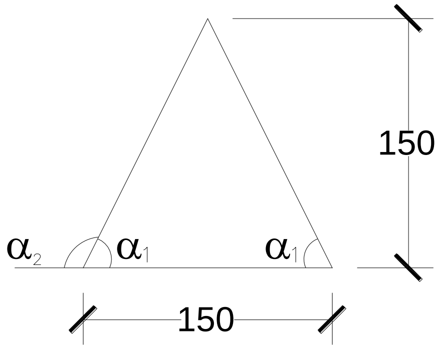
$V_{Rd,c} = [0.12\times2(100 \times 0.02 \times 25)^{1/3} + (0.15\times 0)]\,(1200 \times 30) =$ **31.83kN** >17.82kN

- Stirrup capacity (inclined)

$V_{Rd,s} = \frac{A_{sw}}{s}\, z\, f_{ywd}\, (\cot\theta + \cot\alpha) \sin\alpha$

Since the stirrup has two inclination angles, we shall classify the stirrup capacity into two.

$V_{Rd,s} = V_{Rd,s1} + V_{Rd,s2}$



$\tan\alpha_1 = \frac{150}{75} = 2$

$\alpha_1 = 63.4^0$

$\alpha_2 = 180 - \alpha_1 = 116.6^0$

Figure D.14: Stirrup arrangement (Light Weight)

$V_{Rd,s1} = \frac{A_{sw}}{s}\, z\, f_{ywd}\, (\cot\theta + \cot\alpha_1) \sin\alpha_1$

A_{sw} = 2(2 × a_s) = 113.04mm^2

Z = 0.9d = (0.9 × 30) = 27mm

f_{ywd} = 260.87MPa

$\theta = 45^0$, cot θ = 1

cot α_1= cot (63.4^0) = 0.5

sin α_1= 0.89

$V_{Rd,s1}$ = [113.04 × 27 × 260.87 (1 + 0.5) × 0.89]

= 7.09kN

cot α_2= cot (116.6^0) = -0.5

$V_{Rd,s2}$ = [113.04 × 27 × 260.87 (1 - 0.5) × 0.89]

= 2.36kN

$V_{Rd,s}$ = (7.09 + 2.36)KN = **9.45kN**

- $V_{rd,max}$

$V_{rd,max} = V_{rd,max1} + V_{rd,max2}$

$V_{rd,max1} = \alpha_{cw} b_w z \nu_1 f_{cd} \left(\frac{\cot\theta + \cot\alpha 1}{1+cot^2\theta} \right)$

$V_{rd,max2} = \alpha_{cw} b_w z \nu_1 f_{cd} \left(\frac{\cot\theta + \cot\alpha 2}{1+cot^2\theta} \right)$

Where,

α_{cw} is 1 for non prestressed structures.

Z = 0.9d = 27mm

$\nu_1 = \nu = 0.6 [1 - \frac{f_{ck}}{250}]$ f_{ck} = 25MPa solving this ν_1 = 0.54

$\theta = 45^0$, cot θ = 1

cot α_1= cot (63.4^0) = 0.5 and cot α_2= cot (116.6^0) = -0.5

$V_{rd,max1} = \frac{[1 \times 1200 \times 27 \times 0.54 \times 14.17 \times (1+0.5)]}{1+1} = 185.94\text{kN}$

$V_{rd,max2} = \frac{[1 \times 1200 \times 27 \times 0.54 \times 14.17 \times (1-0.5)]}{1+1} = 61.98\text{kN}$

$V_{rd,max}$ = (185.94 + 61.98)KN = **247.92kN**

- Shear capacity of member

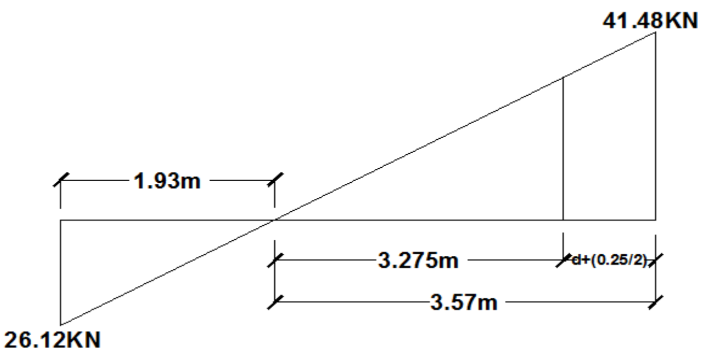
$$V_{Rd} = \text{Min of} \begin{cases} V_{Rd,c} + V_{Rd,s} \\ and \\ V_{rd,max} \end{cases} = \text{min of} \begin{cases} 31.83 + 9.45 = 41.28kN \\ and \\ 247.92kN \end{cases}$$

V_{Rd} = 41.28kN > V_{sd} = 8.74kN………..safe!

D.2.2.2. Final Stage

a) Design Shear

Shear design of this stage follows the procedure used in the construction stage, the only difference being the depth of section considered (d = 170mm) and span length of plates.



Assuming b_w of beam to be 250mm.

3.57-(0.17+0.25/2) = 3.275m

Figure D.15: Maximum design shear of plate – 1 for the final stage (Light Weight)

$$\frac{3.57m}{3.275m} = \frac{41.48\ kN}{Vsd}$$

$\boldsymbol{v_{sd}}$ **= 38.1kN**

b) Section Capacity

- Concrete capacity

$V_{Rd,c} = [C_{Rd,c} K (100\rho_1 f_{ck})^{1/3} + K_1 \sigma_{cp}] b_w d \geq (V_{min} + K_1 \sigma_{cp}) b_w d$

Where,

f_{ck} = 25MPa

$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with d = 170mm

$K = 1 + \sqrt{\frac{200}{170}} = 2.08 > 2.0$ take K = 2

$\rho_1 = \frac{A_{s1}}{b_w d} = \frac{10\times78.5}{1200\times170} = 0.00385 < 0.02$ take $\rho_1 = 0.00385$

$\sigma_{cp} = \frac{N_{Ed}}{A_c d} < 0.2$ fcd, but $N_{Ed} = 0$ therefore $\sigma_{cp} = 0$

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

$K_1 = 0.15$

$V_{min} = 0.035\, K^{3/2}\, fck^{1/2} = (0.035)(2)^{3/2}\,(25)^{1/2} = 0.495$

$(V_{min} + K_1\, \sigma_{cp})\, b_w d = (0.495 + 0)\,(1200 \times 170) = 100.98$kN

$V_{Rd,c} = [0.12\times2(100 \times 0.00385 \times 25)^{1/3} + (0.15\times 0)]\,(1200 \times 170) =$ **104.14kN** > 100.98kN

- Stirrup capacity (inclined)

  $V_{Rd,s} = \frac{A_{sw}}{s}\, z\, f_{ywd}\,(\cot\theta + \cot\alpha)\sin\alpha$

Since the stirrup has two inclination angles, we shall classify the stirrup capacity into two (calculated in the previous section)

$V_{Rd,s} = V_{Rd,s1} + V_{Rd,s2}$

$A_{sw} = 2(2 \times a_s) = 113.04\text{mm}^2$

$Z = 0.9d = (0.9 \times 170) = 153$mm

f_{ywd} = 260.87MPa

$\theta = 45^0$, $\cot\theta = 1$

$\cot\alpha_1 = \cot(63.4^0) = 0.5$

$\sin \alpha_1 = 0.89$

$V_{Rd,s1}$ = [113.04 × 153 × 260.87 (1 + 0.5) × 0.89]

= 40.15kN

$\cot \alpha_2 = \cot (116.6^0) = -0.5$

$V_{Rd,s2}$ = [113.04 × 153 × 260.87 (1 - 0.5) × 0.89]

= 13.38kN

$V_{Rd,s}$ = (40.15 + 13.38)KN = **53.53kN**

- $V_{rd,max}$

$$V_{rd,max} = V_{rd,max1} + V_{rd,max2}$$

$$V_{rd,max1} = \alpha_{cw} b_w z \nu_1 f_{cd} \left(\frac{\cot\theta + \cot \alpha 1}{1+cot^2\theta} \right)$$

$$V_{rd,max2} = \alpha_{cw} b_w z \nu_1 f_{cd} \left(\frac{\cot\theta + \cot \alpha 2}{1+cot^2\theta} \right)$$

Where,

α_{cw} is 1 for non prestressed structures.

Z = 0.9 × 170 = 153mm

$\nu_1 = \nu = 0.6 [1 - \frac{f_{ck}}{250}]$ f_{ck} = 25MPa solving this $\nu_1 = 0.54$

$\theta = 45^0$, $\cot \theta = 1$

$\cot \alpha_1 = \cot (63.4^0) = 0.5$ and $\cot \alpha_2 = \cot (116.6^0) = -0.5$

$$V_{rd,max1} = \frac{[1 \times 1200 \times 153 \times 0.54 \times 14.17 \times (1+0.5)]}{1 + 1} = 1053.6\text{kN}$$

$$V_{rd,max2} = \frac{[1 \times 1200 \times 153 \times 0.54 \times 14.17 \times (1-0.5)]}{1 + 1} = 351.22\text{kN}$$

$V_{rd,max}$ = (1053.6 + 351.22)KN = **1404.82kN**

- Shear capacity of member

$$V_{Rd} = \text{Min of} \begin{cases} V_{Rd,c} + V_{Rd,s} \\ and \\ V_{rd,max} \end{cases} = \text{min of} \begin{cases} 104.1 + 53.53 = 157.63kN \\ and \\ 1404.82kN \end{cases}$$

V_{Rd} = 157.63kN > V_{sd} = 38.1kN…………safe!

c) Shear at the Interface Between Concrete Cast at Different Times

In addition to the previous shear requirements, the shear stress at the interface between concrete cast at different times should also satisfy the following:

$V_{Edi} \le V_{Rdi}$ (EN 1992-1-1:2004: **6.23**)

$V_{Edi} = \frac{\beta V_{Ed}}{z\, b_i}$ (EN 1992-1-1:2004: **6.24**)

$V_{Rdi} = C\, f_{ctd} + \mu\, \sigma_n + \rho\, f_{yd}\, (\mu \sin\alpha + \cos\alpha) \le 0.5\, \nu\, f_{cd}$ (EN 1992-1-1:2004: **6.25**)

- Shear stress at the interface (V_{Edi})

The constants are previously defined in *section 3.4.2.2.*

In the flexure design of the final stage, it is learned that the neutral axis lies nearly on the top reinforcement and therefore all the tension force in the section is resisted by the compression force in the concrete which lies above the neutral axis.

For μ_{sd} = 0.073 (from flexure design), read k_x = 0.13 from the general design chart

$k_x = \frac{x}{d} = \frac{x}{170}$ = 0.13, x = 22.1mm ≈ 25mm depth of the top reinforcement

From equilibrium,

$c_c = f_{yt1} + f_{t2}$ where, Cc is compression force in the concrete above the neutral axis and

f_{yt1} is tension force in the yielded bottom reinforcement

f_{t2} is tension force in the top reinforcement

$c_c = A_{s1} f_{yd} + A_{s2} f_s$

c_c = (785×260.87) + (157 ×70) = 215.77kN

Since the concrete below the neutral axis do not contribute to tension resistance,

$\beta = \frac{\text{longitudinal force in the new concrete area}}{\text{longitudinal force either in the compression or tension zone}} = \frac{215.77kN}{215.77kN} = 1$

V_{Ed} = maximum design shear for the final stage = 38.1kN

z = 0.9d = 0.9 × 170 = 153mm

b_i = 1200mm

$V_{Edi} = \frac{\beta V_{Ed}}{z\, b_i} = \frac{(1\times 38.1\times 10^3)N}{(153\times 1200)\text{mm}^2}$ = **0.208MPa**

- Design shear resistance at the interface (V_{Rdi})

According to section 6.2.5 (2) of EN 1992-1-1:2004, the roughness coefficients c and μ are taken to be 0.35 and 0.6 respectively for smooth surface or a free surface left without further treatment after vibration.

$\rho = \frac{A_s}{A_i} = \frac{n\times a_s}{A_c} = \frac{(296\times\ 28.26)\text{mm}^2}{1200mm\times 5500\text{mm}} = 0.00127$

f_{ctd} = 1.01Mpa, f_{yd} = 260.87MPa, σ_n = 0 and α = 63.4^0

V_{Rdi} = C fctd + μ σn + ρ f_{yd} (μ sin α + cos α) ≤ 0.5 ν f_{cd}

V_{Rdi} = (0.35 × 1.01) + (0) + (0.00127×260.87 [(0.6×0.89) + 0.45]) < (0.5×0.54×14.17)

= 0.68MPa < 3.83MPa

Take V_{Rdi} = 0.68MPa > V_{Edi} = 0.208MPa……….safe!

D.2.3. Serviceability Limit State: Deflection Control

D.2.3.1. Final Stage

For the final stage, the limiting span/depth ratio can be estimated using the provided expressions in Eurocode and multiplying it by a correction factor to allow for the type of reinforcement used and other variables.

$\frac{l}{d} = K\,[11 + 1.5\sqrt{f_{ck}}\ \frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\ (\frac{\rho_o}{\rho} - 1)^{3/2}]$ if $\rho \le \rho_o$ (EN 1992-1-1:2004: **7.16a**)

$\frac{l}{d} = K\,[11 + 1.5\sqrt{f_{ck}}\ \frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\ \sqrt{\frac{\rho'}{\rho_o}}]$ if $\rho > \rho_o$ (EN 1992-1-1:2004: **7.16b**)

Where,

ρ_o - is the reference reinforcement ratio = $\sqrt{f_{ck}}\ 10^{-3}$

$\rho_o = \sqrt{25MPa} \times 10^{-3} = 0.005$

ρ - is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers), it is given by $\frac{A_{s1}}{bd}$

$\rho = \frac{A_{s1}}{bd} = \frac{10\times78.5}{1200\times170} = 0.00385 < \rho_o = 0.005$ hence use the first formula.

Summarized value of $\frac{l}{d}$ calculated from the above formula is given on table 7.4N of EN 1992-1-1:2004 based on the value of 'ρ' and position of member.

ρ = 0.00385 therefore take ρ = 0.5% (lightly reinforced)

Position of member: End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side.

Based on these data, read $\frac{l}{d}$ = 26 from table

Correction factor = $\frac{310}{\sigma_s}$ (EN 1992-1-1:2004: **7.17**)

$= \frac{500}{\frac{f_{yk} \times A_{s,req}}{A_{s,prov}}} = \frac{500}{300\times1} = 1.67$ assuming $\frac{A_{s,req}}{A_{s,prov}} = 1$

$(\frac{l}{d})_{limit} = 26 \times 1.67 = 43.42$

$(\frac{l}{d})_{actual} = \frac{maximum\ span\ length\ between\ supports}{effective\ depth\ of\ section} = \frac{5500}{170}$ = **32.35** < $(\frac{l}{d})_{limit}$ = 43.42……..safe!

APPENDIX E: Cost Estimation

E.1. Concrete Takeoff Sheet

No	LxWxH	Product	Description	No	LxWxH	Product	Description
			1. Ribbed Slab				**3. Precast lattice slab**
			a. Beams /C-25/				(light weight concrete)
							a. Plates
1	107.50			37	5.50		Plate - 1
	0.25				1.20		
	0.50	13.44			0.20	48.84	
1	35.00			20	4.50		Plate - 2
	0.25				1.20		
	0.60	5.25			0.20	21.60	
		18.69	**m3 Total sum for Item No. 1-a**	8	4.50		Plate - 3
					1.20		
			b. rib slab		0.20	8.64	
1	35.00					**79.08**	**m3 Total sum for Item No. 3-a**
	10.00	350.00					**b. beams**
1	11.00			1	142.50		
9	4.50	49.50			0.25		
		399.50	**m2 Total sum for Item No. 1-b**		0.50	17.81	
			2. Precast lattice slab			**175.97**	**m3 Total sum for Item No. 3-b**
			(normal weight concrete)				
			a. Plates				
37	5.50		Plate - 1				
	1.20						
	0.20	48.84					
20	4.50		Plate - 2				
	1.20						
	0.20	21.60					
8	4.50		Plate - 3				
	1.20						
	0.20	8.64					
		79.08	**m3 Total sum for Item No. 2-a**				
			b. beams				
1	107.50						
	0.25						
	0.50	13.44					
1	35.00						
	0.25						
	0.60	5.25					
		18.69	**m3 Total sum for Item No. 2-b**				

Table E.1: Concrete Takeoff Sheet

E.2.Reinforcement Takeoff Sheet

Member	Description	Diam. (mm)	Bar Length	No. of Bar	No. Member	Total No. Bar	Total Length of Bar (M) 6	8	10	12	14	16	20	24	Ø32mm
	Ribbed Slab														
Mesh	Mesh	6	1,000.00	117.00	1.00	117.00	1,170.00								
		6	3,500.00	34.00	1.00	34.00	1,190.00								
		6	450.00	37.00	1.00	37.00	166.50								
		6	1,100.00	15.00	1.00	15.00	165.00								
Ribs	PB-1	14	600.00	1.00	59.00	59.00					354.00				
		16	560.00	2.00	59.00	118.00						660.80			
	PB-2	12	600.00	1.00	59.00	59.00				354.00					
		14	560.00	2.00	59.00	118.00					660.80				
	PB-3	12	600.00	1.00	19.00	19.00				114.00					
		14	560.00	2.00	19.00	38.00					212.80				
shear reinforcement	PB-1	6	25.10	148.00	59.00	8,732.00	2,191.73								
	PB-2	6	25.10	120.00	59.00	7,080.00	1,777.08								
	PB-3	6	25.10	120.00	19.00	2,280.00	572.28								
Negative Reinforcement	Middle	14	300.00	2.00	59.00	118.00					354.00				
		14	300.00	2.00	19.00	38.00					114.00				
	Edge	16	200.00	2.00	59.00	118.00						236.00			
Beam main reinforcement	Beam - A	16	1,020.00	3.00	1.00	3.00						30.60			
		16	300.00	3.00	1.00	3.00						9.00			
		16	150.00	2.00	1.00	2.00						3.00			
		20	1,000.00	3.00	1.00	3.00							30.00		
		20	300.00	3.00	1.00	3.00							9.00		
	Beam - B	20	3,870.00	3.00	1.00	3.00							116.10		
		20	3,750.00	3.00	1.00	3.00							112.50		
		20	400.00	8.00	1.00	8.00							32.00		
		20	240.00	2.00	1.00	2.00							4.80		
		20	800.00	3.00	1.00	3.00							24.00		
	Beam - C	20	3,870.00	3.00	1.00	3.00							116.10		
		20	3,750.00	4.00	1.00	4.00							150.00		
		20	400.00	8.00	1.00	8.00							32.00		
		20	240.00	2.00	1.00	2.00							4.80		
		20	800.00	3.00	1.00	3.00							24.00		
	Beam - D	20	3,870.00	3.00	1.00	3.00							116.10		
		20	3,750.00	4.00	1.00	4.00							150.00		
		20	400.00	5.00	1.00	5.00							20.00		
		20	240.00	3.00	1.00	3.00							7.20		
		20	200.00	3.00	1.00	3.00							6.00		
	Beam 2 and 8	14	1,040.00	3.00	2.00	6.00					62.40				
		16	1,080.00	3.00	2.00	6.00						64.80			
	Beam 4 and 5'	14	490.00	3.00	2.00	6.00					29.40				
		16	530.00	3.00	2.00	6.00						31.80			
stirrup	Beam A	8	150.00	19.00	1.00	19.00		28.50							
		10	150.00	49.00	1.00	49.00			73.50						
	Beam B and C	8	150.00	79.00	2.00	158.00		237.00							
		10	150.00	157.00	2.00	314.00			471.00						
		10	150.00	18.00	2.00	36.00			54.00						
	Beam D	8	160.00	79.00	1.00	79.00		126.40							
		10	160.00	193.00	2.00	386.00			617.60						
		Total Length (Ml)					**7,232.59**	**391.90**	**1,216.10**	**468.00**	**1,787.40**	**1,036.00**	**954.60**	**-**	
		Unit Weight (kg/ml)					0.222	0.395	0.617	0.888	1.209	1.579	2.467	3.552	
		Total Weight (Kg)					**1,605.64**	**154.80**	**750.33**	**415.58**	**2,160.97**	**1,635.84**	**2,355.00**	**-**	

Member	Description	Diam. (mm)	Bar Length	No. of Bar	No. Member	Total No. Bar	Total Length of Bar (M) 6	8	10	12	14	16	20	24	Ø32mm
	Precast lattice slab (Normal Weight)														
Mesh	Mesh	6	1,000.00	117.00	1.00	117.00	1,170.00								
		6	3,500.00	34.00	1.00	34.00	1,190.00								
		6	450.00	37.00	1.00	37.00	166.50								
		6	1,100.00	15.00	1.00	15.00	165.00								
Plate - 1	tension reinf.	10	550.00	11.00	37.00	407.00			2,238.50						
	comp reinf.	10	550.00	2.00	37.00	74.00			407.00						
	secondary reinf.	6	120.00	13.00	37.00	481.00	577.20								
	shear reinf.	6	16.67	296.00	37.00	10,952.00	1,825.70								
Plate - 2	tension reinf.	10	450.00	8.00	20.00	160.00			720.00						
	comp reinf.	10	450.00	2.00	20.00	40.00			180.00						
	secondary reinf.	6	120.00	10.00	20.00	200.00	240.00								
	shear reinf.	6	16.67	240.00	20.00	4,800.00	800.16								
Plate - 3	tension reinf.	8	450.00	8.00	8.00	64.00		288.00							
	comp reinf.	8	450.00	2.00	8.00	16.00		72.00							
	secondary reinf.	6	120.00	10.00	8.00	80.00	96.00								
	shear reinf.	6	16.67	240.00	8.00	1,920.00	320.06								
Neg. reinf	Part A	10	300.00	13.00	20.00	260.00			780.00						
	Part B	10	300.00	11.00	8.00	88.00			264.00						
		10	300.00	9.00	8.00	72.00			216.00						
Beam main reinf.	Beam A	14	1,020.00	3.00	1.00	3.00					30.60				
		14	300.00	1.00	1.00	1.00					3.00				
		16	300.00	3.00	1.00	3.00						9.00			
		20	1,000.00	2.00	1.00	2.00							20.00		
	Beam B	16	400.00	2.00	1.00	2.00						8.00			
		20	3,870.00	2.00	1.00	2.00							77.40		
		20	3,750.00	3.00	1.00	3.00							112.50		
		20	400.00	4.00	1.00	4.00							16.00		
		24	400.00	3.00	1.00	3.00								12.00	
	Beam C	14	180.00	1.00	1.00	1.00					1.80				
		16	3,870.00	3.00	1.00	3.00						116.10			
		20	3,750.00	4.00	1.00	4.00							150.00		
		20	400.00	11.00	1.00	11.00							44.00		
		20	220.00	2.00	1.00	2.00							4.40		
		24	400.00	9.00	1.00	9.00								36.00	
	Beam D	14	3,870.00	3.00	1.00	3.00					116.10				
		16	3,750.00	4.00	1.00	4.00						150.00			
		20	400.00	7.00	1.00	7.00							28.00		
	Beam 2 and 8	12	1,080.00	2.00	2.00	4.00				43.20					
		14	1,040.00	2.00	2.00	4.00					41.60				
	Beam 4 and 5'	12	530.00	2.00	2.00	4.00				21.20					
		14	490.00	2.00	2.00	4.00					19.60				
stirrup		8	150.00	356.00	1.00	356.00		534.00							
		8	170.00	48.00	1.00	48.00		81.60							
		8	150.00	39.00	1.00	39.00		58.50							
		8	150.00	25.00	1.00	25.00		37.50							
		8	150.00	116.00	1.00	116.00		174.00							
		8	150.00	42.00	1.00	42.00		63.00							
		8	150.00	145.00	1.00	145.00		217.50							
		8	150.00	35.00	1.00	35.00		52.50							
neg reinf	Part A	10	200.00	10.00	30.00	300.00			600.00						
	Part B	10	200.00	8.00	30.00	240.00			480.00						
		Total Length (Ml)					**6,550.62**	**1,578.60**	**4,805.50**	**64.40**	**212.70**	**283.10**	**452.30**	**48.00**	
		Unit Weight (kg/ml)					0.222	0.395	0.617	0.888	1.209	1.579	2.467	3.552	
		Total Weight (Kg)					**1,454.24**	**623.55**	**2,964.99**	**57.19**	**257.15**	**447.01**	**1,115.82**	**170.50**	

Member	Description	Diam. (mm)	Bar Length	No. of Bar	No. Member	Total No. Bar	Total Length of Bar (M)								
							6	8	10	12	14	16	20	24	Ø32mm
	Precast lattice slab (Light Weight)														
Mesh	Mesh	6	1,000.00	117.00	1.00	117.00	1,170.00								
		6	3,500.00	34.00	1.00	34.00	1,190.00								
		6	450.00	37.00	1.00	37.00	166.50								
		6	1,100.00	15.00	1.00	15.00	165.00								
Plate - 1	tension reinf.	10	550.00	10.00	37.00	370.00			2,035.00						
	comp reinf.	10	550.00	2.00	37.00	74.00			407.00						
	secondary reinf.	6	120.00	13.00	37.00	481.00	577.20								
	shear reinf.	6	16.67	296.00	37.00	10,952.00	1,825.70								
Plate - 2	tension reinf.	10	450.00	7.00	20.00	140.00			630.00						
	comp reinf.	10	450.00	2.00	20.00	40.00			180.00						
	secondary reinf.	6	120.00	10.00	20.00	200.00	240.00								
	shear reinf.	6	16.67	240.00	20.00	4,800.00	800.16								
Plate - 3	tension reinf.	8	450.00	8.00	8.00	64.00		288.00							
	comp reinf.	8	450.00	2.00	8.00	16.00		72.00							
	secondary reinf.	6	120.00	10.00	8.00	80.00	96.00								
	shear reinf.	6	16.67	240.00	8.00	1,920.00	320.06								
Neg. reinf (middle)	Part A	10	300.00	10.00	20.00	200.00			600.00						
	Part B	10	300.00	9.00	8.00	72.00			216.00						
		10	300.00	7.00	8.00	56.00			168.00						
Beam main reinf.	Beam A	14	1,020.00	3.00	1.00	3.00					30.60				
		14	300.00	3.00	1.00	3.00					9.00				
		16	1,000.00	1.00	1.00	1.00						10.00			
		20	1,000.00	1.00	1.00	1.00							10.00		
	Beam B	14	3,870.00	3.00	1.00	3.00					116.10				
		14	400.00	2.00	1.00	2.00					8.00				
		16	3,750.00	4.00	1.00	4.00						150.00			
		16	400.00	6.00	1.00	6.00						24.00			
		20	400.00	4.00	1.00	4.00							16.00		
	Beam C	14	180.00	1.00	1.00	1.00					1.80				
		16	3,870.00	3.00	1.00	3.00						116.10			
		20	3,750.00	4.00	1.00	4.00							150.00		
		20	400.00	11.00	1.00	11.00							44.00		
		20	220.00	2.00	1.00	2.00							4.40		
		24	400.00	9.00	1.00	9.00								36.00	
	Beam D	12	400.00	3.00	1.00	3.00				12.00					
		14	3,870.00	3.00	1.00	3.00					116.10				
		16	3,750.00	3.00	1.00	3.00						112.50			
		16	400.00	4.00	1.00	4.00						16.00			
	Beam 2 and 8	12	1,080.00	2.00	2.00	4.00				43.20					
		14	1,040.00	2.00	2.00	4.00					41.60				
	Beam 4 and 5'	12	530.00	2.00	2.00	4.00				21.20					
		14	490.00	2.00	2.00	4.00					19.60				
stirrup		8	150.00	713.00	1.00	713.00		1,069.50							
		Total Length (Ml)					**6,550.62**	**1,429.50**	**4,236.00**	**76.40**	**342.80**	**428.60**	**224.40**	**36.00**	
		Unit Weight (kg/ml)					0.222	0.395	0.617	0.888	1.209	1.579	2.467	3.552	
		Total Weight (Kg)					**1,454.24**	**564.65**	**2,613.61**	**67.84**	**414.45**	**676.76**	**553.59**	**127.87**	

Table E.2: Rebar Takeoff Sheet

E.3. Summary of Concrete and Rebar Takeoff

Materials	Type of Slab		
	Ribbed Slab	Normal Weight Lattice Slab	Light Weight Lattice Slab
Slab Concrete	400 m^2	79.08 m^3	79.08 m^3
Beam Concrete	18.69 m^3	18.69 m^3	17.81 m^3
Rebar	9078.16 kg	7090.45 kg	6473.01 kg
Formwork	585.12 m^2	-	-

Table E.3: Summary of Concrete and Rebar Takeoff

APPENDIX F: Typical Floor Layout

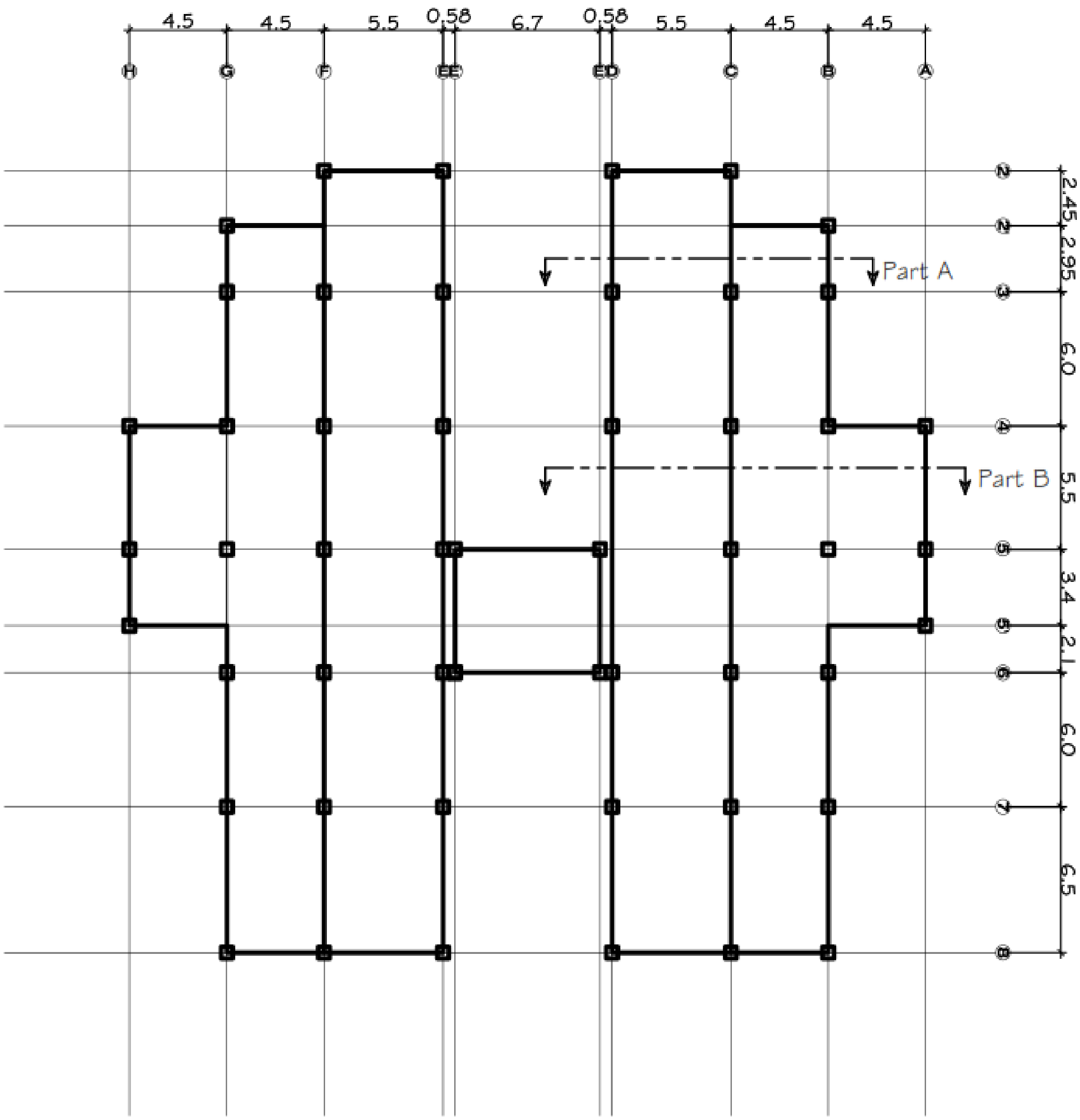


Figure F.1: Typical Floor Plan

APPENDIX G: Rib Arrangement and Detailing of Ribbed Slab

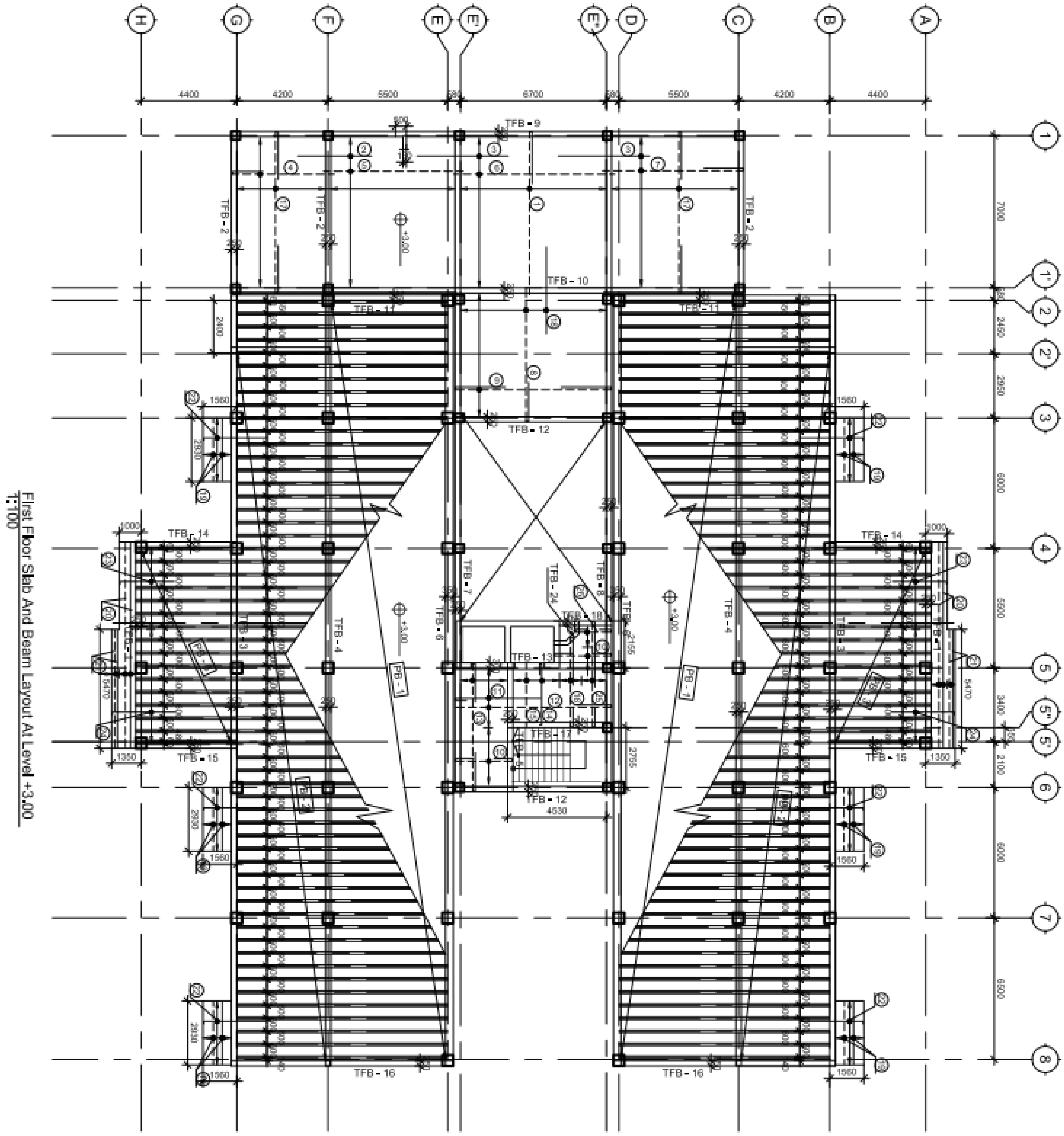


Figure G.1: Arrangement of Ribs on a Typical Floor

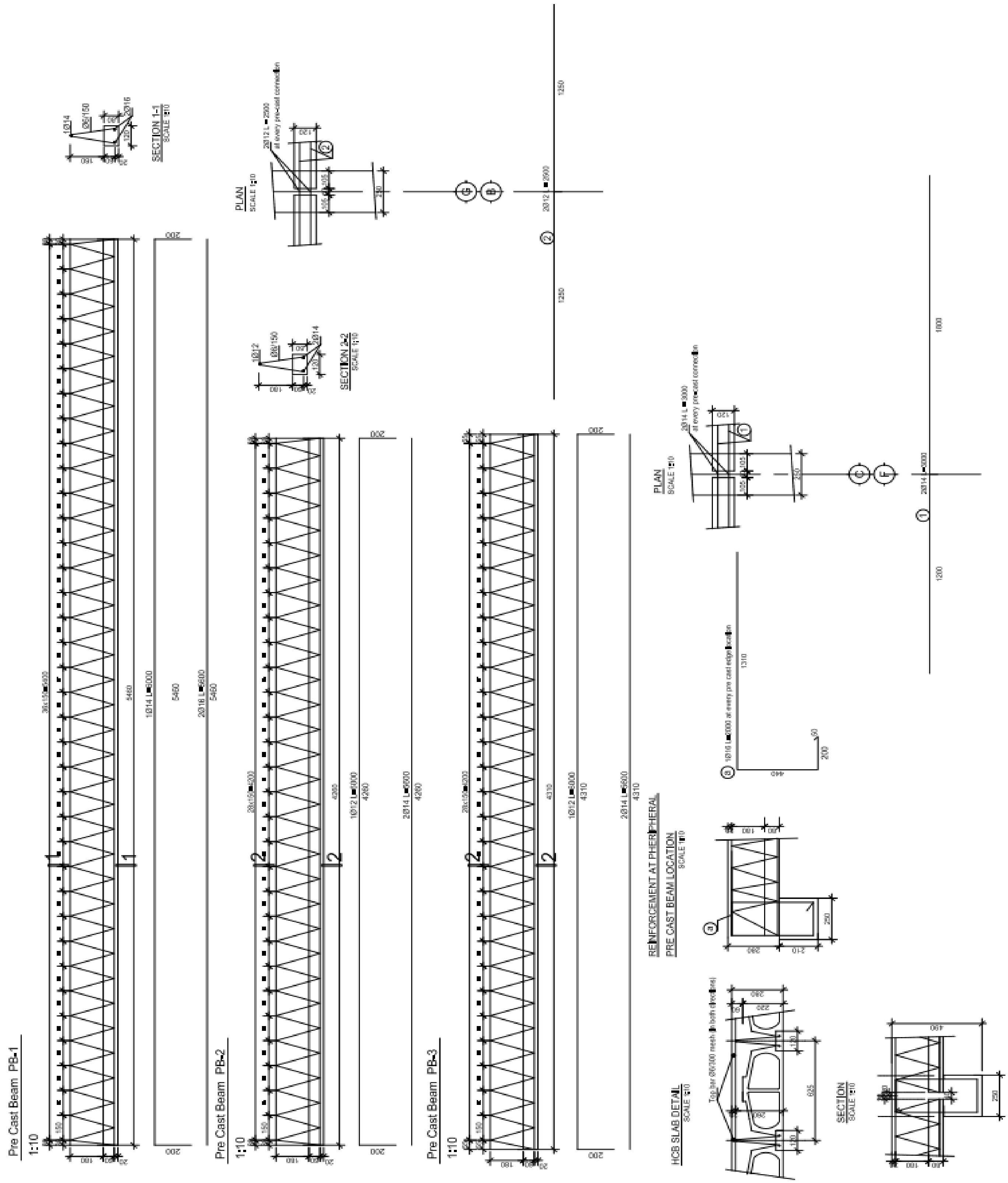


Figure G.2: Detailing of a Sample Rib

APPENDIX H: Joints for Precast Lattice Slab

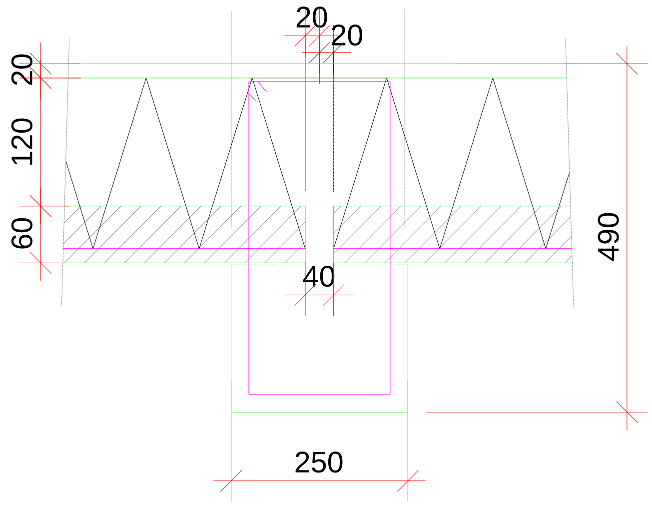


Figure H.1: Plate to Plate Connection (section view)

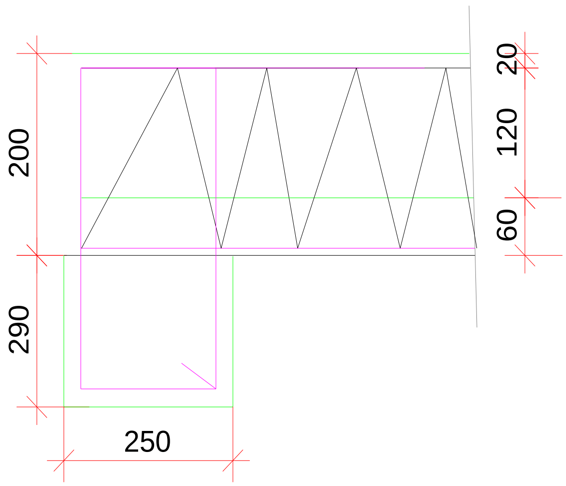


Figure H.2: Beam to Plate Connection

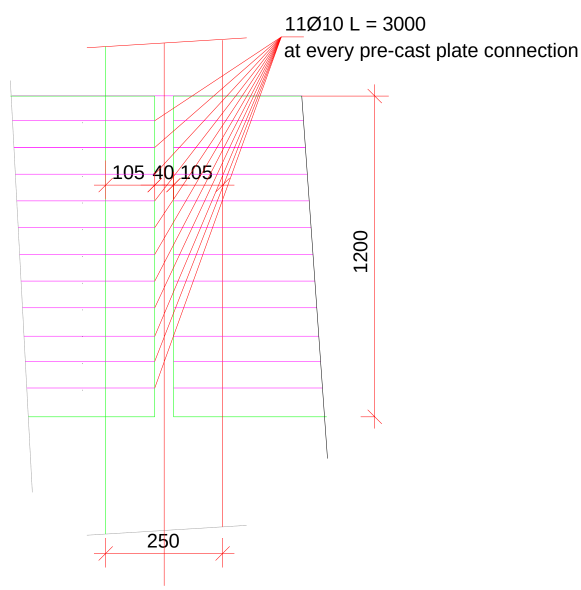


Figure H.3: Plate to plate connection (Plan view)

APPENDIX J: Questionnaire

Aim of survey

To study the progress of precast concrete building element technology in the construction industry and to assess its acceptance by the construction society in the existing market of the country especially in the low cost housing projects.

Remarks from the researcher:

- ✓ Your responses will be treated with confidence and at all times data will be presented in such a way that your identity cannot be connected with specific published data.
- ✓ It is kindly asked of you to address all questions truthfully and neatly.

1. Educational background in Civil Engineering/ Construction related fields
   ☐ Certificate ☐ Diploma ☐ Degree ☐ Masters Degree ☐ PhD
2. Job title
   ☐ Architect ☐ Structural Eng ☐ Construction Manager ☐ Site Eng.
3. Total work experience on the field of construction
   ☐ 0-2 years ☐ 2-5 years ☐ 5-10 years ☐ above 10 years
4. Have you ever worked on building projects other than the low cost housing projects which involved concrete precast building elements?
   ☐ Yes ☐ No
5. While working on building projects which of the following construction method do you prefer for the project under execution?
   ☐ Composite (Precast and Cast insitu) ☐ Cast insitu
6. If your answer to question No. 5 is "Cast insitu", why did you avoid using the composite method of construction? (please tick only 3 options)
   ☐ I am not sure of its quality assurance.
   ☐ It is a relatively time taking method.
   ☐ I do not have enough knowledge and experience in the area.
   ☐ I do not think it is cost efficient.
   ☐ I do not think it allows design freedom.
   ☐ I doubt its ease of construction.
   ☐ I doubt its ease of manufacturing.
   ☐ Other

7. Are you informed of other forms of precast concrete elements other than precast ribs which are used for low cost housing projects in Ethiopian construction market?

☐ Yes ☐ No

8. What is your opinion on the development of precast concrete technology in Ethiopia?

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9. Willingness to take trainings on precast concrete technology (tick only one option)

☐ Very Interested ☐ Interested ☐ Neutral ☐ Not Interested

THANK YOU FOR YOUR TIME!!!