FINAL PROJECT (RC14-1501)

REDESIGNING THE BUILDING's STRUCTURE OF HOTEL NOVOTEL THE SAMATOR SURABAYA BY USING PRECAST CONCRETE

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DEPARTEMENT OF CIVIL ENGINEERING
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Surabaya 2017

TUGAS AKHIR (RC14-1501)

PERENCANAAN ULANG STRUKTUR BANGUNAN HOTEL NOVOTEL THE SAMATOR SURABAYA MENGGUNAKAN BETON PRACETAK

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Surabaya 2017

REDESIGNING THE BUILDING's STRUCTURE OF HOTEL NOVOTEL THE SAMATOR SURABAYA BY USING PRECAST CONCRETE

FINAL PROJECT

Filed to Fulfill One of Conditions
To Get Bachelor Degree in Civil Engineering
at
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Departement of Civil Engineering
Faculty of Civil Engineering and Planning
Tenth of November Institute of Technology Surabaya

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SURABAYA
JULY, 2017

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REDESIGNING THE BUILDING's STRUCTURE OF HOTEL NOVOTEL THE SAMATOR SURABAYA BY USING PRECAST CONCRETE

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ABSTRACT

Precast method nowadays has been used in many civil constructions. The precast concrete has advantages to be compared to cast in situ concrete. The advantages of using the precast concrete are firstly the process of concrete casting is not influenced by weather; secondly, it does not need a lot of formworks, efficiency of times, and better quality controls; thirdly, a new research stated that using precast concrete is eco friendly.

The purpose of this final project is to design of a structure plan of Hotel Novotel THE SAMATOR Surabaya's building with precast method. The objective of this project to design the detail of the concrete's reinforcement, the connection between precast element, the basement's structure and the foundation that support the building. Finally to draw the result of the modification of the building.

Hotel Novotel THE SAMATOR Surabaya's building has 25 floors and two storeis of basement. It was planned using precast method for the beams and the slabs, whereas the columns, stairs, and footing were planned using the cast in situ concrete. The regulation that were used for this planning are SNI 2847:2013 for the structural concrete planning, PPIUG 1983 and SNI 1727:2013 for the gravity loads, SNI 1726:2012 for the lateral (earthquake/seismic) loads. This building was planned using Dual System Special Moment Resisting Frame.

The results of the design modification of Hotel Novotel THE SAMATOR SUrabaya's building were primary beams dimension of 40/65, secondary beams dimension of 30/45, and column's dimension using 80x80. The connection between precast element used wet joints and brackets.

Keywords: Hotel Novotel THE SAMATOR Surabaya, Planning Modification, Precsat, Reinforced Concrete

PERENCANAAN ULANG STRUKTUR BANGUNAN HOTEL NOVOTEL THE SAMATOR SURABAYA DENGAN MENGGUNAKAN BETON PRACETAK

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ABSTRAK

Metode pracetak saat ini telah banyak digunakan dalam pembangunan konstruksi sipil. Hal ini terjadi karena beton pracetak memiliki beberapa kelebihan dibandingkan beton yang dicor di tempat (*castin situ*). Kelebihannya antara lain yaitu proses pembuatannya yang tidak bergantung cuaca, tidak memerlukan banyak bekisting, waktu pengerjaan yang lebih singkat, kontrol kualitas beton lebih terjamin serta menurut penelitian terbaru beton pracetak juga ramah lingkungan

Tujuan dari Tugas Akhir ini adalah menghasilkan perencanaan struktur gedung Hotel Novotel THE SAMATOR Surabaya dengan metode pracetak. Merencanakan *detailing* penulangan dan sambungan pada elemen beton pracetak. Merencanakan pondasi yang menopang bangunan. Dan merancang gambar teknik dari hasil modifikasi gedung ini.

Gedung Hotel Novotel THE SAMATOR ini memliki tinggi 25 lantai dan dua tingkat basement. Gedung ini dirancang ulang menggunakan metode beton pracetak pada bagian balok dan pelat. Standar yang digunakan dalam perencanaan ini adalah perencanaan struktural menggunakan tata cara perhitungan struktur beton untuk bangunan gedung (SNI 2847:2013), untuk menghitung pembebanan gravitasi menggunakan PPIUG 1983 dan tata cara perhitungan pembebanan untuk gedung (SNI 1727:2013), dan pembebanan gempa dihitung menggunakan tata cara perencanaan ketahanan gempa (SNI 1726:2012). Perencanaan gedung ini menggunakan sistem rangka pemikul momen khusus (SRPMK).

Hasil dari modifikasi gedung Hotel Novotel THE SAMATOR Surabaya ini meliputi ukuran balok induk 40/65, ukuran balok anak 30/45, dan ukuran kolom 80x80 cm. Sambungan antar elemen pracetak menggunakan sambungan basah dan konsol pendek.

Kata Kunci: Hotel Novotel THE SAMATOR Surabaya, Modifikasi Perencanaan, Pracetak, Beton Bertulang

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1. Allah SWT, The God Almighty.
2. Prophet Muhammad Sallalahualaihiwassalam, p.b.u.h.
3. Author's parents, Drs. Khabib Isma'i, and Yuliati
4. Data Iranata, ST, MT, PhD., and Dr. Ir. Djoko Irawan, MS. as supervisor lecturer
5. Data Iranata, ST, MT, PhD. as advisor lecturer
6. Friends and families of ITS Surabaya

The author realises that this book may have some inferiority. So, the critics and advices from any parties are welcome.

Surabaya, July 2017

Student

Soca Fahreza Isma'i

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LIST OF CONTENTS

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LIST OF TABLES

LIST OF FIGURES

CHAPTER I
INTRODUCTION

1.1. Background

Recently, there are various of precast concrete structure system which have been developed by various of private company, goverment agency, or state-owned enterprises that support construction sector in Indonesia. Those structure systems have been tested and applied on construction sector in the form of building or bridge.Precast concrete structure system has some superiorities compared to cast in place concrete, including efficiency of working process, efficiency of energy, and enviromentally friendly.

According to Ervianto (2006), compared to cast in place concrete, the precast concrete has better quality.This could happen because the manufacturing process of precast concrete is performed by machine, the conditions of precast concrete manufactory that is relatively constant, and more accurate supervision of precast concrete production process.

According to data from Dispendukcapil (2012), Surabaya has nearly 3.2 millions reasidents (Figure 1.1). The growth population in Surabaya is approximately 80000 residents per year. This makes Surabaya become one of the metropolitan city in Indonesia due to its population and its condition. Surabaya is also known due to its destination for tourist although it might be not as attractive as Bali Island. This makes sense since the local government promoted “SPARKLING SURABAYA” as city motto. Then the need to build hotel is considered to be important due to the tourist visit. The building process needs to be fast and environmental friendly so that the building with precast concrete structure is needed.

Based on the facts above, the author saw the superiority of precast concrete compared to cast in place concrete. So the author would like to file a Final Project Proposal withthe title as following “Redesigning The Building’s Structure of Hotel Novotel THE SAMATOR Surabaya By Using Precast Concrete”. Hotel Novotel THE SAMATOR Surabaya uses cast in place concrete for its construction process in reality. This building has 25 (twenty five) storeys.

Table 1.1. The Population in Surabaya (source: Dispendukcapil, 2012)

Number	**Year**	**Total;**
1	2003	2485761
2	2004	2509833
3	2005	2528777
4	2006	2687456
5	2007	2829552
6	2008	2903382
7	2009	2938225
8	2010	2929528
9	2011	3024321
10	2012	3125376

1.2. Problems

The problems of this final project are divided into two as following: main problem and detail of problems

1. Main Problem

1. How to design the building of Hotel Novotel THE SAMATOR Surabaya using precast concrete?

2. Detail of Problems

1. How is the design of *preliminarydesign*of precast concrete element?
2. How is the building load calculation?
3. How is the building structure analysis?
4. How to design the precast concrete element dimension?

5. How to design the detail of precast component connection?
6. How to design the building foundation?
7. How to draw the technical drawing of calculation result and design of the building?

1.3. Final Project's Goals

The goals of this final project are the following:

1. To design the *preliminary design* of precast concrete element
2. To calculate the building load
3. To analyze the building structure
4. To design the precast concrete element dimension
5. To design the detail of precast component connection
6. To design the building foundation
7. To draw the technical drawing of calculation result and design of the building

1.4. Limitation of Problems

1. Not counting the project budget
2. Not counting the project schedule
3. Not designing the electrical and mechanical detailing of the building
4. Not designing the archithectural detailing of the building building
5. Not designing the drainage detailing of the building
6. Not analyzing the social and economical aspect of project
7. Not using precast concrete on these elements: pillar, shear wall, and basement
8. Using precast concrete only on these elements: primary beam, secondary beam, and slab

1.5. Final Project's Benefits

The expected benefits of this final project are:

1. Understanding the designing process of multi-storey building structure using precast concrete
2. Adding knowledge for the author about multi-storey building structure by using precast concrete

CHAPTER II
LITERATURE REVIEW

2.1. General

Precast concrete is an alternative that is used in construction process in addition to cast in place concrete. The precast concrete is made in factory (*fabrication*), then sent to construction site (*transportation*), then connected become a structure (*erection*). The superiority of precast concrete compared to cast in place concrete one of them is faster in working process on construction site, it is one thing that is the cause of the increasing number of the structures that use precast concrete especially building structure or bridge structure.

In reality on construction site, the project of building Hotel Novotel THE SAMATOR Surabaya uses cast in place concrete. But in this final project the building will be redesigned using precast concrete. In this chapter will discuss about the references that used for the design so that the building can receive and bear the planned load in accordance with requirement and applicable provision.

2.2. Building's System Structure

There are some structure systems that commonly used as the bearing of sesimic force on building design.In this final project the building's system structure of Hotel Novotel THE SAMATOR Surabaya will use dual system structure system (see SNI, 2012 Table 9). Dual system structure is the combination of bearers moment frame system and shear wall that work together to restrain seismic force. Bearers moment frame system bears gravitational load and lateral load. While the shear wall only bears the lateral load.

2.3. Earthquake Resistance Construction

The minimum safety standardfor building and house that fits into earthquake resistance building category shall meet the following conditions (PU, 2006).

1.as exposed to weak earthquake, the building does not experience damage at all
2.as exposed to medium earthquake, the building can experience damage on non-structural element, but should not experience damage on structural element
3.as exposed to very strong earthquake, the building should not collapse either partially or completely; the building should not experience damage that can not be repaired; the building can experience damage but the damage should can be repaired quickly so the building's functions become normal again.

2.4. Precast Concrete System in Building

2.4.1. Elementof Precast Concrete and Cast in Place Concrete

The precast concrete element in this final project include beam and slab, for pillar and shear wall use cast in place concrete. The precast concrete type that will be used for those two components is non-prestressed precast concrete. For the various of precast concrete element that commonly used see Figure 2.1

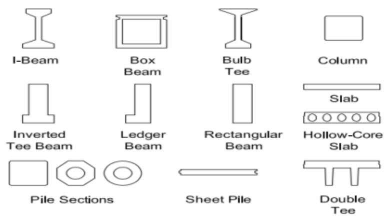
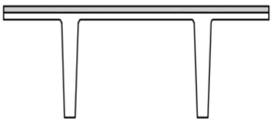
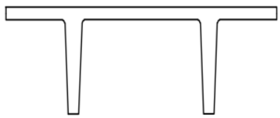


Figure 2.1. The variety of precast concrete elements (source: PCI, 2004)

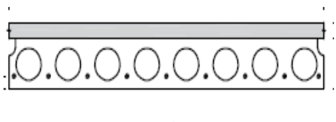
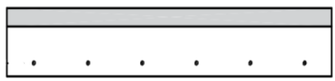
2.4.1.1. Slab

Slab is thin structure that made from concrete which its field is horizontal and the working load works perpendiculary to it. The field slab's thickness is relatively very small compared to its length or its width. In building's structure commonly slab functionates as horizontal stiffener element which useful to support the stiffness of portal beam.

According to PCI (2004),there are various of precast concrete slab including *hollow-core slab*, *solid flat slab*,*double tees slab*, and*pretopped double tee slab* (Figure 2.2). In this final project will use *solidflat slab*.

a. double tees slab *b. pretopped double tees slab*

c. hollow-core slab *d. solid flat slab*

Figure 2.2. The variety of precast concrete slab elements (source: PCI, 2004)

2.4.1.2. Beam

Beam is a component which bears its own weight and slab's load and distribute to the pillar. According to PCI (2004), precast concrete beam element that commonly used are including *inverted tee beam, ledger beam*, and*rectangular beam*(Figure2.3). In this final project will use *rectangular beam*.

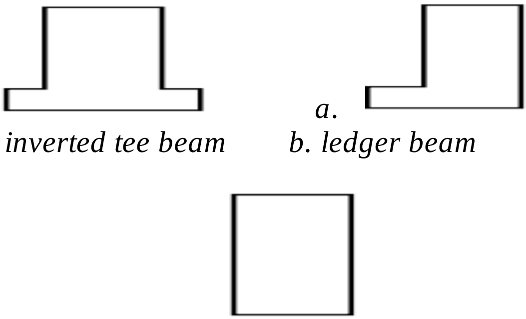


Figure 2.3. The variety of precast concrete beam elements (source: PCI, 2004)

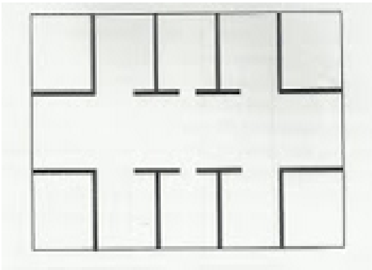
2.4.1.3. Shear Wall

According to SNI (2002) states that shear wall is structure component which used to increases structure's stiffness and bearthe lateral forces. Based on its layout and its function, shear wall can be classified as the following (Figure 2.4)

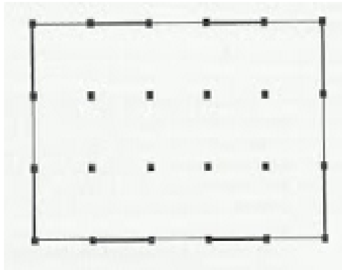
1.bearing walls is shear wall which also supports most of gravity load. These walls also uses adjacent partition walls

2.frame walls is shear wall which also bears lateral load which the gravity load comes from reinforced concrete frame. These walls are built between pillars

3.core walls is shear wall which is located in central core zone in the building that usually filled by stairs or elevator shaft.



a. bearing walls



b. frame walls

.

c. core walls

Figure 2.4. The variety of shear wall
(source: Private Documentation)

2.4.1.4. Pillar

According to SNI (2002) which itstates that pillar is structure component which has high ratio to the smallest lateral dimension which exceed 3 that used especially to support compressive axial load. Pillar functionates as a load bearing which its come from primary beam and distribute to the foundation. In this final project will use cast in place concrete pillar.

2.4.2. Connection Between Precast Concrete Elements

According toErvianto (2006)the precast concrete component connection can be solved in two ways: wet connection and dry connection.

2.4.2.1. Wet Connection

Wet connection can be divided into two types as the following

1. *In-situ Concrete Joint*

This type of connection can be applied to precast concrete component between pillar with pillar, pillar with beam, and slab with beam. The implimentation method is doing casting to the components assembly. It is expected that the components assembly can merge. While for the reinforcement assembly can be solve by*overlapping*.

In the process of connecting of precast components it should be avoided the connection which has large number of components at one point. This can be solved by placing connection between the pillar on connection point between pillar and beam. The implementation process of *in-situ concrete joint*connection based on its phase implementation can be divided into two as the following

- one phase implementation

the meaning here is the process of connection implementation between beam and pillar which casted in once casting process. The process of steel connection can be solved using *overlapping*.

- two phases implementation

the two phases implementation is applied to merge precast concrete components which can be solved into two phases. The example of this condition is the merging process of beam and pillar, the first phase is the merging implementation between pillar and beam then proceed to casting process between pillar and pillar (Figure 2.5, Figure 2.6, and Figure 2.7).

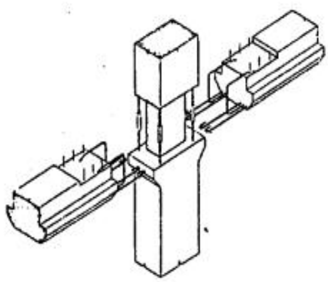


Figure 2.5. The connection between pillar and beam by cast in place concrete, one phase implementation. The steel bar connection is merged using weld. The unity of beam and pillar is accomplished by adding steel bar on the top of the end of beam
(source: Ervianto, 2006)

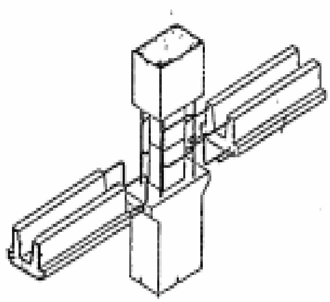


Figure 2.6. The connection between pillar and beam by cast in place concrete, two phases implementation. The first phase is the casting process of the connection between the bottom of pillar and beam which given socket for the placement of the top of the pillar. The second phase is the casting process of the connection of between pillars
(source: Ervianto, 2006)

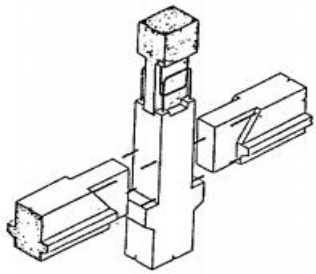


Figure 2.7. The connection between pillar that its place differs with the connection between pillarsand beam (source: Ervianto, 2006)

2. *Pre-Packed Aggregate*

This type of connection is placing aggregate in the part which will be connected and then injected by cement paste in that part using hydraulic pump so that the cement paste will fill the aggregate void.

2.4.2.2. Dry Connection

The dry connection can be divided into two types as the following

1. Weld Connection

This type of connection uses steel plate which attached into precast concrete that will be connected. Then these two plates are connected by weld. Through this steel plate the force will be forwarded to the relevant component. After the welding process then continued with closing the connected plate using concrete to protect the plate from corosion (Figure 2.8).

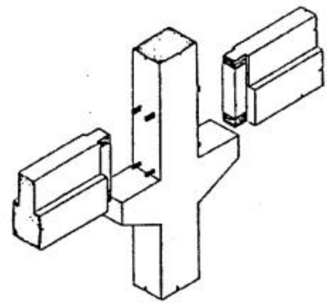


Figure 2.8. The rigid connection between beam and pillar with continuous pillar by weld connection tool
(source: Ervianto, 2006)

2. Bolt Conenction

This type of connection also need the steel plate in both of precast concrete element which will be connected. These two components are connected by connection tool which in the form of high tensile strength bolt. Then the plate is casted using concrete in order to protect from the corosion (Figure 2.9).

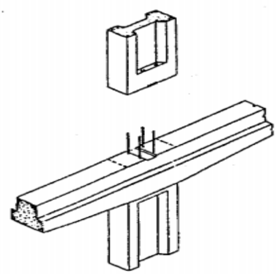


Figure 2.9. Pin-joint connection in pillar by bolt as connection tool
(source: Ervianto, 2006)

2.4.3. Lifting Points of Precast Concrete Element

According to PCI (2004), there are some types of lifting points of precast concrete panels as the following: *one edge*lifting points, *one edge*four lifting points, *stripped flat*four lifting points, and *stripped flat*eight lifting points (Figure 2.10, Figure 2.11, Figure 2.12, dan Figure 2.13).To ease the lifting implementation of precast concrete element, can use*spreader beam* so that the lifting load which received by precast concrete element can be spreaded (Figure 2.14, and Figure 2.15). For precast concrete beam element the calculation of the lifting moment has to be calculated (Figure 2.16).

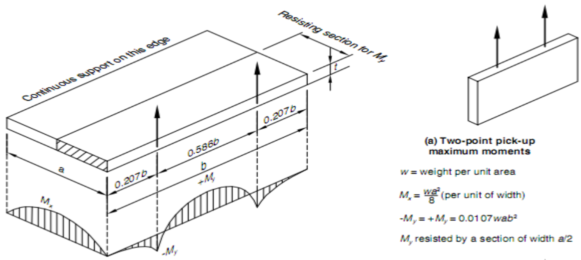


Figure 2.10. Two lifting points of *one edge panels* of precast concrete (source: PCI, 2004)

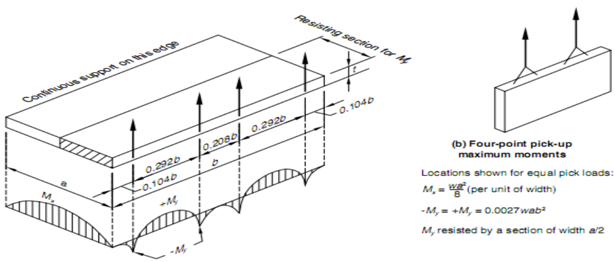


Figure 2.11. Four lifting points of *one edge panels* of precast concrete (source: PCI, 2004)

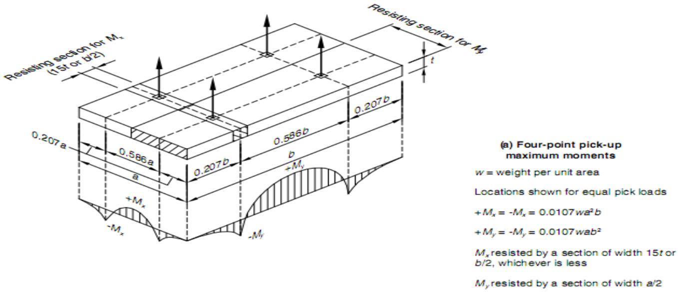


Figure 2.12. Four lifting points of *stripped flat panels* of precast concrete (source: PCI, 2004)

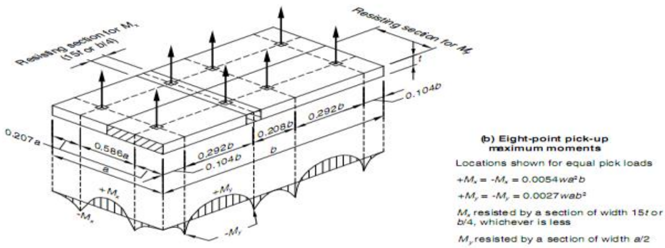


Figure 2.13. Eight lifting points of *stripped flat panels* of precast concrete (source: PCI, 2004)

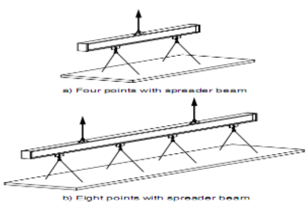


Figure 2.14. The use of spreader beam for lifting panels of precast concrete (source: PCI, 2004)

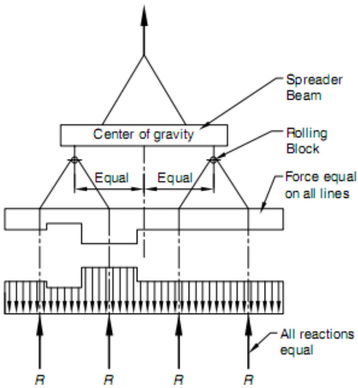


Figure 2.15. The configuration when lifting process of panels of precast concrete elementso that the lifting load which received by precast concrete element can be spreaded (source: PCI, 2004)

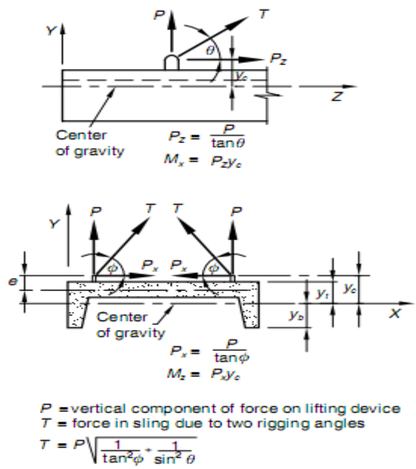


Figure 2.16. Moment force that occurs as a result of lifting of precast concrete beam element (source: PCI, 2004)

2.4.4. Handling Phases of Precast Concrete Element

According to PCI (2004) there are four handling phases of precast concrete as the following: *stripping, yarding and storage, transportation,* and *erection*.

1. *Stripping* (phase of lifting process from formwork)

The things that need to be considered when *stripping*process occur are the following

a.the orientation of precast concrete element whether it is horizontally, vertically, or form an angle

b.the attachement between concrete surface and formwork

c.the number and the location of lifting

d.the weight of precast concrete element and additional loads like carried formwork when lifting process of precast concrete element occurs

2. *Yarding and Storage* (phase of placement and storage)

The things that need to be considered when *yarding and storage* process occur are the following

a.the orientation of precast concrete element whether it is horizontally, vertically, or form an angle

b.the location of temporary lifting points of precast concrete element

c.the support location due to the other precast concrete element that also stored

d.the protection of precast concrete element from the weather like sunshine and rain

3. *Transportation* (phase of transference to construction site)

The things that need to be considered when *transportation* process occur are the following

a.the orientation of precast concrete element whether it is horizontally, vertically, or form an angle

b.the support location of precast concrete element whether it is vertically or horizontally

c.the condition of transpot vehicle, the condition of road that will be passed by the vehicle, and load limitation on the road
d.dynamic considerations while transporting

4. *Erection* (phase of erecting)

The things that need to be considered when *erection* process occur are the following
a.the orientation of precast concrete element whether it is horizontally, vertically, or form an angle
b.the location and the number of lifting points
c.the location and the number of support points
d.the temporary loads like employees, equipments during construction process, and the weight of *overtopping* concrete.

2.5. Basement Structure

For designing of basement structure, it consists of designing of basement pillar, designing of basement primary beam and secondary beam, designing of basement slab, and designing of basement wall (turap). The basement structure's element will use cast in place concrete (without precast concrete). For the basement wall will funcionate as a soil bearing wall which it bear the soil's lateral force behind the wall plus water lateral force (Figure 2.17). For the steps of designingbasementstructure will be same as steps of designing other precast concrete element.

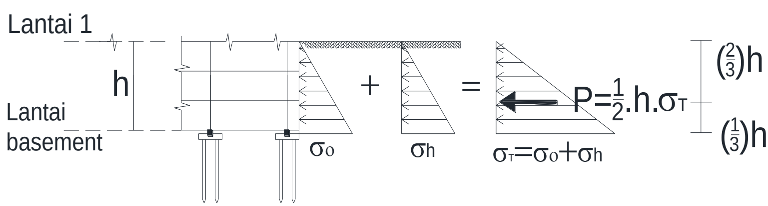


Figure 2.17. Soil's lateral force plus water lateral force which are retained by basement wall
(source: Private Documentation)

2.6. Steps of Implimentation on Construction Site

For steps of implimentation on construction site see the description below (note that the basement structure uses cast in place concrete while the upper structure uses precast concrete except for pillar and shear wall, this chapter will be explained more detailed in the next chapter)

a. producing the precast concrete element
b. stripping the precast concrete element (Figure 2.18)

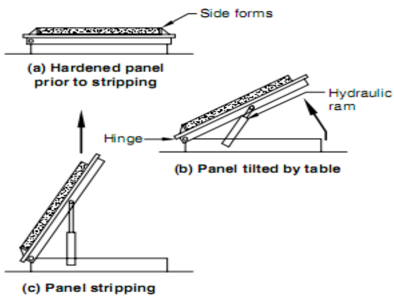


Figure 2.18. Stripping process of precast concrete element (source: PCI, 2004)

c. transporting the precast concrete element (Figure 2.20), note that before transporting begin should be considered about lifting process on factory (Figure 2.19)

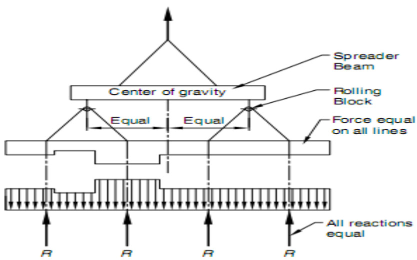


Figure 2.19. Lifting process of precast concrete element (source: PCI, 2004)

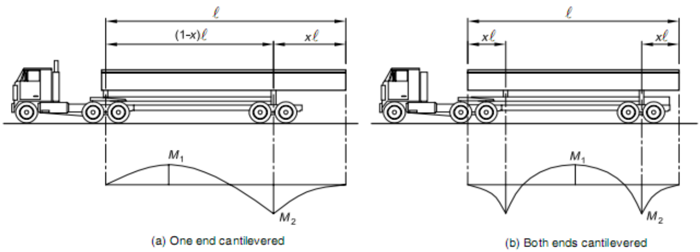


Figure 2.20. The configuration when transporting precast concrete element
(source: PCI, 2004)

d. yarding the precast concrete element on construction site (Figure 2.21)

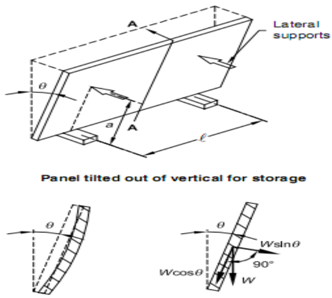


Figure 2.21. The example of configuration when yarding precast concrete element
(source: PCI, 2004)

e. before lifting process of precast concrete element, should be considered that the construction site already has the equipments for construction working such as excavator, tower crane, etc
f. the construction site shall be trim by bulldozer, excavator, or another equipment

g. the excavation process shall be done according to basement and foundation depth (approx. 4 meters) (note that to bear the lateral soil force due to excavation, shall use temporary bearer such as plaster (turap), etc.
h. the installation procees of foundation shall be done first, then install the sloof and poer bar and cast using cast in place concrete to form poer and and sloof (note that adding the formwork firstly before casting process)
i. the casting process of basement pillar shall be done after poer and sloof formed, then the basement pillar is casted using cast in place concrete (note that adding the formwork firstly before casting process, for basement pillar's formwork should be supported by sideway cantilever)
j. the casting process of basement wall shall be done after basement pillar formed, then the basement wall is casted using cast in place concrete (note that adding the formwork firstly before casting process, for basement wall's formwork should be supported by sideway cantilever)
k. the casting process of basement primary beam can be done after basement pillar formed, then the basement primary beam is casted using cast in place concrete (note that adding the formwork firstly before casting process, for basement primary beam's formwork should be supported by scaffolding)
l. the casting process of basement slab can be done after basement primary beam formed, then the basement primary beam is casted using cast in place concrete (note that adding the formwork firstly before casting process, for basement primary beam's formwork should be supported by scaffolding) (see Figure 2.22

for ilustation about basement structure implementation)

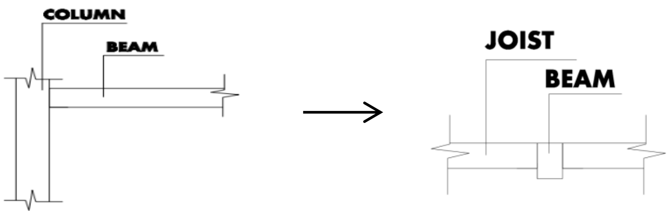


Figure 2.22. Ilustation about basement structure implementation
(source: Private Documentation)

m. the shear wall's structure shall be installed continuously after basement's structure completed and it is built per step storey
n. for upper structure the steps of implimentation are not much different from basement structure, firsly cast the pillar, then lift the precast primary beam and connnect it to pillar, then lift the precast secondary beam and connect it to primary beam, then last lift the precast slab (Figure 2.18) and connect it to primary beam and secondary beam (note that using scaffolding to temporary bear the load produced by precast element's load)

o. do and repeat the step m until the building's structure finished (see Figure 2.23 for ilustation about upper structure implementation)

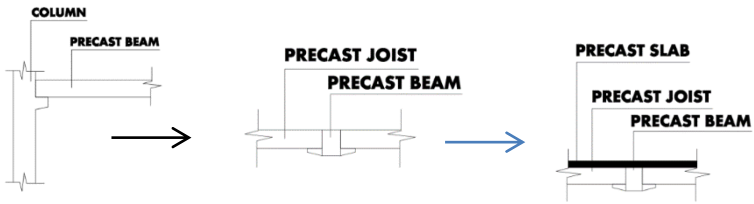


Figure 2.23. Ilustation about upper structure implementation (source: Private Documentation)

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CHAPTER III
METHODOLOGY

3.1. Flowchart of Methodology

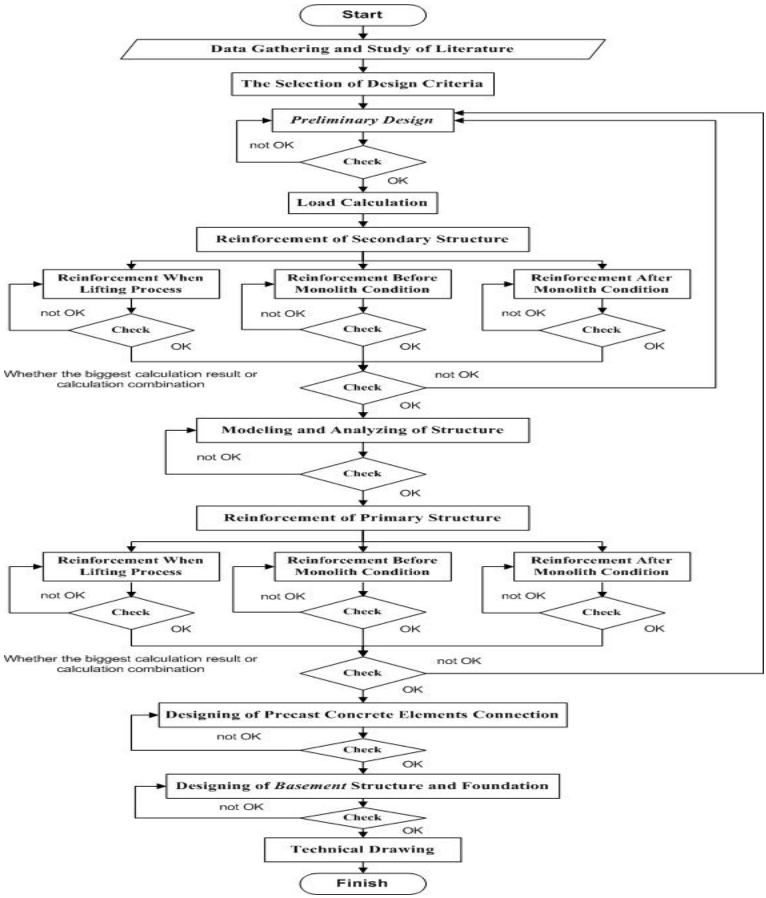

Figure 3.1. Flowchart of methodology
(source: Private Documentation)

3.2. Data Gathering and Study of Literature

As the following will be presented the data of the designing both of original data of designing or modified data of designing.

1. The original data of designing
   - building's name : Hotel Novotel THE SAMATOR Surabaya
   - location : Kedungbaruk Road no. 28 Surabaya
   - building's function : hotel
   - number of storey : 25 stories and 2 stories of basement
   - main stucture : cast in place concrete
   - material's quality : cast in place concrete with compressive strength f'_c 30 MPa and bar with tensile strength f_y 390 MPa
2. The modified data of designing
   - building's name : Hotel Novotel THE SAMATOR Surabaya
   - location : Kedungbaruk Road no. 28 Surabaya
   - building's function : hotel
   - number of storey : 10 storeis and 1 storey of basement
   - main stucture : precast concrete, except for basement structure, pillar, and shear wall
   - material's quality : precast concrete with compressive strength f'_c 30 MPa and bar with tensile strength f_y 390 MPa

For study of literature will use some literatures as the following

- Badan Standarisasi Nasional. 2013. Tata Cara Perhitungan Struktur Beton Untuk Bangunan Gedung (SNI 2847:2013)
- Badan Standarisasi Nasional. 2002. Tata Cara Perhitungan Struktur Beton Untuk Bangunan Gedung (SNI 2847:2002)
- Badan Standarisasi Nasional. 2013. Tata Cara Perancangan Beton Pracetak dan Beton Prategang Untuk Bangunan Gedung (SNI 7833:2012)
- Badan Standarisasi Nasional. 2012. Tata Cara Perencanaan Ketahanan Gempa Untuk Struktur Bangunan Gedung dan Non Gedung (SNI 1726:2012)
- Badan Standarisasi Nasional. 2013. Beban Minimum Untuk Perancangan Bangunan Gedung dan Struktur Lain (SNI 1727:2013)
- Departemen Pekerjaan Umum. 1983. Peraturan Pembebanan Indonesia Untuk Gedung (PPIUG 1983)
- Departemen Pekerjaan Umum dan Tenaga Listrik. 1971. Peraturan Beton Bertulang Indonesia (PBBI 1971)
- PCI Design Handbook : Precast and Prestressed Concrete 7th Edition (PCI 2004)

3.3. The Selection of Design Criteria

For the selection of design criteria will follow the rules in SNI 1726:2012.Some of the selection's aspects include as the following: strength, service ability, and structure system. Furthermore it also should be considered about site class, risk category, seismic design category, combination of connector system, etc. For seismic design category can be seen on Table 3.1. for seismic design category based on acceleration respond parameter at short period, S_s, and Table 3.2 for seismic design category based on acceleration respond parameter at one second period, S_1.

Table 3.1. Seismic Design Category Based On Acceleration Respond Parameter at Short Period, S_s (source: SNI, 2012)

S_{DS} **Value**	**Risk category**	
	I or II or III	**IV**
$S_{DS} < 0,167$	A	A
$0,167 \leq S_{DS} < 0,33$	B	C
$0,33 \leq S_{DS} < 0,50$	C	D
$0,50 < S_{DS}$	D	D

Table 3.2. Seismic Design Category Based On Acceleration Respond Parameter at One Second Period, S_1 (source: SNI, 2012)

S_{D1} **Value**	**Risk category**	
	I or II or III	**IV**
$S_{D1} < 0,167$	A	A
$0,067 \leq S_{D1} < 0,133$	B	C
$0,133 \leq S_{D1} < 0,20$	C	D
$0,20 < S_{D1}$	D	D

To determine whichever the structure belongs to risk category I, II, III, or IV can be seen in SNI (2012) Table 1.

3.4. *Preliminary Design*

In this chapter, *preliminary design* will determine initial design for structure's elements resisting frame. Then the result of the initial design will be used for next calculation.

3.4.1. Designing of Slab Dimension

The steps for designing of slab dimension are the following

1. determining whichever it is one way slab or two ways slab. The step is by dividing Ly by Lx (Figure 3.2). If the calculation's result produces the value which greater or same as two, then the slab can be determined as one way slab. But if the calculation's result produces the value which less than two, then the slab can be determined as two ways slab

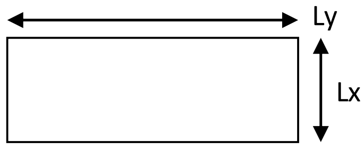

Figure 3.2. Ilustration of slab
(source: Private Documentation)

2. determining the minimum thickness of slab, if the slab is one way slab (non prestressed) then the slab's thickness must follows the conditions in SNI (2013) Table 9.5(a). If the slab is two ways slab (non prestressed) then the slab's thickness must follow the coditions in SNI (2013) Table 9.5(c)

3. checking the slab's thickness based on the conditions in SNI 2847:2013.

a. according to SNI (2013) paragraph 9.5.3.2 states that for slab without interior beam which reaches between two supports and have the ratio of long span to short span which less than 2, the minimum thickness shall meet the conditions shown in SNI (2013) Table 9.5(c) and must not less than the following:

- for slab without panel drop (with thickening) = 125 mm
- for slab with panel drop (without thickening) = 100 mm

b. for α_{fm} which has same value or less than 0,2, shall follow condition in SNI (2013) paragraph 9.5.3.2

c. for α_{fm} which has greater value than 0,2 but not more than 2,0, the minimum thickness of slab must not less than

$$h = \frac{\ell_n \left(0{,}8 + \frac{f_y}{1400}\right)}{36 + 5\beta\left(\alpha_{fm} - 0{,}2\right)}$$ (source: SNI, 2013)

and must not less than 125 mm

d. for α_{fm} which has greater value than 2,0, the minimum thickness of slab must not less than

$$h = \frac{\ell_n \left(0{,}8 + \frac{f_y}{1400}\right)}{36 + 9\beta}$$ (source: SNI, 2013)

and must not less than 90 mm

in which α_{fm} is an average value of α_f for all beam which at the edge of a panel.

3.4.2. Designing of Beam Dimension

According to SNI (2013) Table 9.5(a) states that the minimum thickness of non prestressed beam or one way slab if the deflection not calculated shall meet the conditions shown below

Table 3.3. The Minimum Thickness of Structure Slab and/or Beam (source: SNI, 2013)

<table>
<tr><td colspan="5">Minimum thickness, h</td></tr>
<tr><td rowspan="2">Structure's component</td><td>Simple supported</td><td>One end continuously</td><td>Both ends continuously</td><td>Cantilever</td></tr>
<tr><td colspan="4">The structure's component which does not support or not connected by partition or other construction which may be damaged by large deflection</td></tr>
<tr><td>One way massive slab</td><td>$\boldsymbol{\ell/20}$</td><td>$\boldsymbol{\ell/24}$</td><td>$\boldsymbol{\ell/28}$</td><td>$\boldsymbol{\ell/10}$</td></tr>
<tr><td>Beam or one way rib slab</td><td>$\boldsymbol{\ell/16}$</td><td>$\boldsymbol{\ell/18.5}$</td><td>$\boldsymbol{\ell/21}$</td><td>$\boldsymbol{\ell/8}$</td></tr>
<tr><td colspan="5"><u>NOTE:</u>
The length of span is in mm.
The values above shall be used directly by structure's component with normal concrete and bar's quality of 420 MPa. For other condition, the value above shall be modified as the following:
(a) For lightweight concrete structure with density (equilibrium density), $\boldsymbol{w_c}$, between 1440 and 1840 kg/m^3, the value shall be multiplied by (1,65-0,0003$\boldsymbol{w_c}$) but must not less than 1,09.
(b) For `besides 420 MPa, the value shall be multiplied by (0,4 + $\boldsymbol{f_y}$/700)</td></tr>
</table>

3.4.3. Designing of Pillar Dimension

According to SNI (2013) paragraph 9.3.2.2 (a), the value of strength reduction factor, **ϕ**, for compressive controlled section for reinforced structure component is 0,65. For determining the pillar dimension can use equation below as the following

$$A = \frac{W}{\phi\, x\, f'c}$$ (source: SNI, 2013)

in which A is the area of pillar (mm^2), $f'c$ is compressive strength of concrete (MPa), and W is axial load which received by pillar (N).

3.4.4. Designing of Stair Dimension

For designing of stair it is assumed that the stair's support which used is simple support roller-pins and using cast in place concrete. For designing of stair dimension should meet the following criteria

- $64 \leq 2t + i \leq 65$
- for the conditions of stair's slope is $20^0 \leq \alpha \leq 40^0$

in which t issteps stamping height (in cm), i is steps stamping width (in cm), and α is stair's slope. For designing of landing dimension *(bordes)*, it is suggested that the landing width must not less than 120 cm.

3.4.5. Designing of Shear Wall Dimension

For designing of shear wall dimension shall meet the conditions shown in SNI (2013) paragraph 14.5.3.1 and paragraph 14.5.3.2 which state that the thickness of support wall (or shear wall) must not less than 1/25 of height or the length of supported span, whichever is shorter, or less than 100 mm.

3.5. Load Calculation

For load calculation, there are several loads including dead load, live load, earthquake load, and wind load. The conditions for load designing shall meet the conditions shown in PPIUG (1983) and SNI 1727:2013.

3.5.1. Dead Load

According to PPIUG (1983) paragraph 2.1.(1) states that the self load from construction materials are important and some of building's components that have to be considered in order to determine the dead load of the building shall meet the conditions shown onTable 2.1 PPIUG (1983). As the following will be presented Table 3.4 which shows some of the values of dead loads on the building. For the value of reduction factor of dead load on the building's structure which is 0.9 can be neglected (PPIUG, 1983).

Table 3.4. Some of Dead Loads Values on Building Structure (source: PPIUG, 1983)

Dead load	The value of load
Materials	
Natural stone	2600 kg/m^3
Smashed stone	1450 kg/m^3
Concrete	2200 kg/m^3
Reinforced concrete	2400 kg/m^3
Gravel	1650 kg/m^3
Sand (water saturated)	1800 kg/m^3
Building's Components	
Mortar (mixture), per cm of thickness (from cement)	21 kg/m^2
Wall built from red brick (1/2 brick)	250 kg/m^2
Ceiling and wall (including its flanks, without ceiling hanger or bracing), consist of asbestos cement and glass with 3-4 mm thickness	11 kg/m^2 (asbestos cement) + 10 kg/m^2 (glass)
Ceiling hanger	7 kg/m^2
Floor covering made from portland cement tile, teraso, and concrete, without mortar, per cm	24 kg/m^2

3.5.2. Live Load

According to SNI (2013) paragraph 4.7.1 states that exceptfor other minimum prevalent distributed live load, L_o, shown inSNI (2013) Table 4-1 can be reduced based on conditions shown in paragraph4.7.2 until paragraph 4.7.6 SNI 1727:2013. Follow the limitation and exception from paragraph 4.7.3 until paragraph 4.7.6 SNI 1727:2013, the structure's components which have $K_{LL}A_T$value are 400 ft^2area (37,16 m^2) or more allowed to be designed with reduced live load based on equation below.

$$L = L_o\left(0{,}25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right) \text{(source: SNI, 2013)}$$

in which L is reduced planning live load per $m^2$from area which supported by structure's component (in kN/m^2), L_o is non-reduced planning live load per $m^2$from area which supported by structure's component (in kN/m^2), K_{LL} is live load element, and A_T is tributari area in m^2. For some of L_o values and K_{LL} values can be seen on Table 3.5 dan Table 3.6.

Table 3.5. Some of Minimum Distributed Live Loads Values, L_o (source: SNI, 2013)

For House or other functions	**The value of distributed load**
Stair and exit way	4,79 kN/m^2
House for living (one family and two families) for all room except stair and balcony	1,92 kN/m^2
Flat roof, and/or arch roof	0,96 kN/m^2
Balcony and deck	4,79 kN/m^2
Garage/Parking lot	1,92 kN/m^2
Public room and balcony	4,79 kN/m^2

Table 3.6. The Factor of Live Load Elements, K_{LL}
(source: SNI, 2013)

Element	K_{LL} Value
Interior pillars	4
Exterior pillars without cantilever slab	4
Periphery pillars with cantilever slab	3
Corner pillars with cantilever slab	2
Periphery pillars without cantilever slab	2
Interior beams	2
All of structure's components which not mentioned above	1

3.5.3. Earthquake Load (Seismic Load)

Surabaya is zone which has site class SE (soft soil) (see SNI 2012, Table 3). For the calculation of earthquake load will use dynamic analysis using structure fundamental tremble simplify design response spectrum chart based on SNI 1726:2012. The steps for plotting design response spectrum chart (for example of chart see Figure 3.5) are the following

a. determining site class

- site class is based on SNI (2012)paragraph 6.1.2 which divided into SA, SB, SC, SD, SE, or SF

b. determining the coefficients of site and the parameters of acceleration spectral response quake maximum that considered risk-targetted (MCE_R)

- determining the value F_a (vibration amplification factor related to acceleration in short period vibration)and the value of F_v (vibration amplification factor related to acceleration in one second period vibration), the value of F_a and $F_v$can be seen on SNI (2012) Table 4 dan Table 5

- determinig the value of S_s and S_1 (Figure 3.3 and Figure 3.4), for simplifying the calculation, the site classcan be assumed as SB site class

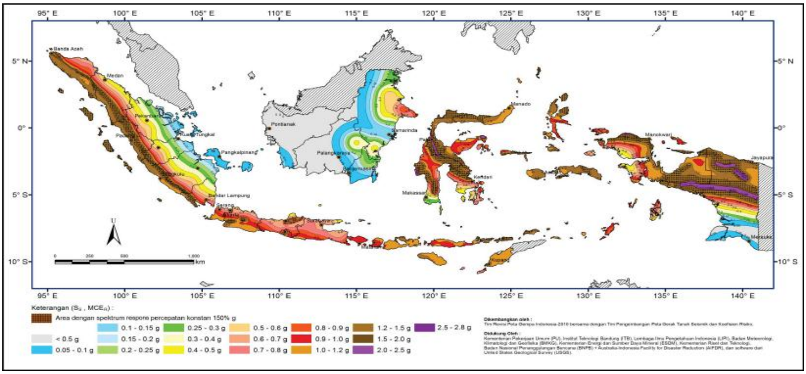

Figure 3.3. S_s, Seismic maximum that considered risk-targetted (MCE_R), SB site class (source: SNI, 2012)

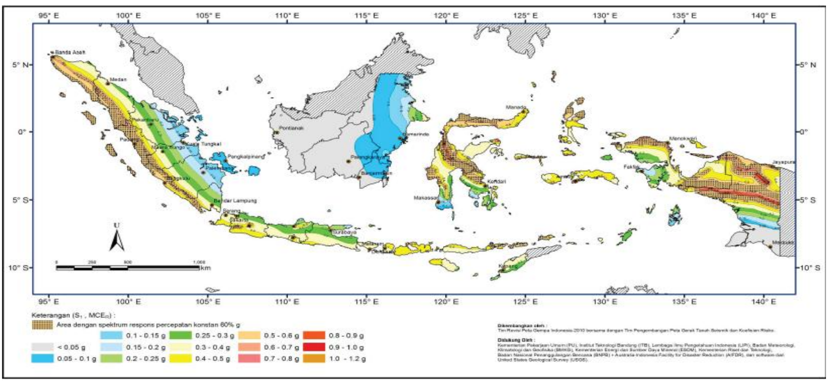

Figure 3.4. S_1, Seismic maximum that considered risk-targetted (MCE_R), SB site class (source: SNI, 2012)

- calculating the value of S_{MS} (acceleration response spectrum parameter in short period) and S_{M1} (acceleration response spectrum parameter in one second period) by equationsas the following
  $S_{MS} = F_a\ x\ S_S$ (source: SNI, 2012)
  $S_{M1} = F_v\ x\ S_1$ (source: SNI, 2012)

c. determining the design spectral acceleration parameter

- for S_{DS} value (design spectral acceleration parameter for short period) can be calculated using equation below as following
  $S_{DS} = 2/3\ x\ S_{MS}$ (source: SNI, 2012)
- for S_{DS} value (design spectral acceleration parameter for one second period) can be calculated using equation below as following
  $S_{D1} = 2/3\ x\ S_{M1}$ (source: SNI, 2012)

d. determining design response spectrum

- see SNI (2012) paragraph 6.4 for more detailinformation about design response spectrum. Equation below shows the way to calculate S_a (design acceleration response spectrum) for the period bigger than T_s, for other $S_a$value for smaller period than T_0or period between T_0 and T_s, seeSNI (2012) paragraph 6.4
  $S_a = \frac{S_{D1}}{T}$ (source: SNI, 2012)
  in whichT is structure fundamental vibration period, for T value can be calculated using SNI (2012) paragraph 7.8.2.1.

For calculating *base shear*, V, using equation below while calculating distribution of earthquake vertical force using equation 30 until 31 on SNI (2012)

$V = C_s x\, W$ (source: SNI, 2012)

in which C_s value is calculated usingequation 22 until 25 on SNI (2012), W is building effective seismic weight

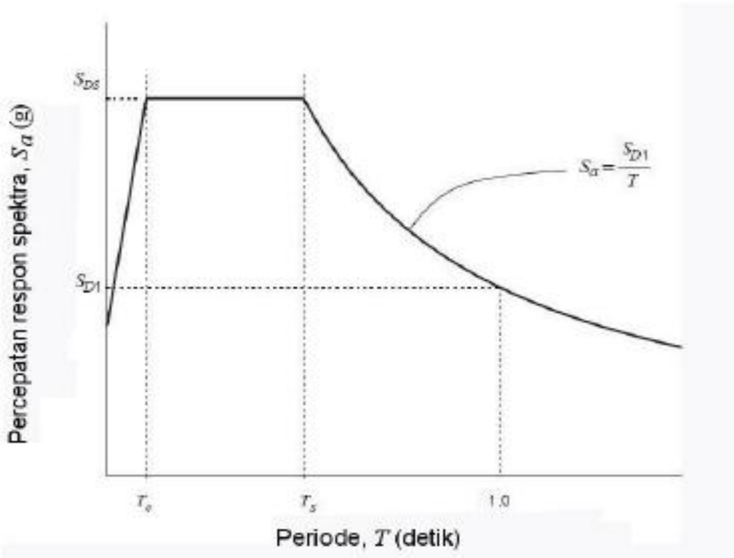

Figure 3.5. The example of design response spectrum chart (source: SNI, 2012)

3.5.4. Wind Load

Surabaya is the area located near the coast. PPIUG (1983) chapter 4explains about wind load which in PPIUG (1983) paragraph 4.2(2) states that inflatable pressure at sea and seaside until 5 km away from coast shall be taken minimum of 40 kg/m^2, except that determined inPPIUG (1983) verse (3) and (4) paragraph 4.2.

3.5.5. Load Combinations

In SNI (2013) paragraph 9.2.1 explains about load combination as the following

- $U = 1{,}4\ D$
- $U = 1{,}2\ D + 1{,}6\ L + 0{,}5\ (R \text{ or } L_r)$
- $U = 1{,}2\ D + 1{,}6\ (L_r \text{ or } R) + (1{,}0\ L \text{ or } 0{,}5\ W)$
- $U = 1{,}2\ D + 1{,}0\ W + 1{,}0\ L + 0{,}5\ (L_r \text{or } R)$
- $U = 1{,}2\ D + 1{,}0\ L \pm 1{,}0\ E$
- $U = 0{,}9\ D + 1{,}0\ W$
- $U = 0{,}9\ D + 1{,}0\ E$

in which D is dead load, L is live load, W is wind load, E is earthquake load (seismic load), $L_r$is roof load, R is rain load, and U is *ultimate* load.

3.6. Reinforcement of Secondary Structure

For reinforcement of secondary structure, it consists of reinforcement when lifting process, reinforcement before monolith condition, and reinforcement after monolith condition. The secondary structures are slab, secondary beam, and stair.

3.6.1. Reinforcement Due to Lifting Process

The lifting process is important process for precast concrete element. In this chapter the secondary structures which are lifted consists of slab and secondary beam. The reinforcement is needed due to load which retained by precast element reinforcement when lifting process occured.

3.6.1.1. Slab Element

When lifting process of slab element occurs, it should be considered about permit tension. The permit tension when lifting process of condition when *stripping* process, when *rotating* process, and when *storage* process use the assumption of 3 days concrete age. While the permit tension when lifting process of condition when *erection* and *turning* process use the assumption of 28 days concrete age. For calculating f'$_{ci}$and f'$_r$use equationsbelow

$f'_{ci} = coefficient\ x\ f'_c$ (source: PBBI, 1971)
$f'_r\ = 0{,}7\ x\sqrt{f'_{ci}}$ (source: PBBI, 1971)

For coefficient value can be seen in PBBI (1971) Table 4.1.4
For designing the reinforcement bar of slab element when lifting process shall be explained generally as the following, for more detail calculation shall be explained on another chapter

a. checking the permit tension when *stripping* process
$f_t \leq f'_r$ (source: PBBI, 1971)

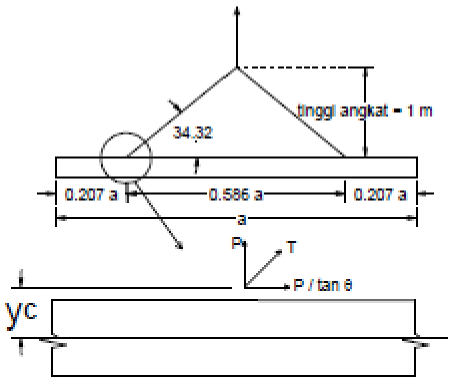

Figure 3.6. The position when lifting process of slab element occurs
(source: Private Documentation)

b. checking the permit tension when *turning* process
$f_a \leq f'_r$ (source: PBBI, 1971)
$f_b \leq f'_r$ (source: PBBI,1971)

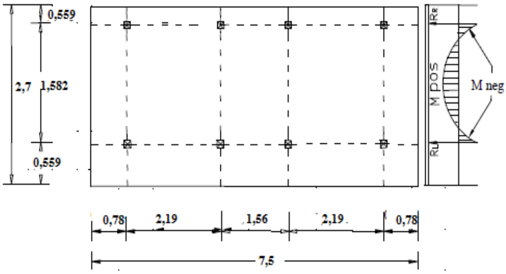

Figure 3.7. The example of lifting bar distance and moment calclulation for slab element (source: Private Documentation)

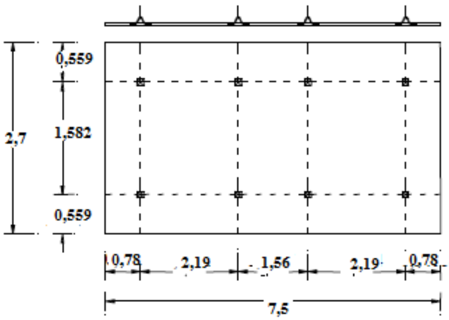

Figure 3.8. The example of result of lifting bar distance for slab element (source: Private Documentation)

c. calculating the strain cable

$P \leq F_{ijin}$ (source: PBBI, 1971)

d. calculating the reinforcement bar and shear connector

$A_{v\,min} \leq \frac{s\,x\,b_w}{3\,x\,f_y}$ (source: SNI, 2013)

3.6.1.2. Secondary Beam Element

Secondary beam element needs to be installed the lifting reinforcement bar so that it shall bear the force and the moment which are produced when lifting process ocurs. The steps to design the reinforcement bar generally shall be explained as following, for more detail calculation shall be explain on another chapter

a. calculating moment force when lifting process occurs

$$+M = \frac{WL^2}{8}\left(1 - 4x + \frac{4yc}{L\tan\theta}\right) \text{ (source: PCI, 2004)}$$

$$-M = \frac{WX^2L^2}{2} \text{ (source: PCI, 2010)}$$

b. calculating X (Figure 3.9 and Figure 3.10)

$$X = \frac{1+\frac{4yc}{L\tan\theta}}{2\left(1+\sqrt{1+\frac{Yt}{Yb}\left(1+\frac{4yc}{L\tan\theta}\right)}\right)} \text{ (source: PCI, 2004)}$$

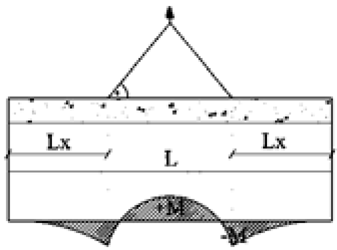

Figure 3.9. The position when lifting process of precast secondary beam occurs
(source: Private Documentation)

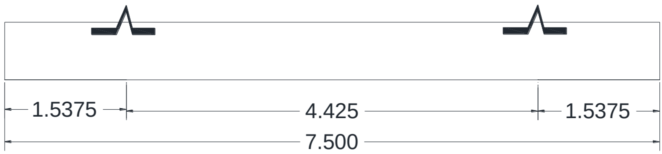

Figure 3.10. The example of result of lifting bar distance for secondary beam element (source: Private Documentation)

c. calculating the reinforcement bar for lifting process

$$\emptyset_{\text{lifting bar}} \geq \sqrt{\frac{Pu}{\sigma_{\text{permit}} \times \pi}} \text{(source: PBBI, 1971)}$$

3.6.2. Reinforcement Before Monolith Condition

The before monolith condition occurs when the *overtopping* concrete is not drying yet and it becomes load to precast concrete element (Figure 3.11 and Figure 3.12). The reinforced elements which will be explained on this chapter are slab, secondary beam, and stair.

3.6.2.1. Slab Element

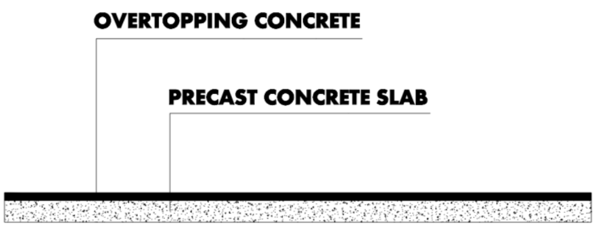

Figure 3.11. Condition of slab precast element before monolith condition (source: Private Documentation)

The reinforcement of slab includes designing and calculating process of flexural bar of slab, shrinkage bar of slab, divider bar of slab, the length of hook development, and deflection control and crack of slab. For designing and calculating process of flexural bar of slab shall be explained as the following

a. determining the data for designing slab (f'_c, f_y, m, b, dx, Mu)

- the design criteria for flexural is $\phi Mn \geq Mu$, the value of ϕ shall meet the condition shown in SNI (2013) paragraph 9.3, for clear cover see SNI (2013) paragraph 7.7.1
- for slab's thickness shall meet the condition shown in SNI (2013) Table 9.5(a) and SNI (2013) Table 9.5(c), and SNI (2013) paragraph 9.5.3.3
- the value of Mu shall be calculated based on load calculation result

b.calculating $\rho_b$and bar ratio ρ_{min}

- for calculating ρ_b using equation $\rho_b = \frac{0{,}85\beta_1 f_c'}{f_y}\left(\frac{600}{600+f_y}\right)$ (source: SNI, 2013)in which $\rho_b$is the ratio of A_s to bd which produces balance strain *(see SNI, 2013 Enclosure B)*, $\beta_1$is factor which relates the height of equivalent square compressive stress block with the height of neutral axis *(the value of β_1 shall meet the condition in SNI, 2013 paragraph 10.2.7.3)*
- for calculating the bar ratio ρ_{min}, its value must not be greater than 0,025 *(SNI, 2013 paragraph 21.5.2.1)*, it must not be greater than 0,75 ρ_b*(SNI, 2013 paragraph B.10.3.3)*, and it must not less than $0{,}25\sqrt{f_c'}/f_y$. The value of $\rho_{min}$is taken, whichever the biggest calculation result, as the decisive value

c. calculating the required bar ratio, ρ_{need}

- calculating the value of m using equation below
  $m = \frac{f_y}{0{,}85\ x\ f'_c}$ (source: SNI, 2013)
- calculating the value of R_n using equation below
  $R_n = \frac{M_u}{\emptyset\ x\ b\ x\ d^2}$ (source: SNI, 2013)
  in which the value of ϕ is taken as big as 0.75 *(see SNI, 2013 paragraph 9.3.2.2 and SNI, 2013 Figure S9.3.2)*
- calculating the value of ρ_{need} using equation below

  $$\rho_{perlu} = \frac{1}{m}(1 - \sqrt{1 - \frac{2\ x\ m\ x\ R_n}{f_y}})$$ (source: SNI, 2013)

d. calculating the required bar area, A_s,
and the number of bar, n

- calculating $A_s$and $n$using equations below (n is the number of bar)
  $A_s = \rho_{perlu} x\ b\ x\ d$ (source: SNI, 2013)
  $n = \frac{A_s}{A\ tulangan}$ (source: SNI, 2013)

e. calculating the space between bars, *s*

- in SNI (2013) paragraph 7.6.5 states that in wall and slab except from concrete secondary beam construction, the main flexural bars shall have spacing not more than thrice of wall's or slab's thickness, or not more than 450 mm

For designing of shrinkage bar shall meet the conditions in SNI (2013) paragraph 7.12.2.1 which states that the area of shrinkage bar shall provide at least having the ratio of bar area to concrete section gross area as the following, but not less than 0,0014

- slab which using spiral bar or welded wire bar with quality 420 MPa should uses minimum ratio value of 0,0018

and the shrinkage bar and temperature bar shall be assembled with spacing not more than five times of slab's thickness, or not more than 450 mm *(see SNI, 2013 paragraph 7.12.2.2)*. For divider bar shall use equation below

$$A_s = \frac{0{,}18\, x\, b\, x\, \mathrm{h}}{100} \text{ (source: SNI, 2013)}$$

in which A_s is the area of bar (note: the spacing between the bar is calculated too)

For the length of hook development shall follow the condition in SNI (2013) paragraph 12.5.1. For checking of deflection shall follow the condition in SNI (2013) paragraph 9.5.3.1 and Table 9.5(b) while for checking of crack of slab shall use the condition in SNI (2013) paragraph 10.6.4 which states that the spacing of nearest bar to tensile surface, s, must not be greater than the value which calculated using equation below

$$s = 380\left(\frac{280}{f_s}\right) - 2{,}5c_c \text{ (source: SNI, 2013)}$$

in which the value of s must not be greater than 300 x (280/f_s), and the value of c_c is the shortest distance from the surface of bar or prestressed steel to tensile surface. If there is only one reinforcement bar or the nearest wire to furthest surface, then the value of *s* shall use the furthest tensile surface width. The stress of bar, f_s, may be taken as 2/3 f_y.

3.6.2.2. Secondary Beam Element

The reinforcement of secondary beam includes the designing and calculating of flexural bar of secondary beam, the transversal bar of secondary beam, the development of bar, the discontinuation of bar, and checking for torsion. For designing and calculating process of reinforcement bar of secondary beam shall be explained on the next chapter. For designing and calculating process of reinforcement bar of lift beam should be same as secondary beam and/or primary beam.

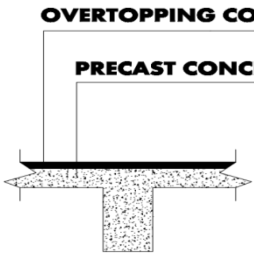

Figure 3.12. Condition of secondary beam precast element before monolith condition
(source: Private Documentation)

3.6.2.3. Stair Element

For reinforcement of stair should be calculated working load in stair first. Then it can be obtained the value of reaction and moment that worked in stair by assuming the support as pins-roller support. For reinforcement of stair, reinforcement of landing plate *(pelat bordes)*, reinforcement of *bordes* beam, the calculation processes for all of them are same as steps and calculation of beam and slab. The stair element does not have after/before monolith condition because it is made from *in-situ concrete*.

3.6.3. Reinforcement After Monolith Condition

The after monolith condition occurs when the *overtopping* concrete has dried completely and it bears load together with precast concrete element (Figure 3.13 and Figure 3.14). The reinforced elements which will be explained on this chapter are slab and secondary beam.

3.6.3.1. Slab Element

The designing process of reinforcement of slab element after monolith condition occurs shall be same as previous chapter which already described earlier. Only there are some differences in calculation process which shall be explained later.

Figure 3.13. Condition of slab precast element after monolith condition (source: Private Documentation)

3.6.3.2. Secondary Beam Element

The reinforcement of secondary beam includes the designing and calculating of flexural bar of secondary beam, the transversal bar of secondary beam, the development of bar, the discontinuation of bar, and checking for torsion. For designing and calculating process of reinforcement bar of secondary beam shall be explained on the next chapter. The designing process of reinforcement of secondary beam element after monolith condition occurs shall be same as designing process of reinforcement of secondary beam element before monolith condition occurs. Only there are some differences in calculation process which shall be explained later.

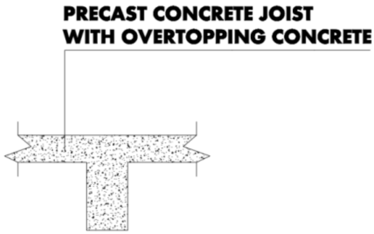

Figure 3.14. Condition of secondary beam precast element after monolith condition
(source: Private Documentation)

3.7. Modeling and Analyzing of Structure

For modeling and analyzing of structure will use SAP 2000® programme. Structure will be made like model based on preliminary design and loads. Then it will proceed to running process in order to know the result of structure analysis.

3.8. Reinforcement of Primary Structure

For reinforcement of primary structure, it consists of reinforcement when lifting process, reinforcement before monolith condition, and reinforcement after monolith condition. The primary structures are primary beam, pillar, and shear wall.

3.8.1. Reinforcement Due to Lifting Process

The lifting process is important process for precast concrete element. In this chapter the primary structures which are lifted consists of primary beam only. The reinforcement is needed due to load which retained by precast element reinforcement when lifting process occurred.

3.8.1.1. Primary Beam Element

Primary beam element needs to be installed the lifting reinforcement bar so that it shall bear the force and the moment which are produced when lifting process ocurs. The steps to design the reinforcement bar generally shall be explained as following, for more detail calculation shall be explain on another chapter

a. calculating moment force when lifting process occurs

$$+M = \frac{WL^2}{8}\left(1 - 4x + \frac{4yc}{L\tan\theta}\right) \text{ (source: PCI, 2004)}$$

$$-M = \frac{WX^2L^2}{2} \text{ (source: PCI, 2004)}$$

b. calculating X (see Figure 3.15and Figure 3.16)

$$X = \frac{1+\frac{4yc}{L\tan\theta}}{2\left(1+\sqrt{1+\frac{Yt}{Yb}\left(1+\frac{4yc}{L\tan\theta}\right)}\right)} \text{ (source: PCI, 2004)}$$

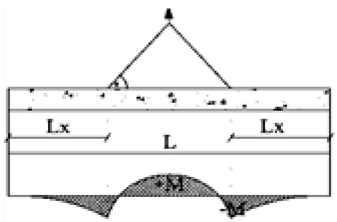

Figure 3.15. The position when lifting process of precast primary beam occurs
(source: Private Documentation)

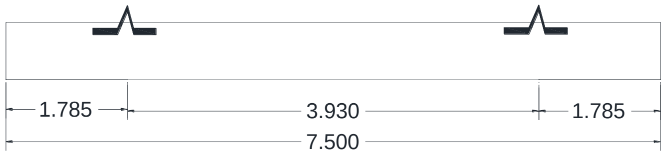

Figure 3.16. The example of result of lifting bar distance for primary beam element (source: Private Documentation)

c. calculating the reinforcement bar for lifting process

$$\varnothing_{\text{lifting bar}} \geq \sqrt{\frac{Pu}{\sigma_{permit} \times \pi}} \text{ (source: PBBI, 1971)}$$

3.8.2. Reinforcement Before Monolith Condition

The before monolith condition occurs when the *overtopping* concrete is not drying yet and it becomes load to precast concrete element (Figure 3.17). The reinforced elements which will be explained on this chapter are primary beam, pillar, and shear wall.

3.8.2.1. Primary Beam Element

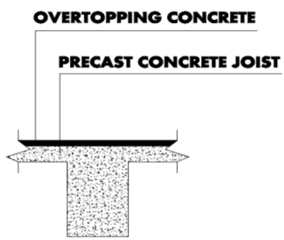

Figure 3.17. Condition of primary beam precast element before monolith condition (source: Private Documentation)

The reinforcement of primary beam includes the designing and calculating of flexural bar of primary beam, the transversal bar of primary beam, the development of bar, the discontinuation of bar, and checking for torsion. For designing and calculating process of flexural bar of primary beam shall be explained as the following

a. determining the data for designing primary beam (f'$_c$, f_y, m, b, dx, Mu)

- the design criteria for flexural is ϕMn ≥ Mu, the value of ϕ shall meet the condition shown in SNI (2013) paragraph 9.3, for clear cover see SNI (2013) paragraph 7.7.1
- the value of Mu shall be calculated based on load calculation result

b.calculating ρ_b and bar ratio ρ_{min}

- for calculating ρ_b using equation below
  $\rho_b = \frac{0,85\beta_1 f_c'}{f_y}\left(\frac{600}{600+f_y}\right)$(source: SNI, 2013)
  in which ρ_b is the ratio of A_s to *bd* which produces balance strain *(see SNI, 2013 Enclosure B)*, β_1 is factor which relates the height of equivalent square compressive stress block with the height of neutral axis *(the value of β_1 shall meet the condition in SNI, 2013 paragraph 10.2.7.3)*
- for calculating the bar ratio ρ_{min} , its value must not be greater than 0,025 *(SNI, 2013 paragraph 21.5.2.1)*, it must not be greater than 0,75 ρ_b *(SNI, 2013 paragraph B.10.3.3)*, and it must not less than $0.25\sqrt{f_c'}/f_y$. The value of ρ_{min} is taken, whichever the biggest calculation result, as the decisive value

c. calculating the required bar ratio, ρ_{need}

- calculating the value of *m* using equation below

$m = \frac{f_y}{0,85\ x\ f_c'}$ (source: SNI, 2013)

- calculating the value of R_n using equation below
  $R_n = \frac{M_u}{\emptyset\ x\ b\ x\ d^2}$ (source: SNI, 2013)
  in which the value of ϕ is taken as big as 0,75 *(see SNI, 2013 paragraph 9.3.2.2 and SNI, 2013 Figure S9.3.2)*
- calculating the value of ρ_{need} using equation below

$\rho_{perlu} = \frac{1}{m}(1 - \sqrt{1 - \frac{2\ x\ m\ x\ R_n}{f_y}})$ (source: SNI, 2013)

d. calculating the required bar area, A_s,
and the number of bar, n

- calculating $A_s$and $n$using equations below
  $A_s = \rho_{perlu} x\ b\ x\ d$ (source: SNI, 2013)
  $n = \frac{A_s}{A\ tulangan}$ (source: SNI, 2013)

e. calculating the clear spacing between bars, s

- calculating s using equation below
  $s = \frac{b - n \times \emptyset L - 2d' - 2\emptyset S}{n-1}$ (source: SNI, 2013)
  in which Ø is the diameter of bar

For designing the transversal bar of primary beam shall follow the condition in SNI (2013) paragraph 11 and should consider the things as the following

a. calculating V_c and V_s

- calculating $V_c$using equation below
  $V_c = 0,17\,\lambda \sqrt{f_c'}$ bw d (source: SNI, 2013)
- calculatingV_s usingequationbelow
  $V_s \leq 0,66 \sqrt{f_c'}$ bw d (source: SNI, 2013)

b. calculating V_n and checking V_n

- calculating $V_n$using equation below
  $V_n = V_c + V_s$ (source: SNI, 2013)

- checking $V_n$using equation below
  $\phi\ V_n \geq V_u$ (source: SNI, 2013)

c. calculating the minimum requirement of transversal bar based on the condition in SNI (2013) paragraph 11.4.6. The spacing between transversal bar shall be calculated using equation below

$s = \frac{Av \times fy \times d}{Vs}$ (source: SNI, 2013)

and it must not less than $d/4$, $16D$, and 150 mm *(see SNI, 2013 paragraph 21.5.3.2).* For the development of bar shall follow the condition in SNI (2013) paragraph 12.2 until 12.3 and 12.5.

For the discontinuation of bar shall follow the condition in SNI (2013) paragraph 12.12.3 which states that at least a third of total tensile bar which is assembled for negative moment on the support shall has attached/hook length which passing inflection point less than d, $12d_b$, or $\ell_n/16$, whichever is greater value. For checking torsion uses the condition in SNI (2013) paragraph 11.5.1(a). When checking torsion does not meet the condition in the paragraph, then torsion bar shall be assembled. But when checking torsion does meet the condition in the paragraph, then torsion influence can be neglected.

For reinforcement in primary beam on edge shall consider about using *reinforced concrete bearing* if needed (PCI, 2004).

3.8.2.2. Pillar Element

For reinforcement of pillar shall consider the conditions below

a. for calculating the minimum flexural strength of [pillar, it shall meet the condition in SNI (2013) paragraph 21.6.2.1, paragraph 21.6.2.2, and paragraph 21.6.2.3 (for the figure of design of shear for primary beam and pillar see Figure 3.18)

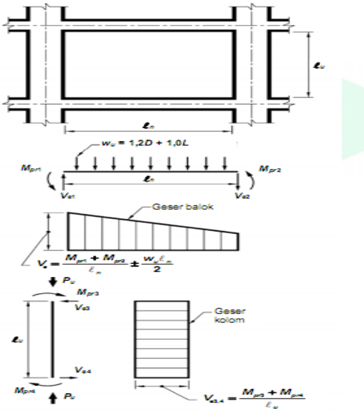

Figure 3.18. Design of shear for primary beam and pillar (source: SNI, 2013)

<u>NOTE for Figure 3.18:</u>

- the direction of shear force V_e depends on gravity load relative size and shear which produced by edge moments
- edge moments (probable moment), M_{pr}, value based on tensile stress of steel bar as big 1,25f_y is assigned yield strength (both of edge moments shall be review in both ways, clockwise or counterclockwise)
- edge moment (probable moment), M_{pr}, value for pillar must not be greater than moments which produced by primary beams M_{pr} value which frame into pillar-beam *joint*, V_e must not less than the value which required by structure analysis

b. for calculating longitudinal bar shall follow the condition in SNI (2013) paragraph 21.6.3. In SNI (2013) paragraph 21.6.3.1 states that the area of longitudinal bar, A_{st}, must not less than 0,01A_g or greater than 0,06A_g. In SNI (2013) paragraph 21.6.3.3 states that the connection throughput allowed only in a half of center length of structure's component, shall be designed as tensile connection throughput, and shall be surrounded in transversal bar which fullfil the conditions in SNI (2013) paragraph 21.6.4.2 and paragraph 21.6.4.3

c. for the length of spiral bar's development and spiral wire in tensile condition (see verse b above) shall follow the conditions in SNI (2013) paragraph 12.2.2 and paragraph 12.2.3 (see equation below)

$$\ell_d = \frac{f_y}{1{,}1\lambda\sqrt{f_c'}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} (d_b) \text{ (source: SNI, 2013)}$$

in which the segment resistant $(c_b + K_{tr})/d_b$ must not greater than 2,5 and the value of K_{tr} is

$$K_{tr} = \frac{40 A_{tr}}{sn} \text{ (source: SNI, 2013)}$$

in which n is the number of bars or wires which connected or distributed along furcation area. It is allowed to take $K_{tr} = 0$ as the simplification of design although transversal bar be found
d. for calculating the need of pillar transversal bar should follow the condition in SNI (2013) paragraph 21.6.4 (Figure 3.19)

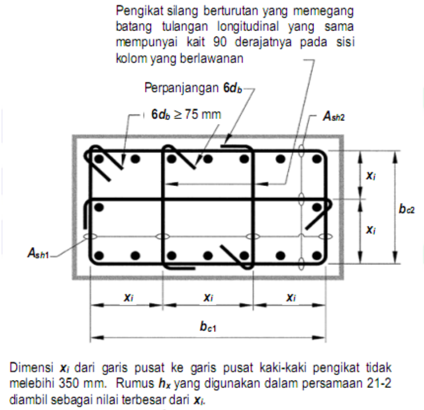

Figure 3.19. The example of transversal reinforcement in pillar (source: SNI, 2013)

3.8.2.3. Reinforcement of Shear Wall

For reinforcement of shear wall, it is assumed that it only bear shear force load. For reinforcement of shear wall shall consider the things as the following *(note that the pillar and shear wall element do not have after/before monolith condition because it is made from in-situ concrete)*

a. checking V_n

- according to SNI 2847:2013 paragraph 11.9.3 states that V_n in horizontal section for shear in wall section must not greater than $0{,}83\sqrt{f'_c}\,hd$, where h is wall's thickness, and d defined as in paragraph 11.9.4 SNI 2847:2013

b. checking bar layer

- according to SNI 2847:2013 page 197 paragraph 21.9.2.2 states that at least two layers of bar shall be used in a wall if V_u is greater than $0{,}17A_{cv}\lambda\sqrt{f'_c}$

c. calculating shear reinforcement

- for shear reinforcement shall follow the condition in SNI 2847:2013 paragraph 11.9.9, previously for calculating shear strength shall consider the condition in SNI 2847:2013 paragraph 21.9.4.

3.8.3. Reinforcement After Monolith Condition

The after monolith condition occurs when the *overtopping* concrete has dried completely and it bears load together with precast concrete element (Figure 3.20). The reinforced elements which will be explained on this chapter are only primary beam.

3.8.3.1. Primary Beam Element

The designing process of reinforcement of primary beam element after monolith condition occurs shall be same as designing process of reinforcement of secondary beam element before monolith condition occurs. Only there are some differences in calculation process which shall be explained later.

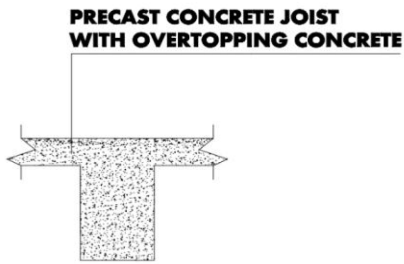

Figure 3.20. Condition of primary beam precast element after monolith condition
(source: Private Documentation)

3.9. Designing of Precast Concrete Elements Connection

For designing of precast concrete elements connection it consists of the connection between precast concrete primary beam and pillar, the connection between precast concrete primary beam and precast concrete secondary beam, and the connection between precast concrete primary beam and precast concrete slab.

a. The connection between precast concrete primary beam and pillar

For the connection between precast concrete primary beam and pillar will use wet connection which the precast concrete primary beam will be supported by the short console on pillar (see Figure 3.21).

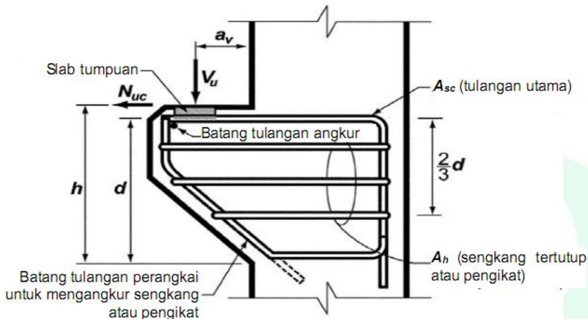

Figure 3.21. Short console on pillar
(source: SNI, 2013)

For the calculation of short console on pillar shall follow the condition in SNI (2013) paragraph 11.8. The length of development of primary beam's bar shall be considered because it will be used as hook. For the length of development of primary beam's bar shall follow the conditions in SNI (2013) paragraph 12.3 and paragraph 12.5. For the connection between precast concrete primary beam and pillar shall follow the condition in SNI (2013) paragraph 21.8.

b. The connection between precast concrete primary beam and precast concrete secondary beam

For the connection between precast concrete primary beam and precast concrete secondary beam will use wet connection which the secondary beam is supported by short console on primary beam (seeFigure 3.22).

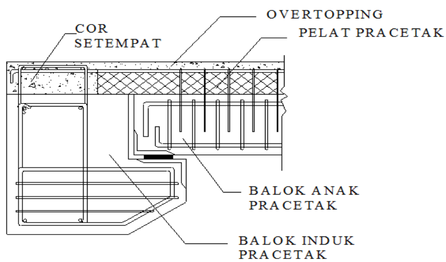

Figure 3.22. The configuration of connection between precast primary beam and precast secondary beam (source: Private Documentation)

For the calculation of short console on primary beam shall follow the condition in SNI (2013) paragraph 11.8. For the length of development of secondary beam's bar shall be considered because it will be used as hook. For the length of development of secondary beam's bar shall follow the condition in SNI (2013) paragraph 12.3 andparagraph 12.5.

c. The connection between precast concrete primary beam and precast concrete slab

For the connection between precast concrete primary beam and precast concrete slab will use slab's studs and *overtopping* which is made of concrete (Figure 3.23). The thickness of *overtopping*is usually between 50-100 mm.

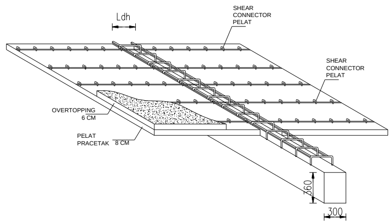

Figure 3.23. The configuration of connection between precast secondary beam and precast slab (source: Private Documentation)

For the calculation of slab's stud shall follow the condition in SNI (2013) paragraph 17.5 and 17.6. For the length of development of secondary beam's bar shall be considered because it will be used as hook. For the length of development of secondary beam's bar shall follow the condition in SNI (2013) paragraph 12.3 and paragraph 12.5 and consider the value of A_{vmin}*(see SNI, 2013 paragraph 11.4.6.3)*. For the development of precast slab's bar shall follow the conditions in SNI (2013) paragraph 12.2 until 12.3 and 12.5. And the requirement of steel bar for bearing friction shear force is calculated using equation below

$A_{vf} = \frac{Vn}{fy.\mu} \geq A_{vf}\,min$ (source: SNI, 2013).

3.9. Designing of Basement's Structure and Foundation

For basement structure the designing process of its elements should be same as slab, secondary beam, primary beam, pillar, and shear wall which have been described in previous chapters. For more detail calculation and designing shall be explained on another chapter (basement structure's elements use in-situ concrete). For foundation's design shall use pile foundation made of precast concrete.

Some of equations which will be used for designing of pile foundation are the following

a. calculating the bearing capacity of pile foundation

- the bearing capacity of pile foundation is calculated using equations belowwhich are introduced by Luciano Decourt (1982) as the following

  $Q_L = Q_S + Q_P$

  (source: Wahyudi, 1999)

  $Q_P = q_P \bullet A_P = (N_P \bullet \mathrm{K}) \bullet A_P$

  (source: Wahyudi, 1999)

  $Q_S = q_S \bullet A_S = \left(\frac{N_S}{3} + 1\right) \bullet A_S$

  (source: Wahyudi, 1999)

in which Q_L is maximum bearing capacity of soil on foundation, Q_P is ultimate resistance in foundation base, Q_S is ultimate resistance due to soil lateral force, N_P is average value of SPT, between 4B above until 4B belowpile foundation base (B = diameter of foundation) = $\sum_{i=1}^{n} Ni/n$, K is characteristic coefficient of soil for silt sand soil as big as 35 t/m^2, for clay sand soil as big as 30 t/m^2, for sand clay soil as big as 22 t/m^2, for silt clay soil as big as 15 t/m^2, A_P is section area of foundation base, q_P is stress in foundation edge, q_S is stress due to lateral attachement in t/m^2, N_S is average value of N along the foundation which attached with limitation of $3 \leq \mathrm{N} \leq 50$, A_S is perimeter x attached foundation's length (surface areaof foundation)

b. calculating the bearing capacity of piles foundation group

- the bearing capacity of piles foundation group is calculated using equation below as following

  $Q_L\,(group) = Q_L(1\,pile)\ x\ n\ x\ C_e$

  (source: Wahyudi, 1999)

in which n is the number of piles in a group. The value of C_e is calculated using equation below which introduced by

Converse-Labarreas following

$$C_e = 1 - \frac{arc\tan(\frac{\emptyset}{S})}{90^0} \bullet \left(2 - \frac{1}{m} - \frac{1}{n}\right)$$

(source: Wahyudi, 1999)

in which$\emptyset$ is the diameter of pile, *S*is the distance of axis to axis between piles in a group, *m* is the number of row of piles in a group, *n* is the number of pillar of piles in a group

c. loads repartition on piles in a group

- if on the piles foundation in group which are connected by *poer* work the vertical loads (*V*), horizontal loads (*H*), and moment (*M*), then the value of equivalent vertical load (P_V) which works in a pile is

  $P_V = \frac{V}{n} \pm \frac{M \bullet x}{\sum x^2}$(source: Wahyudi, 1999)

in which P_V is the value of equivalent vertical load which works in a pile, *n* is the sum of piles in a group, *e* is the distance between resultant intersection on *pilecap* base and neutral axis from piles group, *x* is the distance between a pile and the neutral axis of piles group, the value of *x* is positive if its direction same as *e* direction and the value of *x* is negative if its direction opposite to *e*, and *M* is the sum of moment (Figure 3.24).

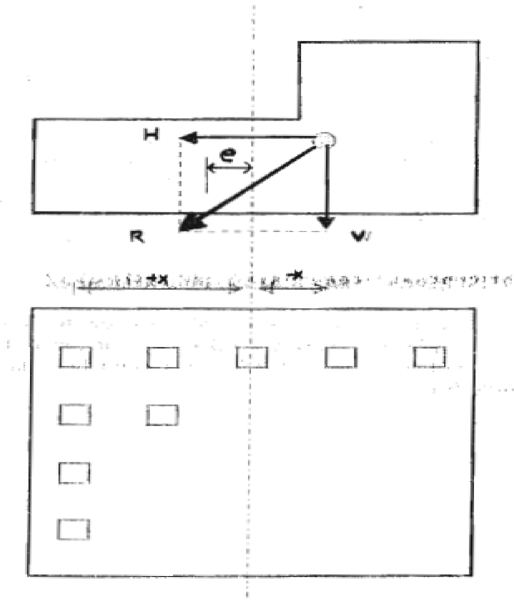

Figure 3.24. Repartition of loads on top of pile foundation group (source: Wahyudi, 1999)

Beside the calculations above, the calculation of shear in *pilecap* (shear *ponds*), the calculation of *poer*'s bar, and the calculation of sloof beam all of them shall be calculated based on SNI 2847:2013.

3.11. Technical Drawing

For technical drawing, this final project will use AutoCAD 2014® programme.

CHAPTER IV
CONTENTS

4.1. Preliminary Design
4.1.1. General

Preliminary design of the structure elements in this chapter consists of preliminary design of primary beam, secondary beam, slab, pillar, and shear wall. Note that the structure elements which are precast concrete elements consists of primary beam, secondary beam, and slab. While the structure elements which are cast in place concrete elements consists of pillar, basement structure, and shear wall.

4.1.2. General Data

Before designing the *preliminary design* of structure elements, it should be considered about general data of the building and the list of the load which shall be retained by the building's structure. The general data consists of the modified data of the building which is shown below

- project's name : Hotel Novotel THE SAMATOR Surabaya
- building's function : Hotel
- project's location : Kedungbaruk Road no. 28 Surabaya
- number of storey : 10 storeis and 1 storey of basement
- storey's height : 4 meters (for all storey including basement)
- building's height : 40 meters (from ground surface)
- building's area : 50 x 25 m^2 (1250 m^2)
- building's location : approx. < 5 km (from coast)

- f'_c (28 days) : 30 MPa
- f_y : 390 MPa

4.1.3. List of Loads

The list of loads below arebased on PPIUG (1983) and SNI 1727:2013. Exceptionally for earthquake load will follow the conditions which stated in SNI 1726:2012. The eartquake load will be explained later on another chapter. For loads in this final project, the gravity acceleration is assumed as 10 m/s^2.

1. Static Load
   - Dead Load
     - reinforced concrete : 2400 kg/m^3
     - floor covering made from portland cement tile, teraso, and concrete, without mortar, per cm thickness (tegel) : 24 kg/m^2
     - brick wall (1/2 brick) : 250 kg/m^2
     - ceiling (plafond) : 11 kg/m^2
     - plumbing and ducting : 25 kg/m^2
     - mortar (mixture), per cm of thickness (from cement) : 21 kg/m^2
   - Live Load
     - house for living (one family and two families) for all room except stair and balcony : 1,92 kN/m^2
     - Flat roof, and/or arch roof : 0,96 kN/m^2
     - Stair and exit way : 4,79 kN/m^2

2. Wind Load
   - wind load (approx. < 5 km away from coastal line) : 40 kg/m^2

4.1.4. Preliminary Design of Precast Element

4.1.4.1. Preliminary Design of Precast Primary Beam

In this final project, the rectangular shape will be used for precast primary beam's shape and precast secondary beam's shape. For preliminary design of precast primary beam sould consider the conditions which are stated on SNI 2847:2013.

$h_{min} = 1/16 \times L_b$ (source : SNI, 2013)

in which L_b is the gross span of primary beam (axis to axis)

If $f_y$value is not same as 420 Mpa, the h_{min} value must be multiplied by $(0,4 + f_y/700)$ (SNI, 2013). For primary beam width, b, it should be taken as 2/3 of h_{min} value.

$b_{min} = 2/3 \times h_{min}$

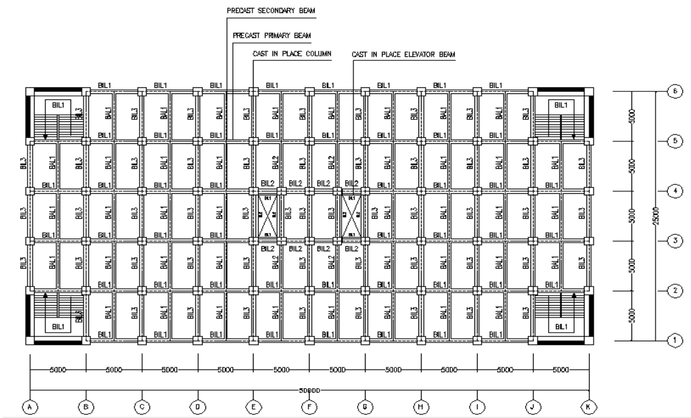

Figure 4.1. The blueprint of beam's design
(source: Private Documentation)

From figure 4.1, it can be determined about the value of L_b which is 5 meter.

$h_{min} = 1/16 \times 5 \times (0,4+400/700)$ m = 0,303 m ≈ 65 cm

$b_{min} = 2/3 \times h_{min} = 2/3 \times 0,303$ m = 0,202 m ≈ 40 cm

The preliminary design of precast primary beam is taken as h = 65 cm and b = 40 cm. So, the preliminary design of primary beam has met the condition in SNI 2847:2013 (note: the length of transverse primary beam is same as the longitudinal primary beam and the calculation of the dimension does not include *overtopping*).

4.1.4.2. Preliminary Design of Precast Secondary Beam

In this final project, the rectangular shape will be used for precast secondary beam's shape. For *preliminary design* of secondary beam should consider the conditions which are stated in SNI 2847:2013.

h_{min} = 1/21 x L_b (source : SNI, 2013)

in which L_b is the gross span of secondary beam (axis to axis)

If f_y value is as not same as 420 Mpa, the h_{min} value must be multiplied by (0,4 + f_y/700) (SNI, 2013). For secondary beam width, b, it should be taken as 2/3 of h_{min} value.

b_{min} = 2/3 x h_{min}

From figure 4.1, it can be determined about the value of L_b which is 5 meter.

h_{min} = 1/21 x 5 x (0,4+400/700) m = 0,231 m ≈ 45 cm

b_{min} = 2/3 x h_{min} = 2/3 x 0,231 m = 0,154 m ≈ 30 cm

The *preliminary design* of precast secondary beam is taken as h = 45 cm and b = 30 cm. So, the preliminary design of secondary beam has fulfill the condition in SNI 2847:2013 (note: there is only transverse secondary beam and the calculation of the dimension does not include *overtopping* concrete).

For the recapitulation of precast primary beam and precast secondary beam dimension can be seen on Table 4.1.

Table 4.1. Recapitulation of Precast Primary Beam and Precast Secondary Beam Dimension (Before Monolith Condition)

Code	L_b (m)	h_{min} (cm)	b_{min} (cm)	b_{use} (cm)	h_{use} (cm)	d (mm)	Dimension
Primary Beam (for G floor – 9th floor)							
BIL1	5	30,3	20,2	40	65	50	40/65
BIL2	2.5	15,2	10,2	40	65	50	40/65
BIL3	5	30,3	20,2	40	65	50	40/65
Secondary Beam (for G floor – 9th floor)							
BAL1	5	23,1	15,5	30	45	40	30/45
BAL2	5	23,1	15,5	30	45	40	30/45
Primary Beam (for roof)							
BIA1	5	30,3	20,2	40	65	50	40/65
BIA2	2.5	15,2	10,2	40	65	50	40/65
BIA3	5	30,3	20,2	40	65	50	40/65
Secondary Beam (for roof)							
BAA1	5	23,1	15,5	30	45	40	30/45
BAA2	5	23,1	15,5	30	45	40	30/45

4.1.4.3. Preliminary Design of Precast Slab

Preliminary Design of Precast Slab

For *preliminary design* of precast slab will follow the conditions in SNI 2847:2013. The steps of *preliminary design* of precast slab generally will be explained below

1. generating the general data
2. checking β value of slab
3. calculating effective width, b_e
4. calculating α_f and α_{fm}value
5. calculating h_{min}

Generating The General Data

The general data for *preliminary design* of precast slab will be shown below

- precast slab type : LA1 (500 x 250) (Figure 4.2)

- f'_c : 30 MPa
- f_y : 390 MPa
- slab thickness : estimated 15 cm (with *overtopping* concrete)

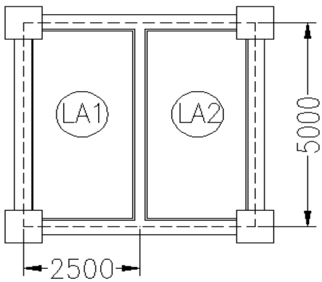

Figure 4.2. The precast slab type LA1 & LA2
(source: Private Documentation)

- $L_n = 500 - (40/2 + 40/2) = 460$ cm
- $S_n = 250 - (40/2 + 30/2) = 215$ cm

Checking β Value of Precast Slab

The βvalue of precast slab is obtained by dividing the value of L_n by S_n. If β value is greater than 2, then it is one way slab. But if If β value is less than 2, then it is two ways slab.

$\beta = L_n/S_n$ = 460 cm/215 cm = 2,139 > 2 (one way slab)

Calculating Effective Flens Width, b_e

The effective flens width, b_e, shall follow the conditions stated in SNI 2847:2013 which will be explained below (note, use h_f = 15 cm).

- for precast interior primary beam/secondary beam (with precast slab)

The shape is T beam (Figure 4.3) which will follow the conditions in SNI 2847:2013.

$b_e \leq b_w + 2(8h_f)$ choose whichever the least value of b_e
$b_e \leq b_w + 2(L_n/2)$
(source: SNI, 2013 paragraph 8.12.2)

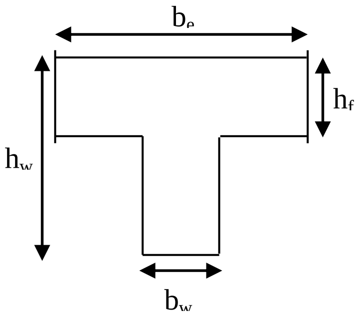

Figure 4.3. The example of precast T beam
(source: Private Documentation)

- for precast exterior primary beam (with precast slab)

The shape is inverted L beam(Figure 4.4) which will follow the conditions in SNI 2847:2013.

$b_e \leq b_w + (L/12)$ choose whichever the least value of b_e
$b_e \leq b_w + (6h_f)$
$b_e \leq b_w + (L_n/2)$
(source: SNI, 2013 paragraph 8.12.3)

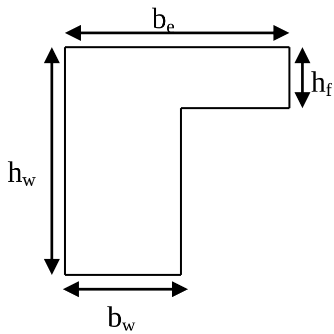

Figure 4.4. The example of precast inverted L beam
(source: Private Documentation)

1. for precast primary beam (assumed in monolith condition)

1.1. precast interior primary beam 40/65, L (axis to axis) = 500 cm (Figure 4.5)

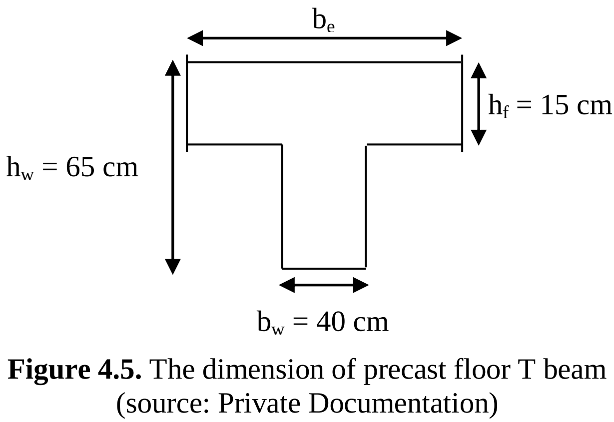

Figure 4.5. The dimension of precast floor T beam
(source: Private Documentation)

$b_e \leq 40 + 2(8x15)$ $= 280$ cm
$b_e \leq 40 + 2(460/2)$ $= 500$ cm
use b_e value 280 cm

1.2. precast exterior primary beam 40/65, L (axis to axis) = 500 cm (Figure 4.6)

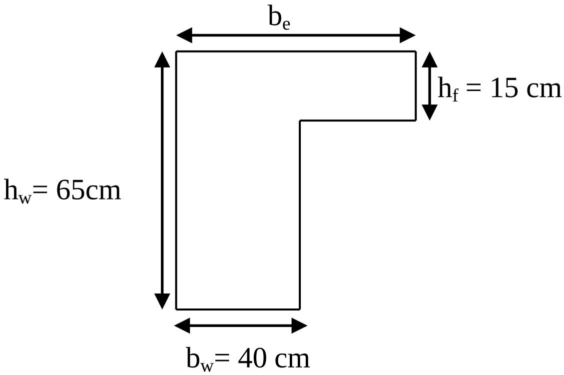

Figure 4.6. The dimension of precast inverted floor L beam (source: Private Documentation)

$b_e \leq 40 + (500/12)$ = 81,667 cm
$b_e \leq 40 + (6x15)$ = 130 cm
$b_e \leq 40 + (460/2)$ = 270 cm
use b_e value 81.667 cm

2. for precast secondary beam (assumed in monolith condition)

2.1. precast interior secondary beam 30/45, L (axis to axis) = 500 cm (Figure 4.7)

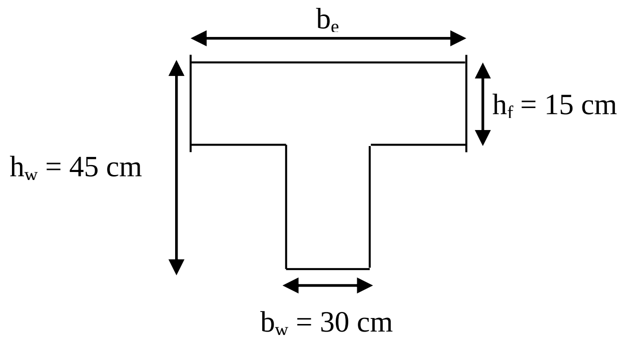

Figure 4.7. The dimension of precast floor T secondary beam (source: Private Documentation)

$b_e \leq 30 + 2(8x15)$ = 270 cm
$b_e \leq 30 + 2(460/2)$ = 490 cm
use b_e value 270 cm

Calculating α_f and α_{fm} Value

The α_f value is obtained by using equation below

$$\alpha_f = \frac{E_{beam}}{E_{slab}} \; x \; \frac{I_{beam}}{I_{slab}}$$

in which E_{beam} and E_{slab} is the value of elasticity modulus which taken as $4700\sqrt{f_c'}$. While the value of α_{fm} is obtained from mean value of α_f.

The value of I_{beam} and I_{slab} is obtained by using equations below

$$I_{beam} = 1/12 \text{ x } b_w \text{ x } h_w^3 \text{ x } k$$
$$I_{slab} = 1/12 \text{ x } L \text{ x } h_f^3$$

in which the shape of beam is T beam so it needs to be multiplied by k. L is the value of span of beam.

$$k = \frac{1+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)\left[4-6\left(\frac{hf}{hw}\right)+4\left(\frac{hf}{hw}\right)^2+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)^3\right]}{1+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)}$$

1. $\alpha_f$value for precast primary beam (assumed in monolith condition)

1.1. precast interior primary beam 40/65, L (axis to axis) = 500 cm

$$k = \frac{1+\left(\frac{280}{40}-1\right)\left(\frac{15}{65}\right)\left[4-6\left(\frac{15}{65}\right)+4\left(\frac{15}{65}\right)^2+\left(\frac{280}{40}-1\right)\left(\frac{15}{65}\right)^3\right]}{1+\left(\frac{280}{40}-1\right)\left(\frac{15}{65}\right)}$$

k = 2,902

I_{beam}= 1/12 x 40x 65^3 x 2,902 cm^4 = 2656666,67 cm^4

I_{slab} = 1/12 x 500 x 15^3 cm^4 = 140625 cm^4

E_{beam}= $4700\sqrt{30}$ MPa = 25742,96 MPa

E_{slab}= $4700\sqrt{30}$ MPa = 25742,96 MPa

α_f= $\frac{25742{,}96\ \text{MPa}}{25742{,}96\ \text{MPa}}\ x\ \frac{2656666{,}67\ \text{cm}^4}{140625\ \text{cm}^4}$ = 18,89

1.2. precast eksterior primary beam 40/65, L (axis to axis) = 500 cm

$$k = \frac{1+\left(\frac{81.667}{40}-1\right)\left(\frac{15}{65}\right)\left[4-6\left(\frac{15}{65}\right)+4\left(\frac{15}{65}\right)^2+\left(\frac{81.667}{40}-1\right)\left(\frac{15}{65}\right)^3\right]}{1+\left(\frac{81.667}{40}-1\right)\left(\frac{15}{65}\right)}$$

k = 2,84

I_{beam}= 1/12 x 40x 65^3 x 2,84 cm^4 = 2600885,417 cm^4

I_{slab} = 1/12 x 500 x 15^3 cm^4 = 140625 cm^4

E_{beam}= $4700\sqrt{30}$ MPa = 25742,96 MPa
E_{slab}= $4700\sqrt{30}$ MPa = 25742,96 MPa
$\alpha_f = \frac{25742{,}96\ \mathrm{MPa}}{25742{,}96\ \mathrm{MPa}}\ x\ \frac{2600885{,}417\ \ \mathrm{cm}^4}{140625\ \mathrm{cm}^4} = 18{,}49$

2. $\alpha_f$value for precast secondary beam (assumed in monolith condition)

2.1. precast interior secondary beam 30/45, L (axis to axis) = 500 cm

$$k = \frac{1+\left(\frac{270}{30}-1\right)\left(\frac{15}{45}\right)\left[4-6\left(\frac{15}{45}\right)+4\left(\frac{15}{45}\right)^2+\left(\frac{270}{30}-1\right)\left(\frac{15}{45}\right)^3\right]}{1+\left(\frac{270}{30}-1\right)\left(\frac{15}{45}\right)}$$

k = 2,741
I_{beam}= 1/12 x 30x 45^3 x 2,741 cm^4 = 624375 cm^4
I_{slab} = 1/12 x 500 x 15^3 cm^4 = 140625 cm^4
E_{beam}= $4700\sqrt{30}$ MPa = 25742,96 MPa
E_{slab}= $4700\sqrt{30}$ MPa = 25742,96 MPa
$\alpha_f = \frac{25742{,}96\ \mathrm{MPa}}{25742{,}96\ \mathrm{MPa}}\ x\ \frac{624375\ \mathrm{cm}^4}{140625\mathrm{cm}^4} = 4{,}44$

$\alpha_{fm} = \frac{18{,}89+18{,}49+4{,}44}{3} = 13{,}942 > 2$

Calculating h_{min}

For α_{fm} which has greater value than 2,0, the minimum thickness of slab must not less than

$$h = \frac{\ell_n\ (0{,}8 + \frac{f_y}{1400})}{36 + 9\beta}$$

and must not less than 90 mm

$$h = \frac{460\ (0{,}8+\frac{400}{1400})}{36+9x2{,}139}\ x\ \left(0{,}4 + \frac{400}{700}\right) = 8{,}780\ \mathrm{cm} \approx 9\ \mathrm{cm}$$

The previous h value of precast slab is 15 cm, which fulfill the condition in SNI 2847:2013 about minimum thickness of slab. So, the h value of slab which will be used is 15 cm for G floor – 9th floor, it consists of 10 cm of slab precast concrete and 5 cm of *overtopping* concrete.

Preliminary Design of Precast Slab (for roof)

For *preliminary design* of precast slab will follow the conditions in SNI 2847:2013. The steps of *preliminary design* of precast slab generally will be explained below

1. generating the general data
2. checking β value of slab
3. calculating effective width, b_e
4. calculating α_f and α_{fm}value
5. calculating h_{min}

Generating The General Data

The general data for *preliminary design* of precast slab will be shown below

- precast slab type : AA1 (500 x 250) (Figure 4.8)
- f'_c : 30 Mpa
- f_y : 390 Mpa
- slab thickness : estimated 14 cm (with *overtopping* concrete)

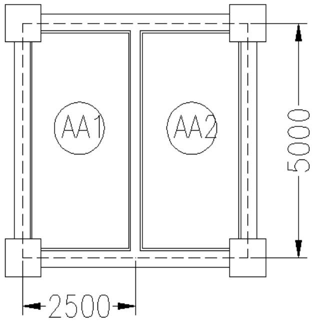

Figure 4.8. The precast slab type AA1 & AA2
(source: Private Documentation)

- L_n = 500 - (40/2 + 40/2) = 460 cm
- S_n = 250 - (40/2 + 30/2) = 215 cm

<u>Checking β Value of Precast Slab</u>

The βvalue of precast slab is obtained by dividing the value of L_n by S_n. If β value is greater than 2, then it is one way slab. But if If β value is less than 2, then it is two ways slab.

$\beta = L_n/S_n$ = 460 cm/215 cm = 2,139 > 2 (one way slab)

<u>Calculating Effective Flens Width, b_e</u>

1. for precast primary beam (assumed in monolith condition)

1.1. precast interior primary beam 40/65, L (axis to axis) = 500 cm (Figure 4.9)

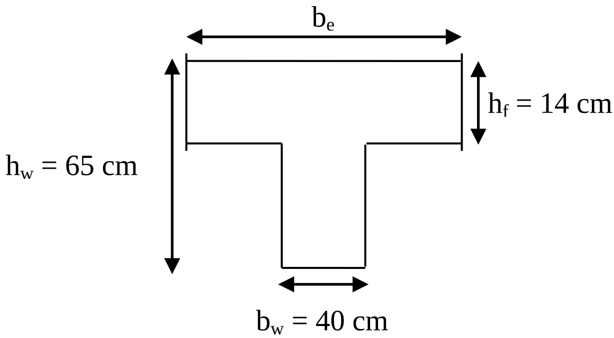

Figure 4.9. The dimension of precast roof T beam
(source: Private Documentation)

$b_e \leq 40 + 2(8x14)$ = 264 cm
$b_e \leq 40 + 2(460/2)$ = 500 cm
use b_e value 248 cm

1.2. precast eksterior primary beam 40/65, L (axis to axis) = 500 cm (Figure 4.10)

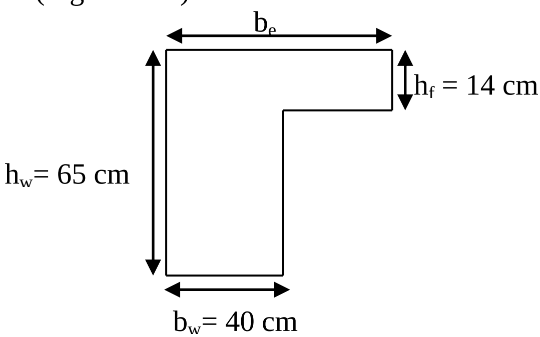

Figure 4.10. The dimension of precast inverted roof L beam
(source: Private Documentation)

$b_e \leq 40 + (500/12)$ = 81,667 cm
$b_e \leq 40 + (6x14)$ = 124 cm

$b_e \leq 40 + (460/2)$ = 270 cm
use b_e value 81.667 cm

2. for precast secondary beam (assumed in monolith condition)

2.1. precast interior secondary beam 30/45, L (axis to axis) = 500 cm (Figure 4.11)

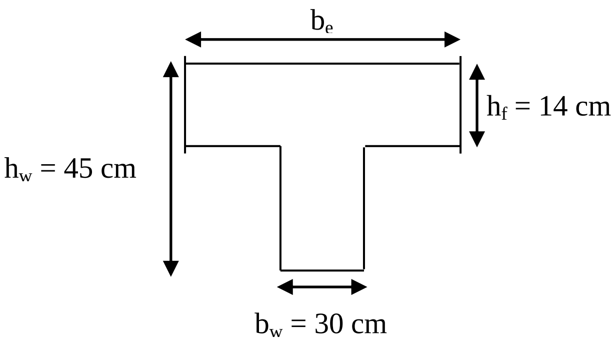

Figure 4.11. The dimension of precast roof T secondary beam (source: Private Documentation)

$b_e \leq 30 + 2(8x14)$ = 254 cm
$b_e \leq 30 + 2(460/2)$ = 490 cm
use b_e value 238 cm

Calculating α_f and α_{fm} Value

The α_f value is obtained by using equation below

$$\alpha_f = \frac{E_{beam}}{E_{slab}} \; x \; \frac{I_{beam}}{I_{slab}}$$

in which E_{beam} and E_{slab} is the value of elasticity modulus which taken as $4700\sqrt{f_c'}$. While the value of α_{fm} is obtained from mean value of α_f.

The value of I_{beam} and I_{slab} is obtained by using equations below

I_{beam}= 1/12 x b_w x h_w^3 x k

I_{slab}= 1/12 x L x h_f^3

in which the shape of beam is T beam so it needs to be multiplied by k. L is the value of span of beam.

$$k = \frac{1+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)\left[4-6\left(\frac{hf}{hw}\right)+4\left(\frac{hf}{hw}\right)^2+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)^3\right]}{1+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)}$$

1. $\alpha_f$value for precast primary beam (assumed in monolith condition)

1.1. precast interior primary beam 40/65, L (axis to axis) = 500 cm

$$k = \frac{1+\left(\frac{248}{40}-1\right)\left(\frac{14}{65}\right)\left[4-6\left(\frac{14}{65}\right)+4\left(\frac{14}{65}\right)^2+\left(\frac{248}{40}-1\right)\left(\frac{14}{65}\right)^3\right]}{1+\left(\frac{248}{40}-1\right)\left(\frac{14}{65}\right)}$$

k = 2,949

I_{beam}= 1/12 x 40x 65^3 x 2,949 cm^4 = 2699754,667 cm^4

I_{slab} = 1/12 x 500 x 14^3 cm^4 = 114333,333 cm^4

E_{beam}= $4700\sqrt{30}$ MPa = 25742,96 MPa

E_{slab}= $4700\sqrt{30}$ MPa = 25742,96 MPa

α_f= $\frac{25742{,}96\ \mathrm{MPa}}{25742{,}96\ \mathrm{MPa}}\ x\ \frac{2699754{,}667\ \mathrm{cm}^4}{114333{,}333\ \mathrm{cm}^4}$ = 23,613

1.2. precast eksterior primary beam 40/65, L (axis to axis) = 500 cm

$$k=\frac{1+\left(\frac{81.667}{40}-1\right)\left(\frac{14}{65}\right)\left[4-6\left(\frac{14}{65}\right)+4\left(\frac{14}{65}\right)^2+\left(\frac{81.667}{40}-1\right)\left(\frac{14}{65}\right)^3\right]}{1+\left(\frac{81.667}{40}-1\right)\left(\frac{14}{65}\right)}$$

k = 2,903

I_{beam}= 1/12 x 40x 65^3 x 2,903 cm^4 = 2658061,111 cm^4

I_{slab} = 1/12 x 500 x 14^3 cm^4 = 114333,333 cm^4

E_{beam}= $4700\sqrt{30}$ MPa = 25742,96 MPa

E_{slab}= $4700\sqrt{30}$ MPa = 25742,96 MPa

$\alpha_f = \frac{25742{,}96\ \text{MPa}}{25742{,}96\ \text{MPa}}\ x\ \frac{2658061{,}111\ \text{cm}^4}{114333{,}333\ \text{cm}^4} = 23{,}24$

2. $\alpha_f$value for precast secondary beam (assumed in monolith condition)

2.1. precast interior secondary beam 30/45, L (axis to axis) = 500 cm

$$k=\frac{1+\left(\frac{238}{30}-1\right)\left(\frac{14}{45}\right)\left[4-6\left(\frac{14}{45}\right)+4\left(\frac{14}{45}\right)^2+\left(\frac{238}{30}-1\right)\left(\frac{14}{45}\right)^3\right]}{1+\left(\frac{238}{30}-1\right)\left(\frac{14}{45}\right)}$$

k = 2,745

I_{beam}= 1/12 x 30x 45^3 x 2,745 cm^4 = 625421,33 cm^4

I_{slab} = 1/12 x 500 x 14^3 cm^4 = 114333,333 cm^4

E_{beam}= $4700\sqrt{30}$ MPa = 25742,96 MPa

E_{slab}= $4700\sqrt{30}$ MPa = 25742,96 MPa

$\alpha_f = \frac{25742{,}96\ \text{MPa}}{25742{,}96\ \text{MPa}}\ x\ \frac{625421{,}33\text{cm}^4}{114333{,}333\ \text{cm}^4} = 5{,}47$

$\alpha_{fm} = \frac{23{,}613+23{,}24+5{,}47}{3} = 17{,}44 > 2$

<u>Calculating h_{min}</u>

For α_{fm} which has greater value than 2.0, the minimum thickness of slab must not less than

$$h = \frac{\ell_n \left(0{,}8 + \frac{f_y}{1400}\right)}{36 + 9\beta}$$

and must not less than 90 mm

$$h = \frac{460\ (0{,}8+\frac{400}{1400})}{36+9x2{,}139}\ x\ \left(0{,}4 + \frac{400}{700}\right) = 8{,}7802 \text{ cm} \approx 9 \text{ cm}$$

The previous h value of precast slab is 14 cm, which fulfill the condition in SNI 2847:2013 about minimum thickness of slab. So, the h value of slab which will be used is 13 cm for roof, it consists of 9 cm of slab precast concrete and 5 cm of *overtopping* concrete.For the recapitulation of precast slab dimension of precast element can be seen on Table 4.2.

Table 4.2. Recapitulation of Precast Slab Dimension (monolit)

Code/Type	P	L	Ln	Sn	β	h_{min}	h_{use}
	(cm)	(cm)	(cm)	(cm)		(cm)	(cm)
Slab (for G floor – 9th floor)							
LA1/one way slab	500	250	460	215	2,139	9	15
LA2/one way slab	500	250	460	215	2,139	9	15
LB1/one way slab	500	250	460	215	2,139	9	15
LB2/one way slab	500	250	460	215	2,139	9	15
LB3/one way slab	500	250	460	215	2,139	9	15
LB4/one way slab	500	250	460	215	2,139	9	15
LC1/one way slab	500	250	460	215	2,139	9	15
Slab (for roof)							
AA1/one way slab	500	250	460	215	2,139	9	14
AA2/one way slab	500	250	460	215	2,139	9	14
AB1/one way slab	500	250	460	215	2,139	9	14
AB2/one way slab	500	250	460	215	2,139	9	14
AB3/one way slab	500	250	460	215	2,139	9	14
AB4/one way slab	500	250	460	215	2,139	9	14
AC1/one way slab	500	250	460	215	2,139	9	14

4.1.5. Preliminary Design of Basement's Structure

4.1.5.1. Preliminary Design of Basement Primary Beam

In this final project, the rectangular shape will be used for basement primary beam's shape and basement secondary beam's shape. For preliminary design of basement primary beam sould consider the conditions which are stated in SNI 2847:2013.

h_{min} = 1/16 x L_b (source : SNI, 2013)

in which L_b is the gross span of beam

If f_y value is not same as 420 Mpa, the h_{min} value must be multiplied by (0,4 + f_y/700) (SNI, 2013). For primary beam width, b, it should be taken as 2/3 of h_{min} value.

h_{min} = 1/16 x 5 x (0,4+400/700) m = 0,303 m ≈ 80 cm

b_{min} = 2/3 x h_{min} = 2/3 x 0,303 m = 0,202 m ≈ 65 cm

The preliminary design of basement primary beam is taken as h = 80 cm and b = 65 cm. So, the preliminary design of basement primary beam has met the condition in SNI 2847:2013 (note: the length of basement transverse primary beam is same as the basement longitudinal primary beam).

4.1.5.2. Preliminary Design of Basement Secondary Beam

In this final project, the rectangular shape will be used for basement secondary beam's shape. For preliminary design of basement secondary beam should consider the conditions which are stated in SNI 2847:2013.

h_{min} = 1/21 x L_b (source : SNI, 2013)

in which L_b is the gross span of secondary beam

If f_y value is as not same as 420 Mpa, the h_{min} value must be multiplied by (0,4 + f_y/700) (SNI, 2013). For secondary beam width, b, it should be taken as 2/3 of h_{min} value.

b_{min} = 2/3 x h_{min}

h_{min} = 1/21 x 5 x (0.4+400/700) m = 0,231 m ≈ 70 cm

b_{min} = 2/3 x h_{min} = 2/3 x 0,231 m = 0,154 m ≈ 50 cm

The preliminary design of basement secondary beam is taken as h=70 cm and b=50 cm. So, the preliminary design of basementsecondary beam has fulfill the condition in SNI 2847:2013 (note: there is only basementtransverse secondary beam). For the recapitulation of basementprimary beam and basementsecondary beam dimension can be seen on Table 4.3.

Table 4.3. Recapitulation of Basement Primary Beam and Basement Secondary Beam Dimension

Code	L_b (m)	h_{min} (cm)	b_{min} (cm)	b_{use} (cm)	h_{use} (cm)	d (mm)	Dimension
Primary Beam (for LG/Basement floor)							
BIB1	5	30,3	20,2	65	80	50	65/80
BIB2	2,5	15,2	10,2	65	80	50	65/80
BIB3	5	30,3	20,2	65	80	50	65/80
BIB4	2,5	15,2	10,2	40	80	50	40/80
BIB5	5	30,3	20,2	40	80	50	40/80
Secondary Beam (for LG/Basement floor)							
BAB1	5	23,1	15,5	50	70	50	50/70
BAB2	5	23,1	15,5	50	70	50	50/70

4.1.5.3. Preliminary Design of Basement Slab

For preliminary design of basement slab will follow the conditions in SNI 2847:2013. The steps of preliminary design of basement slab generally will be explained below

1. generating the general data
2. checking β value of slab
3. calculating effective width, b_e
4. calculating α_f and α_{fm} value
5. calculating h_{min}

<u>Generating The General Data</u>

The general data for preliminary design of basementslab will be shown below

- basement slab type : A (500 x 250)

- f'_c : 30 MPa
- f_y : 390 MPa
- slab thickness : estimated 20 cm

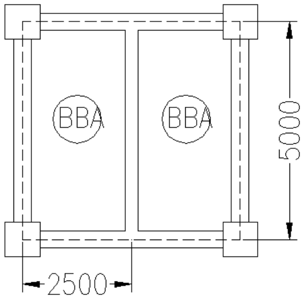

Figure 4.12. The precast slab type BBA
(source: Private Documentation)

- L_n = 500 - (65/2 + 65/2) = 435 cm
- S_n = 250 - (50/2 + 65/2) = 192,5 cm

Checking β Value of Precast Slab

The βvalue of basement slab is obtained by dividing the value of L_n by S_n. If β value is greater than 2, then it is one way slab. But if If β value is less than 2, then it is two ways slab.

$\beta = L_n/S_n$ = 435 cm/192,5 cm = 2,25> 2 (one way slab)

Calculating Effective Flens Width, b_e

1. for basement primary beam

1.1. basement interior primary beam 65/80, L (axis toaxis) = 500 cm (Figure 4.13)

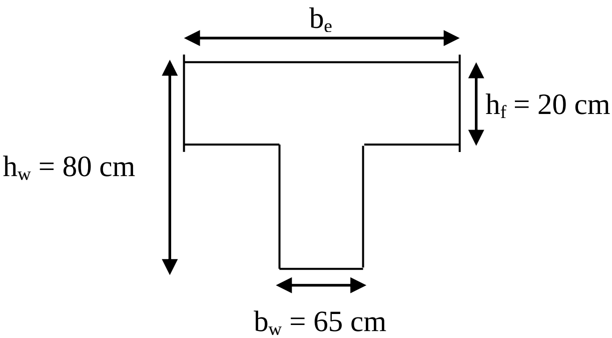

Figure 4.13. The dimension of basement T beam
(source: Private Documentation)

$b_e \leq 65 + 2(8x20)$ = 385 cm
$b_e \leq 65 + 2(435/2)$ = 500 cm
use b_e value 385 cm

1.2. basement eksterior primary beam 65/80, L (axis to axis) = 500 cm (Figure 4.14)

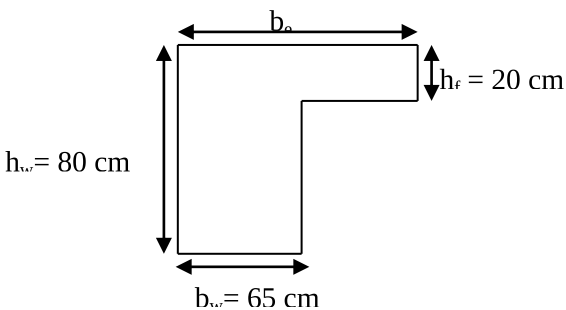

Figure 4.14. The dimension of basement inverted L beam
(source: Private Documentation)

$b_e \leq 65 + (500/12)$ = 106,667 cm
$b_e \leq 65 + (6x20)$ = 185 cm

$b_e \leq 65 + (435/2)$ $= 282,5$ cm
use b_e value 106,667 cm

2. for basement secondary beam
    2.1. basement interior secondary beam 50/70, L (axis to axis) = 500 cm (Figure 4.15)

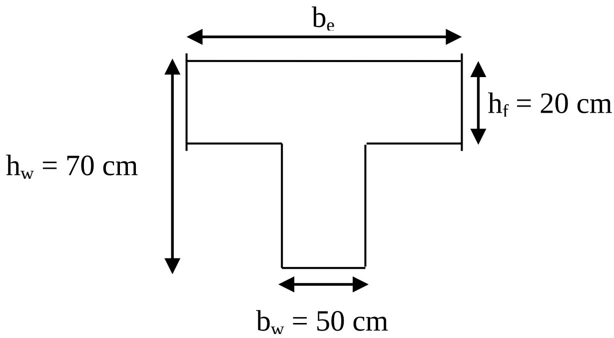

Figure 4.15. The dimension of basement T secondary beam (source: Private Documentation)

$b_e \leq 50 + 2(8x20)$ $= 370$ cm
$b_e \leq 50 + 2(435/2)$ $= 485$ cm
use b_e value 370 cm

Calculating α_f and α_{fm} Value

The α_f value is obtained by using equation below

$$\alpha_f = \frac{E_{beam}}{E_{slab}} \, x \, \frac{I_{beam}}{I_{slab}}$$

in which E_{beam} and E_{slab} is the value of elasticity modulus which taken as $4700\sqrt{f_c'}$. While the value of α_{fm} is obtained from mean value of α_f.

The value of I_{beam} and I_{slab} is obtained by using equations below

$$I_{beam} = 1/12 \text{ x } b_w \text{ x } h_w^3 \text{ x } k$$
$$I_{slab} = 1/12 \text{ x } L \text{ x } h_f^3$$

in which the shape of beam is T beam so it needs to be multiplied by k. L is the value of span of beam.

$$k=\frac{1+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)\left[4-6\left(\frac{hf}{hw}\right)+4\left(\frac{hf}{hw}\right)^2+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)^3\right]}{1+\left(\frac{be}{bw}-1\right)\left(\frac{hf}{hw}\right)}$$

1. $\alpha_f$value for basement primary beam

1.1. basement interior primary beam 65/80, L (axis to axis) = 500 cm

$$k=\frac{1+\left(\frac{385}{65}-1\right)\left(\frac{20}{80}\right)\left[4-6\left(\frac{20}{80}\right)+4\left(\frac{20}{80}\right)^2+\left(\frac{385}{65}-1\right)\left(\frac{20}{80}\right)^3\right]}{1+\left(\frac{385}{65}-1\right)\left(\frac{20}{80}\right)}$$

k = 2,827

I_{beam} = 1/12 x 65x 80^3 x 2,827 cm^4 = 7840000 cm^4

I_{slab} = 1/12 x 500 x 20^3 cm^4 = 333333,33 cm^4

E_{beam} = $4700\sqrt{30}$ MPa = 25742,96 MPa

E_{slab} = $4700\sqrt{30}$ MPa = 25742,96 MPa

$\alpha_f = \frac{25742{,}96\ \mathrm{MPa}}{25742{,}96\ \mathrm{MPa}}\ \mathrm{x}\ \frac{7840000\ \mathrm{cm}^4}{333333{,}33\mathrm{cm}^4}$ = 23,52

1.2. basement exterior primary beam 65/80, L (axis to axis) = 500 cm

$$k=\frac{1+\left(\frac{106{,}667}{65}-1\right)\left(\frac{20}{80}\right)\left[4-6\left(\frac{20}{80}\right)+4\left(\frac{20}{80}\right)^2+\left(\frac{106{,}667}{65}-1\right)\left(\frac{20}{80}\right)^3\right]}{1+\left(\frac{106{,}667}{65}-1\right)\left(\frac{20}{80}\right)}$$

k = 2,760

I_{beam} = 1/12 x 65x 80^3 x 2,760 cm^4 =7654444,444 cm^4

I_{slab} = 1/12 x 500 x 20^3 cm^4 = 333333,33 cm^4

E_{beam} = $4700\sqrt{30}$ MPa = 25742,96 MPa
E_{slab} = $4700\sqrt{30}$ MPa = 25742,96 MPa
$\alpha_f = \frac{25742{,}96\ \text{MPa}}{25742{,}96\ \text{MPa}}\ x\ \frac{7654444{,}444\ \text{cm}^4}{333333{,}33\text{cm}^4}$ = 22,963

2. $\alpha_f$value for basement secondary beam

2.1. basement interior secondary beam 50/70, L (axis to axis) = 500 cm

$$k = \frac{1+\left(\frac{370}{50}-1\right)\left(\frac{20}{70}\right)\left[4-6\left(\frac{20}{70}\right)+4\left(\frac{20}{70}\right)^2+\left(\frac{370}{50}-1\right)\left(\frac{20}{70}\right)^3\right]}{1+\left(\frac{370}{50}-1\right)\left(\frac{20}{70}\right)}$$

k = 2,762
I_{beam} = 1/12 x 50x 70^3 x 2,762 cm^4 = 3946666,7 cm^4
I_{slab} = 1/12 x 500 x 20^3 cm^4 = 333333,33 cm^4
E_{beam} = $4700\sqrt{30}$ MPa = 25742,96 MPa
E_{slab} = $4700\sqrt{30}$ MPa = 25742,96 MPa
$\alpha_f = \frac{25742{,}96\ \text{MPa}}{25742{,}96\ \text{MPa}}$ x $\frac{3946666{,}7\ \text{cm}^4}{333333{,}33\text{cm}^4}$ = 11,84

$\alpha_{fm} = \frac{23{,}52+22{,}963+11{,}84}{3}$ = 19.4411 > 2

<u>Calculating h_{min}</u>

For α_{fm} which has greater value than 2.0, the minimum thickness of slab must not less than

$$h = \frac{\ell_n\ (0{,}8+\ \frac{f_y}{1400})}{36+9\beta}$$

and must not less than 90 mm

$$h = \frac{460\ (0{,}8+\frac{400}{1400})}{36+9x2{,}25}\ x\ \left(0{,}4+\frac{400}{700}\right) = 8{,}14\text{ cm} \approx 9\text{ cm}$$

The previous h value of basement slab is 20 cm, which fulfill the condition in SNI 2847:2013 about minimum thickness of slab. So, the h value of slab which will be used is 20 cm for basement. For the recapitulation of basement slab dimension can be seen on Table 4.4.

Table 4.4. Recapitulation of Basement Slab Dimension

Code/Type	P	L	Ln	Sn	β	h_{min}	h_{use}
	(cm)	(cm)	(cm)	(cm)		(cm)	(cm)
Slab (for LG/Basement floor)							
BBA/one way slab	500	250	435	192,5	2,25	9	20
BBB/one way slab	500	250	445	192,5	2,31	9	20
BBC/one way slab	500	250	435	192,5	2,25	9	20
BBD/one way slab	500	250	435	200,2	2,22	9	20
BBE/one way slab	500	250	435	197,5	2,25	9	20

4.1.6. Preliminary Design of Pillar

Pillar has to bear the axial load and maximum moment from the axial load which is located on nearest span from the pillar. It is assumed that the area which its loads and moment retained by one pillar as 500 cm x 500 cm area (note b pillar’s early dimension is 900 mm x 900 mm). The pillar itself, which will be used for preliminary designcalculation, is pillar C3 (Figure 4.16) because it is assumed to bear the maximum loads and moment.

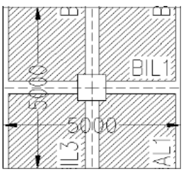

Figure 4.16. The area of loads of one pillar
(source: Private Documentation)

The list of axial loads which retained by pillar C3 (pillar C3 is located on LG floor/basement) are shown below

1. dead load (1st floor – 9th floor)

slab	: 5x5x0,15x2400x9 kg	= 81000 kg
primary beam		
transversal	: 0,4x0,8x5x2400x9 kg	=34560 kg
longitudinal	: 0,4x0,8x5x2400x9 kg	=34560 kg
secondary beam		
transversal	:2x0,3x0,6x5x2400x9 kg	= 38880 kg
pillar	: 0,9x0,9x4x2400x9 kg	=69984 kg
brick	: 5x5x250x9 kg	= 56250 kg
ceiling (plafond)	: 5x5x11x9 kg	= 2475 kg
hanger	: 5x5x7x9 kg	= 1575 kg
plumbing and ducting	: 5x5x25x9 kg	= 5625 kg
sanitary	: 5x5x20x9 kg	= 4500 kg
mortar mixture (spesi 2cm)	: 5x5x2x21x9 kg	= 9450 kg
tile (1 cm)	: 5x5x24x9 kg	= 5400 kg
DL (total)	:	= 344259 kg

2. live load (1st floor – 9th floor)

floor	: 5x5x1,92 kN/m^{2}x9 kg	= 48000 kg
LL (total)	:	= 48000 kg

3. dead load (G floor)

slab	: 5x5x0,15x2400x1 kg	= 9000 kg
primary beam		
transversal	: 0,4x0,8x5x2400x1 kg	= 3840 kg
longitudinal	: 0,6x0,8x5x2400x1 kg	= 3840 kg
secondary beam		
transversal	: 2x0,3x0,6x5x2400x1 kg	= 4320 kg
pillar	: 0,9x0,9x4x2400x1 kg	= 7776 kg
brick	: 5x5x250x1 kg	= 6250 kg

plumbing and ducting	: 5x5x25x1 kg	= 625 kg
sanitary	: 5x5x20x1 kg	= 500 kg
mortar mixture (spesi 2cm)	: 5x5x2x21x1 kg	= 1050 kg
tile (1 cm)	: 5x5x24x1 kg	= 600 kg
DL (total)	:	= 37201 kg

4. live load (G floor)

floor	: 5x5x1,92 kN/m^{2}x1 kg	= 4800 kg
LL (total)	:	= 4800 kg

5. dead load (roof)

slab	: 5x5x0,14x2400x1 kg	= 8400 kg
primary beam		
transversal	: 0,4x0,8x5x2400x1 kg	= 3840 kg
longitudinal	: 0,4x0,8x5x2400x1 kg	= 3840 kg
secondary beam		
transversal	: 2x0,3x0,6x5x2400x1 kg	= 4320 kg
ceiling (plafond)	: 5x5x11x1 kg	= 275 kg
hanger	: 5x5x7x1 kg	= 175 kg
plumbing and ducting	: 5x5x25x1 kg	= 625 kg
asphalt (1 cm)	: 5x5x14x1 kg	= 350 kg
mortar mixture (spesi 2cm)	: 5x5x2x21x1 kg	= 1050 kg
DL (total)	:	= 22875 kg

6. live load (roof)

roof	: 5x5x0,96 kN/m^{2}x1 kg	= 2400 kg
rain	: 5x5x20 kg/m^{2}x1 kg	= 500 kg
LL (total)	:	= 2900 kg

DL (total) = 404335 kg
LL (total) = 50900 kg

According to SNI (2013), the LL total does not need to be multiplied by reduction factor. The load combination is taken as 1,2DL + 1,6LL.

W = 1,2DL+1,6LL
= 1,2x404335 kg + 1,6x50900 kg
= 566642kg

$$A = \frac{W}{\phi\ x\ f'c}$$

W = 566642 kg
f'_c = 30 MPa = 300 kg/cm^2
ϕ = 0,65 (SNI, 2013 paragraph 9.3.2.2)

$A = \frac{566642}{0,65 \text{ x } 300}$ = 2905,856 cm^2, if A = b^2 then b = 53,9 cm
b ≈ 80 cm $\longrightarrow$ b = 80 cm for all pillar
note that the dimension of pillar is typical (for all pillar's dimension) but the reinforcement bar will be different.

4.1.7. Preliminary Design of Shear Wall

For designing of shear wall dimension shall meet the conditions shown in SNI (2013) paragraph 14.5.3.1 and paragraph 14.5.3.2 which state that the thickness of support wall (or *shear wall*) must not less than 1/25 of height or the length of supported span, whichever is shorter, or less than 100 mm.

t = thickness of shear wall		= 30 cm
span of shear wall		= 500 cm
height of shear wall		= 400 cm
t ≥ h/25	= 400/25	= 16 cm
t ≥ l/25	= 500/25	= 20 cm

So, the thickness of shear wall is taken as 40 cm.

4.1.8. Preliminary Design of Basement Wall

For designing of basement wall dimension shall meet the conditions shown in SNI (2013) paragraph 14.5.3.1 and paragraph 14.5.3.2 which state that the thickness of support wall (or basementwall) must not less than 1/25 of height or the length of supported span, whichever is shorter, or less than 100 mm.

t = thickness of basementwall		= 30 cm
span of basementwall		= 500 cm
height of basementwall		= 400 cm
$t \geq h/25$	= 400/25	= 16 cm
$t \geq l/25$	= 500/25	= 20 cm

So, the thickness of basementwallis taken as 30 cm.

4.1.9. Preliminary Design of Other Structures

For designing of other structure dimension shall meet the conditions shown in SNI 2847:2013. The other structure are elevator beam and machine room's beam and slab (see Table 4.5 and Table 4.6) and bottom structures.

Table 4.5. Recapitulation of Elevator Beam Dimension (Cast In Place Concrete)

Code	L_b (m)	b_{use} (cm)	h_{use} (cm)	d (mm)	Dimension
Beam (Elevator Beam)					
BL1	1.7	20	65	30	20/65
BL2	4.2	20	65	30	20/65
BL3	1.7	20	65	30	20/65
BPS	1.7	20	65	30	20/65

Table 4.6. Recapitulation of Machine Room's Slab Dimension
(Cast In Place Concrete)

Code/Type	P	L	Ln	Sn	β	h_{min}	h_{use}
	(cm)	(cm)	(cm)	(cm)		(cm)	(cm)
Slab (Machine Room's Slab)							
M/one way slab	500	250	420	170	2,47	9	15

For bottom structures:

- Sloof/*tie beam*→ width: 80 cm, height: 110 cm
- poer → length = width: 300 cm, thickness: 150 cm
- spile/foundation → diameter: 60 cm (approximately)

4.2. Reinforcement of Secondary Structure

4.2.1. Preface

Reinforcement of secondary structure consists of reinforcement of slab, reinforcement of secondary beam, reinforcement of stair, reinforcement of elevator beam, and reinforcement of ramp. Beside the reinforcement of secondary structure, in this chapter will be explained about calculation of lifting point of precast elements such as slab and secondary beam. Note that the concrete's age is important for the calculation (see Table 4.7).

Table 4.7. The Comparison of Concrete's Age (Factor)
(source: PBBI, 1971)

Concrete's age (days)	3	7	14	21	28	90	365
Ordinary Portland cement	0,40	0,65	0,88	0,95	1,00	1,20	1,35

4.2.2. Reinforcement of Precast Floor Slab

4.2.2.1. General

The early design of precast slab LA1 consists of 10 cm of precast slab and 5 cm of *overtopping* concrete. The calculation and designing process of slab's reinforcementare divided into two conditions below

a. before monolith condition

this condition occurs when overtopping concrete does not become monolith yet with precast slab element and it does not bear the load (condition a and condition b)

b. after monolith condition

this condition occurs when overtopping concrete becomes monolith with precast slab element and it bears the load together with precast slab element (condition c).

Note that the assumption of slab designing is based on both condition above. When before monolith condition occurs, the slab is assumed to have two simple supports (roller-pins) and when after monolith condition happens, the slab is assumed to have elastic fixed support. The reinforcement itself is also divided into two conditions above and will be chosen whichever the greater result of reinforcement calculation or the combination of both calculation. The calculation will use the conditions based on SNI 2847:2013.

4.2.2.2. General Data

The general data for reinforcement of slab will be shown below

slab's thickness (overall)	= 15 cm
slab type	= floor, type LA1, etc.
dimension	= 500x250x10 cm^3
diameter of bar (D)	= 13 mm
L_y (clear span)	= 4600mm
L_x (clear span)	= 2150mm
L_y/L_x	= 2,139
b (L_y actual)	= 4680 mm
a (L_x actual)	= 2230 mm

f'_c(28 days) = 30 MPa
f_y = 390 MPa

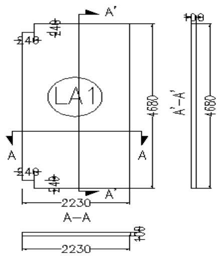

Figure 4.17. The precast floor slab
(source: Private Documentation)

4.2.2.3. Load Calculation

<u>Before Monolith Condition</u>

In this condition itself will be divided into two conditions below

1. condition a

when the *overtopping* concrete is not installed yet and the load consists of working load (construction process' load) and precast slab element's load

2. condition b

when the *overtopping* concrete is already installed (not yet in monolith condition) and the load consists of *overtopping* concrete's load, precast slab element's load and construction process' load

Then the most critical condition between two conditions above will be used for load calculation. Note that the *overtopping* concrete's thickness needs to be added as 2 cm for anticipating the stacks when casting process of the *overtopping* concrete occurs (only for load calculation purpose).

1. dead load

slab	: $0{,}10x2400\ kg/m^2$	$= 240\ kg/m^2$
overtopping concrete	: $(0{,}05+0{,}02)x2400\ kg/m^2$	$= 168\ kg/m^2$
DL (total)	:	$= 408\ kg/m^2$

2. live load

construction load	: $200\ kg/m^2$	$= 200\ kg/m^2$
LL (total)	:	$= 200\ kg/m^2$

After Monolith Condition

In this condition itself will be divided only intoone condition below

1. condition c

when the *overtopping* concrete is installed then bear loads together with slab precast element and the load consists of precast slab element's load and loads such as plumbing, ducting, etc.

1. dead load

slab (overall)	: $0{,}15x2400\ kg/m^2$	$= 360\ kg/m^2$
plafond	: $11\ kg/m^2$	$= 11\ kg/m^2$
hanger	: $7\ kg/m^2$	$= 7\ kg/m^2$
plumbing and ducting	: $25\ kg/m^2$	$= 25\ kg/m^2$
sanitary	: $20\ kg/m^2$	$= 20\ kg/m^2$
mortar mixture (2 cm)	: $2x21\ kg/m^2$	$= 42\ kg/m$
tile (2 cm)	: $2x24\ kg/m^2$	$= 48\ kg/m^2$
DL (total)	:	$= 513\ kg/m^2$

2. live load

live load	: $1{,}92\ kN/m^2$	$= 192\ kg/m^2$
LL (total)	:	$= 192\ kg/m^2$

4.2.2.4. Load Combination of Slab

The combination of load for slab is based on SNI 2847:2013 paragraph 9.2.1. The combination itself is divided into three conditions (see previous pages). The load combination will use the ultimate load, Qu, as 1.2DL + 1.6LL.

load combination of slab

condition a

(precast's age = 14 days)

$Qu = 1{,}2\times240 + 1{,}6\times200 = 608\ kg/m^2$

condition b

(precast's age = 14 days, *overtopping*'s age = 0 day)

$Qu = 1{,}2\times408 + 1{,}6\times200= 809{,}6\ kg/m^2$

condition c

(precast's age > 30 days, *overtopping*'s age > 30 days)

$Qu = 1{,}2\times513 + 1{,}6\times192 = 922{,}8 kg/m^2$

4.2.2.5. Moment Calculation of Slab

The moment calculation of slab before monolith condition (condition a and condition b) will use moment equation $Mu = 1/8 \times q \times L^2$. The moment calculation of slab after monolith condition (condition c) will use the equations in PBBI 1971 Table 13.3.2. For condition a and condition b, the slab's support will be assumed as simple support (roller-pins). For condition c, the slab is assumed as condition II in PBBI (1971) Table 13.3.2 which it fixed (elastic condition) at its all four sides. The moment calculation itself will be divided into three condition (condition a, condition b, and condition c). The equation for moment calculation for condition a and condition b will be shown below

$Mlx = 1/8 \times q \times L_x^2$ (for transversal direction of slab)

$Mly = 1/8 \times qx\ L_y^2$ (for longitudinal direction of slab)

in which q is the distributed load and L is the clear span ofslab (L_y = 460 cm, L_x = 215 cm).

The equation for moment calculation for condition c is based on PBBI (1971). According to PBBI (1971), the X values are determined by L_y/L_x value which as 2,139. The X values are X1 = 62, X2 = 34, X3 = 62, X4 = 34. The equation of moment values are show below

Mlx (+) = 0,001 x qx Lx^2x X1

Mly (+) = 0,001 x qx Lx^2x X2

Mtx (-) = 0,001 x qx Lx^2x X3

Mty (-) = 0,001 x qx Lx^2x X4

moment calculation of slab

condition a (q = 608 kg/m^2, L_x =215 cm, L_y = 460 cm)

Mlx (+) = 351,31 kgm

Mly (+) = 1608,16 kgm

condition b (q = 809,6 kg/m^2, L_x =215 cm,L_y = 460 cm)

Mlx (+) = 467,797 kgm

Mly (+) = 2141,392 kgm

condition c (q = 922,8 kg/m^2, L_x =215 cm)

Mlx (+) = 264,469 kgm

Mly (+) = 145,032 kgm

Mtx (-) = 264,469 kgm

Mty (-) = 145,032 kgm

The Mu values are taken as

condition a

Mu = 1608,16 kgm (+)

condition b

Mu = 2141,392 kgm (+)

condition c

Mu = 264,469 kgm (+)

and then the Mu values are divided into two conditions which it will be taken as the greatest result between condition a and condition b for before monolith condition, then Mu value in

condition c is considered as Mu value for after monolith condition. (note : "+" is compression, and "–" is tension)
before monolith condition
Mu = 2141,392 kgm (+)
after monolith condition
Mu = 264,469 kgm (+)

4.2.2.6. Calculation of Reinforcement Bar

The calculation of reinforcement bar will be divided into two conditions (before monolith condition and after monolith condition, Figure 4.18). The general data for calculation of reinforcement bar of slab is shown below

slab's dimension (actual)	= 4680 mm ×2230 mm
slab's thickness (precast)	= 100 mm
overtopping's thickness	= 50 mm
clear cover	= 30 mm
bar's diameter (D)	= 13 mm
f'_c(28 days)	= 30 MPa
f_y	= 390 MPa
$\beta_1 = L_y/L_x$	= 2,139 (one way slab)

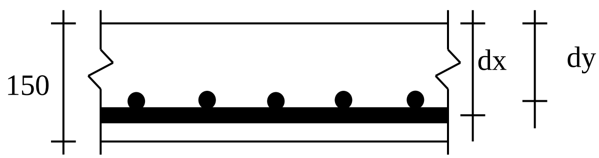

Figure 4.18. The floor slab's cross section
(after monolith condition)
(source: Private Documentation)

<u>before monolith condition</u>

dx = 100 - 30 - (13/2)	= 63,5 mm
dy = 100 - 30 - 13 - (13/2)	= 50,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 14 days = 26,4 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$
$\beta_1 = 0{,}85$
$As = \pi/4xd^2 = 132{,}78\ mm^2$
$a = Asxf_y/(0{,}85xf'_cxb) = 2{,}30\ mm$
$c = a/\beta_1 = 2{,}71\ mm$
$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}052 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)
$m = f_y/(0{,}85xf'_c) = 17{,}37$
$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1)

<u>after monolith condition</u>
dx = 150 - 30 - (13/2) = 113,5 mm
dy = 150 - 30 - 13 - (13/2) = 100,5 mm
For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)
$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$
$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$
$As = \pi/4xd^2 = 132{,}78\ mm^2$
$a = Asxf_y/(0{,}85xf'_cxb) = 2{,}03\ mm$
$c = a/\beta_1 = 2{,}43\ mm$
$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}12 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)
$m = f_y/(0{,}85xf'_c) = 15{,}29$
$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1)

If the slab is determined as one way slab, then the main reinforcement bar of slab is transversal reinforcement bar only and the longitudinal reinforcement bar will funcionate as temperature and shrinkage bearer. Both of them will use spiral bar D13 (diameter = 13 mm, As = 132,7857 mm^2).

<u>Reinforcement Bar Before Monolith Condition</u>

The general data for reinforcement bar of slab before monolith condition will be shown below

slab's thickness = 100 mm, f'_c = 26,4 MPa (14 days)
clear cover = 30 mm
bar's diameter = 13 mm, $As = \pi/4xd^2 = 132{,}78\ mm^2$
b = 1000 mm
d = 63,5 mm
ϕ = 0,9
M_u = 2141,392 kgm = 21413920 Nmm

<u>Main Bar/Transversal Bar</u>

$M_n = M_u/\phi = 23793244$ Nmm

$$Rn = \frac{Mn}{b \times d^2} = 5{,}90 N / mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0179$$

$\rho_{need} > \rho_{min} = 0{,}0179 > 0{,}0020$
use $\rho = \rho_{need} = 0{,}0179$
$As_{need} = \rho xbxd = 1137{,}97mm^2$
$n = As_{need}/As_{bar} = 1137{,}97mm^2/132{,}7857\ mm^2 = 8{,}57$
use n = 10 → $As_{use} = nxAs_{bar} = 1327{,}857\ mm^2$
$As_{use} > As_{need} = 1327{,}857\ mm^2 > 1137{,}97\ mm^2$ (OK)
s = b/n = 1000 mm/10 = 100 mm
So, the transversal bar will use D13-100.

<u>Shrinkage Bar/Longitudinal Bar</u>

$As_{need} = 0{,}002xbxd = 127\ mm^2$ (SNI, 2013 paragraph 7.12.2.1)
$n = As_{need}/As_{bar} = 127\ mm^2/132{,}7857\ mm^2 = 0{,}95$
use n = 5 → $As_{use} = nxAs_{bar} = 663{,}93\ mm^2$
$As_{use} > As_{need} = 663{,}93\ mm^2 > 127\ mm^2$ (OK)
s = b/n = 1000 mm/5 = 200 mm
So, the longitudinal bar will use D13-200.

Development Bar's Length

ℓ_{dh} = 8xd_b = 8x13 mm = 104 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 0,24xfy/d_bx$(f'_c)^{0,5}$ = 37 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 150 mm.

Reinforcement Bar After Monolith Condition

The general data for reinforcement bar of slab after monolith condition will be shown below

slab's thickness	= 150 mm, f'_c = 30 MPa (> 30 days)
clear cover	= 30 mm
bar's diameter	= 13 mm, As = $\pi/4xd^2$ = 132,78 mm^2
b	= 1000 mm
d	= 113,5 mm
ϕ	= 0,9

M_u = 264,469 kgm = 2644698,7 Nmm

Main Bar/Transversal Bar

$M_n = M_u/\phi$ = 2938554 Nmm

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0{,}228 N / mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}000587$$

$\rho_{need} < \rho_{min}$ = 0,000587 < 0,002
use $\rho = \rho_{min}$ = 0,002
As_{need} = ρxbxd = 227 mm^2
n = As_{need}/As_{bar} = 227 mm^2/132,7857 mm^2 = 1,709
use n = 5 → As_{use} = nxAs_{bar} = 663,928 mm^2
$As_{use} > As_{need}$= 663,92857 mm^2 > 227 mm^2 (OK)
s = b/n = 1000 mm/5 = 200 mm
So, the transversal bar will use D13-200.

<u>Shrinkage Bar/Longitudinal Bar</u>
As_{need} = 0,002xbxd = 227 mm^2
n = As_{need}/As_{bar} = 227 mm^2/132,7857 mm^2 = 1,71
use n = 5 → As_{use} = $nxAs_{bar}$ = 663,92857 mm^2
$As_{use} > As_{need}$= 663,92857 mm^2 > 227 mm^2 (OK)
s = b/n = 1000 mm/5 = 200 mm
So, the longitudinal bar will use D13-200.

<u>Development Bar's Length</u>
ℓ_{dh} = $8xd_b$ = 8x13 mm = 104 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = $0,24xfy/d_bx(f'_c)^{0,5}$ = 39,5 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 150 mm.

<u>Reinforcement Bar Due to Lifting Process'Moment</u>

The general data for reinforcement bar of slab due to lifting process' moment will be shown below

slab's thickness	= 100 mm, f'_c = 26,4 MPa (14 days)
clear cover	= 30 mm
bar's diameter	= 13 mm, As = $\pi/4xd^2$ = 132,78 mm^2
b	= 1000 mm
d	= 63,5 mm
ϕ	= 0,9

M_u = 177,1002 kgm = 1771002 Nmm (see 4.2.2.8.2)

<u>Main Bar/Transversal Bar</u>
$M_n = M_u/\phi$ = 1967780 Nmm

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b}\times d^2} = 0{,}488 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1-\sqrt{1-\frac{2\times m\times Rn}{fy}}\right) = 0{,}001265$$

$\rho_{nee\ d} < \rho_{min}$ = 0,001265 < 0,002
use $\rho = \rho_{min}$ = 0,002
$As_{need} = \rho x b x d$ = 127 mm^2
n = As_{need}/As_{bar} = 127 mm^2/132,7857 mm^2 = 0,956
use n = 4 → As_{use} = 4xAs_{bar} = 531,1429 mm^2
$As_{use} > As_{need}$= 531,1429 mm^2 > 127 mm^2 (OK)
s = b/4 = 1000 mm/4 = 250 mm
So, the transversal bar will use D13-250.

<u>Shrinkage Bar/Longitudinal Bar</u>
As_{need} = 0,002xbxd = 127 mm^2
n = As_{need}/As_{bar}= 127 mm^2/132,7857 mm^2 = 0,956
use n = 4 → As_{use} = nxAs_{bar} = 531,1428571 mm^2
$As_{use} > As_{need}$= 531,1428571 mm^2 > 127 mm^2 (OK)
s = b/n = 1000 mm/4 = 250 mm
So, the longitudinal bar will use D13-250.

<u>Development Bar's Length</u>
ℓ_{dh} = 8xd_b = 8x13 mm = 104 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 0,24xfy/d_bx$(f'_c)^{0,5}$ = 37 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 150 mm

For the recapitulation of reinforcement bar will be shown in Table 4.8.

Table 4.8. Recapitulation of Reinforcement Bars of Floor Slab

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Before Monolith Condition										
Type	Dimension (cm)	Flexural Bar					Longitudinal Bar			Stud
		As need (mm^2)	As use (mm^2)	s (mm)	n	Notation	As use (mm^2)	s (mm)	Notation	Notation
LA1	468 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
LA2	468 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
LB1	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
LB2	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
LB3	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
LB4	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
LC1	468 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300

After Monolith Condition										
Type	Dimension	Flexural Bar					Longitudinal Bar			Stud
		As need (mm^2)	As use (mm^2)	s (mm)	n	Notation	As use (mm^2)	S (mm)	Notation	Notation
LA1	468 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
LA2	468 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
LB1	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
LB2	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
LB3	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
LB4	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
LC1	468 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300

Due to Lifting Process' Moment										
Type	Dimension	Flexural Bar					Longitudinal Bar			Stud
		As need (mm^2)	As use (mm^2)	s (mm)	n	Notation	As use (mm^2)	S (mm)	Notation	Notasi
LA1	468 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
LA2	468 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
LB1	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
LB2	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
LB3	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
LB4	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
LC1	468 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300

Use reinforcement bar before monolith condition (flexural bar D13-100 and longitudinal bar
D13-200 with shear connector D13-300).

4.2.2.7. Shear Connector Reinforcement of Slab

In the designing process of element which it consists of precast element and *overtopping* concrete (or cast in place concrete), the need of shear connector is relly important. Because the shear connector funcionates to bind the precast element and *overtopping* concrete (or cast in place concrete). Besides, the shear connector (stud) also has function to distribute the inner forces which retained by elements into horizontal shear force on the elements' surface intersection.

The horizontal shear force, which occurs on composite element, can be divided into two conditions below (Figure 4.19)

Condition 1 : the compressive force which is produced by composite element is less than the compressive force produced by cast in place concrete element (C < Cc, Vnh = C, C = T)

Condition 2 : the compressive force which is produced by composite element is greater than the compressive force produced by cast in place concrete element (C > Cc, Vnh = C, C<T)

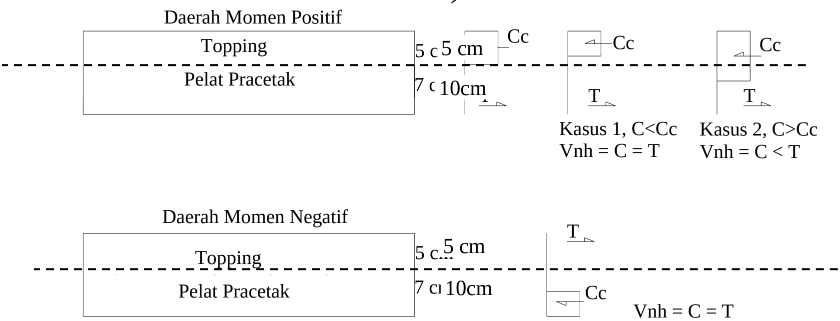


Figure 4.19. The diagram of horizontal shear force (source: Private Documentation)

For floor slab (LA1, etc.) (500 cm x 250 cm)
$Cc = 0{,}85 \times f'_c \times A_{topping}$; $f'_c = 30$ MPa (> 30 days)
$A_{topping}$ is assumed as b x d in which b = 1000 mm and d is the thickness of *overtopping* concrete
Cc = 0,85x30x1000x50 mm = 1275000 N = 1275 kN
use shear connector D 13 mm
$As_{stud} = 132{,}78\ mm^2 \rightarrow A_s$
Use condition $\rightarrow$(Vnh = C, C = T)
$T = A_s \times f_y stud = 132{,}78 \times 390$ N = 51786 N = 51,786 kN
T = Vnh = C = 51,786 kN
$0{,}55\ Ac = 0{,}55 \times b_v \times d$ (SNI, 2013 paragraph 17.5.3.1)
Ac is assumed as $b_v \times d$ in which $b_v = 1000$ mm and d is the thickness of composite element (with *overtopping* concrete)
0,55 Ac = 0,55x1000x15 N = 82500 N = 82,5 kN
Vnh < 0,55 Ac = 51,786 kN <82,5 kN (OK)
(see SNI, 2013 paragraph 17.5.3.1)
$\phi V_s = \phi(n)(A_{se}f_{ut})$ (PCI, 2004)
$0{,}65 \times 82{,}5\ kN = 0{,}65 \times n \times 132{,}78\ mm^2 \times 390\ N/mm^2$
53625 N = 33659,73 N x n
$n = 1{,}59 \approx 3$
$s = 1000\ mm/3 = 333{,}33\ mm \approx 300\ mm$
Use shear connector D13-300.

4.2.2.8. Reinforcement Bar Due to Lifting Process of Slab

When lifting process occurs, the need of reinforcement bar due to lifting process, is necessary. Because when lifting process occurs, it will produce moment force. So, the moment force must be calculated and then the reinforcement bar designed so it can bear the moment force due to lifting process. In this chapter will use four lifting points of slab (Figure 4.20).

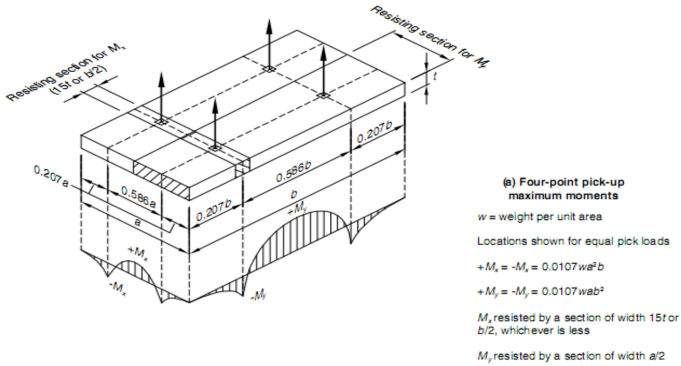


Figure 4.20. Four lifting points of floor slab (source: PCI, 2004)

The general data for calculating lifting bar will be shown below

f'$_c$ (14 days)	= 26,4 MPa
f_y	= 240 MPa = 2400 kg/cm^2
slab's type	= floor, (type LA1, etc.)
dimension	= 500 x 250 mm^2
n (number of points)	= 4
a (slab's width)	= 2230mm
b (slab's length)	= 4680mm
k (shock coefficient)	= 1,2
t (slab's thickness	= 10 cm

(before monolith condition)

w (=t x 2400 kg/m^2)	= 240 kg/m^2
k x DL (= slab's weight + shear connector (1% slab's weight)	= 3035,74 kg
Qu (=1,4 DL x k)	= 4250,036 kg
P (=Qu/n)	= 1062,509 kg

Designing The Lifting Bar

The designing process of lifting bar will use equation below

$$\phi_{\text{tulangamamgkat}} \geq \sqrt{\frac{n \times P}{\pi \times \sigma}}$$

in which n is the number of liting points and σ is $f_y/1{,}5$ = 1600 kg/cm^2 (SF = 1,5, f_y = 240 MPa).

So, the value of ϕ (diameter of lifting bar) is

$$\phi = \sqrt{\frac{4\ x\ 1062{,}509\ kg}{\frac{22}{7}\ x\ 1600\ kg/cm^2}} = 0{,}919\ cm$$

use lifting bar ϕ = 10 mm instead (As = 78,5714 mm^2).

Moment Calculation Due to Lifting Process

The moment calculatin will be divided into two calculations based on slab's direction (direction a and direction b). Direction a is slab's transversal direction and direction b is slab's longitudinal direction (Figure 4.21 and Figure 4.22).

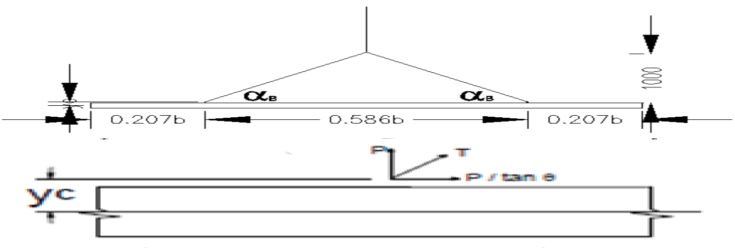


Figure 4.21. Slab's longitudinal direction (direction b) when lifting process occurs
(source: Private Documentation)

<u>direction b</u>

b = 468 cm
n (number of point) = 2
0,207 b = 0,97 m
0,586 b = 2,74 m
rope's height = 1000 mm
$\alpha_b{}^o$ = $\tan^{-1}$ ((rope's height/(0,586 b/2))
= $36{,}12^o$
T_b = P x sin $\alpha_b{}^o$ = 626,43 kg
y_c = 0,5 t + 1" = 0,0754 m
M_b = 0,0107 x w x a^2 x b = 59,765 kgm
$M_{b'}$ = P x y_c/tan $\alpha_b{}^o$ = 109,755 kgm
(moment due to lifting process)
M_b (total) = 169,52 kgm
M_b is retained by area of 15t or b/4 (choose the least value between)
15t = 150 cm
b/4 = 234 cm
use 15t = 150 cm
Z_b = 1/6 x 15t x t^2 = 2500 cm^3

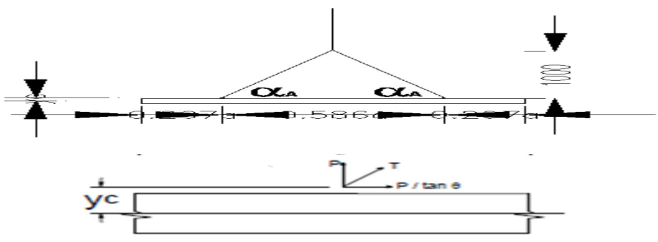


Figure 4.22. Slab's transversal direction (direction a) when lifting process occurs
(source: Private Documentation)

<u>direction a</u>

a	= 223 cm
n (number of point)	= 2
0,207 a	= 0,47 m
0,586 a	= 1,29 m
rope's height	= 1000mm
α_a^o	= $\tan^{-1}$ ((rope's height/(0,586 a/2))
	= $57{,}17^o$
T_a	= P x sin α_a^o = 892,88 kg
y_c	= 0,5 t + 1" = 0,0754 m
M_a	= 0,0107 x w x a x b^2 = 125,427 kgm
$M_{a'}$	= P x y_c/tan α_a^o = 51,673 kgm

(moment due to lifting process)

M_a (total)	= 177,100 kgm

M_a is retained by area of a/2

a/2	= 111,5 cm
Z_a	= 1/6 x a/2 x t^2 = 1858,33 cm^3

Checking The Crack Factor

The lifting bar will use ϕ = 10 mm plain bar.
Checking the f_{cr} value (assumed the concrete's age is 14 days)

$f_{cr} = 0{,}62\ \lambda\sqrt{f'c}$ /SF (SNI, 2013 paragraph 9.5.2.3)
(λ = 1, SNI, 2013 paragraph 8.6.1)
(safety factor = 2)

f_{cr}	= 1,593 MPa
f_b	= M_b (total)/Z_b = 0,56 MPa
f_a	= M_a (total)/Z_a = 0,618 MPa
$f_b < f_{cr}$	= 0,56 MPa < 1,593 MPa (OK)
$f_a < f_{cr}$	= 0,618 MPa < 1,593 MPa (OK)

Checking The Strand Cable

The strand cable for lifting process will use seven-wire strand 5/16 inch diameter with f_{pu} = 250 ksi (see PCI, 2004 Design Aid Table 11.2.3).

f_{pu}	= 250 ksi
A	= 0,058 in^2
f_{pu} x A	= 14,5 kips = 64499,2132 N
SF	= 2
Af_{pu}/SF	= 3224,96kg
T_b	= 626,43 kg
T_a	= 892,88kg
$T_b < Af_{pu}/SF$	= 626,43 kg < 3224,96 kg (OK)
$T_a < Af_{pu}/SF$	= 892,88 kg < 3224,96 kg (OK)

Table 4.9. The Properties of Strand Cable
(source: PCI, 2004)

Seven –Wire Strand, f_{pu} = 250 ksi						
Nominal Diameter, in	1/4	5/16	3/8	7/16	1/2	3/5
Area, square in.	0,036	0,058	0,080	0,108	0,144	0,216
Weight, plf	0,12	0,20	0,27	0,37	0,49	0,74
$0{,}7f_{pu}A_{ps}$, kips	6,3	10,2	14,0	18,9	25,2	37,8
$0{,}7f_{pu}A_{ps}$, kips	7,2	11,6	16,0	21,6	28,8	43,2
f_{pu}, A_{ps}, kips	9,0	14,5	20,0	27,0	36,0	54,0

4.2.2.9. Checking The Deflection of Slab

The deflection of slab is calculated using equation below

δ (deflection) = (5/384) x (q_u x L^4/EI)

δ_{max} = L_n/480 (in cm) (SNI 2847:2013paragraph 9.5.3.1) in which q_u is taken as 922,8 kg/m^2x 1 m (see previous paragraph), I is the moment of inertia of slab ($I = 1/12xbxh^3$), and E equals $4700x(f'c)^{0,5}$, f'c in 14 days

Deflection (Longitudinal, L_n = 460 cm, b =L_n, h = 15 cm)

$\delta_b = (5/384) \times (q_u \times L^4/EI) = 0,172$ cm

$\delta_{max} = L_n/480 = 0,958$ cm

$\delta_b < \delta_{max}$ (OK)

Deflection (Transversal, S_n = 215 cm, b =S_n, h = 15 cm)

$\delta_a = (5/384) \times (q_u \times L^4/EI) = 0,0175$ cm

$\delta_{max} = L_n/480 = 0,447$ cm

$\delta_a < \delta_{max}$ (OK)

4.2.3. Reinforcement of Precast Roof Slab

4.2.3.1. General

The early design of precast slab AA1 consists of 9 cm of precast slab and 5 cm of *overtopping* concrete. The calculation and designing process of slab's reinforcementare divided into two conditions below

a. before monolith condition

this condition occurs when overtopping concrete does not become monolith yet with precast slab element and it does not bear the load (condition a and condition b)

b. after monolith condition

this condition occurs when overtopping concrete becomes monolith with precast slab element and it bears the load together with precast slab element (condition c).

Note that the assumption of slab designing is based on both condition above. When before monolith condition occurs, the slab is assumed to have two simple supports (roller-pins) and when after monolith condition happens, the slab is assumed to have elastic fixed support. The reinforcement itself is also divided into two conditions above and will be chosen whichever the greater result of reinforcement calculation or the combination of both calculation. The calculation will use the conditions based on SNI 2847:2013.

4.2.3.2. General Data

The general data for reinforcement of slab will be shown below

slab's thickness (overall)	= 14 cm
slab type	= roof, (type AA1, etc.)
dimension	= 500x250x9 cm^3
diameter of bar (D)	= 13 mm
L_y (clear span)	= 4600mm
L_x (clear span)	= 2150mm
L_y/L_x	= 2,139
b (L_y actual)	= 4680 mm
a (L_x actual)	= 2230 mm
f'_c (28 days)	= 30 MPa
f_y	= 390 MPa

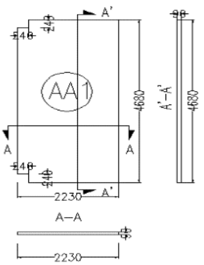


Figure 4.23. The precast slab type AA1
(source: Private Documentation)

4.2.3.3. Load Calculation

<u>Before Monolith Condition</u>

In this condition itself will be divided into two conditions below

1. condition a

when the *overtopping* concrete is not installed yet and the load consists of working load (construction process' load) and precast slab element's load

2. condition b

when the *overtopping* concrete is already installed (not yet in monolith condition) and the load consists of *overtopping* concrete's load, precast slab element's load and construction process' load

Then the most critical condition between two conditions above will be used for load calculation. Note that the *overtopping* concrete's thickness needs to be added as 2 cm for anticipating the stacks when casting process of the *overtopping* concrete occurs (only for load calculation purpose).

1. dead load

slab	: 0,09x2400 kg/m^2	= 216 kg/m^2
overtopping concrete	: (0,05+0,02)x2400 kg/m^2	= 168 kg/m^2
DL (total)	:	= 384 kg/m^2

2. live load

construction load	: 200 kg/m^2	= 200 kg/m^2
LL (total)	:	= 200 kg/m^2

After Monolith Condition

In this condition itself will be divided only intoone condition below

1. condition c

when the *overtopping* concrete is installed then bear loads together with slab precast element and the load consists of precast slab element's load and loads such as plumbing, ducting, etc.

1. dead load

slab (overall)	: 0,14x2400 kg/m^2	= 336 kg/m^2
plafond	: 11 kg/m^2	= 11 kg/m^2
hanger	: 7 kg/m^2	= 7 kg/m^2
plumbing and ducting	: 25 kg/m^2	= 25 kg/m^2
asphalt (1 cm)	: 14 kg/m^2	= 14 kg/m^2
mortar mixture (2 cm)	: 2x21 kg/m^2	= 42 kg/m
DL (total)	:	= 435 kg/m^2

2. live load

live load	: 0,96 kN/m^2	= 96 kg/m^2
rain	: 20 kg/m^2	= 20 kg/m^2
LL (total)	:	= 116kg/m^2

4.2.3.4. Load Combination of Slab

The combination of load for slab is based on SNI 2847:2013 paragraph 9.2.1. The combination itself is divided into three conditions (seeprevious pages). The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of slab

condition a

(precast's age = 14 days)

Qu = 1,2×216 + 1,6×200= 579,2 kg/m^2

condition b

(precast's age = 14 days, *overtopping*'s age = 0 day)

Qu = 1,2×384+ 1,6×200= 700,8 kg/m^2

condition c

(precast's age > 30 days, *overtopping*'s age > 30 days)

Qu = 1,2×435 + 1,6×116 = 707,6 kg/m^2

4.2.3.5. Moment Calculation of Slab

The moment calculation of slab before monolith condition (condition a and condition b) will use moment equation $Mu = 1/8 \times q \times L^2$. The moment calculation of slab after monolith condition (condition c) will use the equations in PBBI 1971 Table 13.3.2. For condition a and condition b, the slab's support will be assumed as simple support (roller-pins). For condition c, the slab is assumed as condition II in PBBI (1971) Table 13.3.2 which it fixed (elastic condition) at its all four sides. The moment calculation itself will be divided into three condition (condition a, condition b, and condition c). The equation for moment calculation for condition a and condition b will be shown below

$Mlx = 1/8 \times q \times L_x^2$(for transversal direction of slab)

$Mly = 1/8 \times qx\ L_y^2$(for longitudinal direction of slab)

in which q is the distributed load and L is the clear span ofslab (L_y = 460 cm, L_x = 215 cm).

The equation for moment calculation for condition c is based on PBBI (1971). According to PBBI (1971), the X values are determined by L_y/L_x value which as 2,139. The X values are X1 = 62, X2 = 34, X3 = 62, X4 = 34. The equation of moment values are show below

Mlx (+) = 0,001 x qx Lx^2x X1

Mly (+) = 0,001 x qx Lx^2x X2

Mtx (-) = 0,001 x qx Lx^2x X3

Mty (-) = 0,001 x qx Lx^2x X4

moment calculation of slab

condition a (q = 579,2 kg/m^2, L_x =215 cm, L_y = 460 cm)

Mlx (+) = 334,669 kgm

Mly (+) = 1531,98 kgm

condition b (q = 700,8 kg/m^2, L_x =215 cm,L_y = 460 cm)

Mlx (+) = 404,93 kgm

Mly (+) = 1853,62 kgm

condition c (q = 707,6 kg/m^2, L_x = 215 cm)
Mlx (+) = 202,79 kgm
Mly (+) = 111,21 kgm
Mtx (-) = 202,79 kgm
Mty (-) = 111,21 kgm

The Mu values are taken as
condition a
Mu = 1531,98 kgm (+)
condition b
Mu = 1853,62 kgm (+)
condition c
Mu = 202,79 kgm (+)

and then the Mu values are divided into two conditions which it will be taken as the greatest result between condition a and condition b for before monolith condition, then Mu value in condition c is considered as Mu value for after monolith condition.
before monolith condition
Mu = 1853,62 kgm (+)
after monolith condition
Mu = 202,79 kgm (+)

4.2.3.6. Calculation of Reinforcement Bar

The calculation of reinforcement bar will be divided into two conditions (before monolith condition and after monolith condition). The general data for calculation of reinforcement bar of slab is shown below

slab's dimension (actual)	= 4680 mm ×2230 mm
slab's thickness (precast)	= 90 mm
overtopping's thickness	= 50 mm
clear cover	= 30 mm
bar's diameter (D)	= 13 mm
f'_c(28 days)	= 30 MPa

f_y = 390 MPa
$\beta_1 = L_y/L_x$ = 2,139 (one way slab)

before monolith condition
dx = 90 - 30 - (13/2) = 53,5 mm
dy = 90 - 30 - 13 - (13/2) = 40,5 mm
For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 14 days = 26,4 MPa, see Table 4.7)
$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$
$\beta_1 = 0{,}85$
$As = \pi/4xd^2 = 132{,}78\ mm^2$
$a = Asxf_y/(0{,}85xf'_cxb) = 2{,}30$ mm
$c = a/\beta_1 = 2{,}71$ mm
$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}041 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)
$m = f_y/(0{,}85xf'_c) = 17{,}37$
$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1)

after monolith condition
dx = 140 - 30 - (13/2) = 103,5 mm
dy = 140 - 30 - 13 - (13/2) = 90,5 mm
For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)
$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$
$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$
$As = \pi/4xd^2 = 132{,}78\ mm^2$
$a = Asxf_y/(0{,}85xf'_cxb) = 2{,}03$ mm
$c = a/\beta_1 = 2{,}43$ mm
$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}047 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)
$m = f_y/(0{,}85xf'_c) = 15{,}29$
$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1)

If the slab is determined as one way slab, then the main reinforcement bar of slab is transversal reinforcement bar only and the longitudinal reinforcement bar will funcionate as temperature and shrinkage bearer. Both of them will use spiral bar D13 (diameter = 13 mm, As = 132,7857 mm^2).

<u>Reinforcement Bar Before Monolith Condition</u>

The general data for reinforcement bar of slab before monolith condition will be shown below

slab's thickness = 90 mm, f'_c = 26,4 MPa (14 days)
clear cover = 30 mm
bar's diameter = 13 mm, As = $\pi/4 x d^2$ = 132,78 mm^2
b = 1000 mm
d = 53,5 mm
ϕ = 0,9
M_u = 1853,62 kgm = 18536160 Nmm

<u>Main Bar/Transversal Bar</u>

$M_n = M_u/\phi$ = 20595733 Nmm

$$Rn = \frac{Mn}{b \times d^2} = 7{,}19 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}023$$

$\rho_{need} > \rho_{min}$ = 0,023 > 0,002
use $\rho = \rho_{need}$ = 0,023
$As_{need} = \rho x b x d$ = 1234,72 mm^2
n = As_{need}/As_{bar} = 1234,72 mm^2/132,7857 mm^2 = 9,29
use n = 10→ $As_{use} = n x As_{bar}$ = 1327,857 mm^2
$As_{use} > As_{need}$= 1327,857 mm^2 > 1234,72 mm^2 (OK)
s = b/n = 1000 mm/10 = 100 mm
So, the transversal bar will use D13-100.

<u>Shrinkage Bar/Longitudinal Bar</u>

As_{need} = 0,002xbxd = 107 mm^2
n = As_{need}/As_{bar} = 107 mm^2/132,7857 mm^2 = 0,81
use n = 5 → As_{use} = $nxAs_{bar}$ = 663,93 mm^2
$As_{use} > As_{need}$= 663,93 mm^2 >107 mm^2 (OK)
s = b/n = 1000 mm/5 = 200 mm
So, the longitudinal bar will use D13-200.

Development Bar's Length

$\ell_{dh} = 8xd_b$ = 8x13 mm = 104 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_bx(f'_c)^{0,5}$ = 37 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 150 mm.

Reinforcement Bar After Monolith Condition

The general data for reinforcement bar of slab after monolith condition will be shown below

slab's thickness	= 140 mm, f'_c = 30 MPa (> 30 days)
clear cover	= 30 mm
bar's diameter	= 13 mm, As = $\pi/4xd^2$ = 132,78 mm^2
b	= 1000 mm
d	= 103,5 mm
ϕ	= 0,9

M_u = 202,79 kgm = 2027946,2 Nmm (see 4.2.3.8.2)

Main Bar/Transversal Bar

$M_n = M_u/\phi$ = 2253274 Nmm

$$Rn = \frac{Mn}{b \times d^2} = 0{,}21 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}00055$$

$\rho_{need} < \rho_{min}$ = 0,00055 < 0,002

use $\rho = \rho_{min} = 0{,}002$
$As_{need} = \rho x b x d = 207\ mm^2$
$n = As_{need}/As_{bar} = 207\ mm^2/132{,}7857\ mm^2 = 1{,}559$
use $n = 5 \rightarrow As_{use} = nxAs_{bar} = 663{,}928\ mm^2$
$As_{use} > As_{need} = 663{,}92857\ mm^2 > 207\ mm^2$ (OK)
$s = b/n = 1000\ mm/5 = 200\ mm$
So, the transversal bar will use D13-200.

<u>Shrinkage Bar/Longitudinal Bar</u>
$As_{need} = 0{,}002xbxd = 207\ mm^2$
$n = As_{need}/As_{bar} = 207\ mm^2/132{,}7857\ mm^2 = 1{,}559$
use $n = 5 \rightarrow As_{use} = nxAs_{bar} = 663{,}92857\ mm^2$
$As_{use} > As_{need} = 663{,}92857\ mm^2 > 207\ mm^2$ (OK)
$s = b/n = 1000\ mm/5 = 200\ mm$
So, the longitudinal bar will use D13-200.

<u>Development Bar's Length</u>
$\ell_{dh} = 8xd_b = 8x13\ mm = 104\ mm$
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 150\ mm$
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_b x(f'_c)^{0{,}5} = 39{,}43\ mm$
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use $\ell_{dh} = 150$ mm.

<u>Reinforcement Bar Due toLifting Process' Moment</u>

The general data for reinforcement bar of slab due to lifting process' moment will be shown below

slab's thickness	= 90 mm, f'_c = 26,4 MPa (14 days)
clear cover	= 30 mm
bar's diameter	= 13 mm, $As = \pi/4xd^2 = 132{,}78\ mm^2$
b	= 1000 mm
d	= 53,5 mm
ϕ	= 0,9

$M_u = 156{,}3062$ kgm = 1563062 Nmm (see paragraph 4.2.3.8)

<u>Main Bar/Transversal Bar</u>

$M_n = M_u/\phi = 1736736$Nmm

$$\text{Rn} = \frac{\text{Mn}}{\text{b} \times d^2} = 0{,}606 N/mm^2$$

$$\rho_{\text{need}} = \frac{1}{\text{m}}\left(1 - \sqrt{1 - \frac{2 \times \text{m} \times \text{Rn}}{\text{fy}}}\right) = 0{,}001577$$

$\rho_{need} < \rho_{min} = 0{,}001577 < 0{,}002$

use $\rho = \rho_{min} = 0{,}002$

$As_{need} = \rho x b x d = 107$ mm^2

n = As_{need}/As_{bar} = 107 mm^2/132,7857 mm^2 = 0,81

use n = 4 → $As_{use} = 4xAs_{bar}$ = 531,1429 mm^2

$As_{use} > As_{need}$= 531,1429 mm^2 > 107 mm^2 (OK)

s = b/4 = 1000 mm/4 = 250 mm

So, the transversal bar will use D13-250.

<u>Shrinkage Bar/Longitudinal Bar</u>

As_{need} = 0,002xbxd = 107 mm^2

n = As_{need}/As_{bar} = 107 mm^2/132,7857 mm^2 = 0,81

use n = 4→ $As_{use} = nxAs_{bar}$ = 531,1429 mm^2

$As_{use} > As_{need}$= 531,1429 mm^2 > 107 mm^2 (OK)

s = b/n = 1000 mm/4 = 250 mm

So, the longitudinal bar will use D13-250.

<u>Development Bar's Length</u>

$\ell_{dh} = 8xd_b$ = 8x13 mm = 104 mm

(SNI, 2013 paragraph 12.5.1)

ℓ_{dh} = 150 mm

(SNI, 2013 paragraph 12.5.1)

$\ell_{dh} = 0{,}24xfy/d_bx(f'_c)^{0,5}$ = 37 mm

(SNI, 2013 paragraph 12.5.2)

So, the development bar's length will use ℓ_{dh} = 150 mm.

For the recapitulation of reinforcement bar will be shown in Table 4.10.

Table 4.10. Recapitulation of Reinforcement Bars of Roof Slab

Before Monolith Condition										
Type	Dimension (cm)	Flexural Bar					Longitudinal Bar			Stud
		As need (mm^2)	As use (mm^2)	s (mm)	n	Notation	As use (mm^2)	s (mm)	Notation	Notation
AA1	468 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
AA2	468 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
AB1	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
AB2	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
AB3	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
AB4	448 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300
AC1	468 x 223	1109,528	1327,857	100	10	D13-100	663,93	200	D13-200	D13-300

After Monolith Condition									
Type	Dimension	Flexural Bar					Longitudinal Bar			Stud
		As need (mm^2)	As use (mm^2)	s (mm)	n	Notation	As use (mm^2)	S (mm)	Notation	Notation
AA1	468 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
AA2	468 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
AB1	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
AB2	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
AB3	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
AB4	448 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300
AC1	468 x 223	227	663,93	200	5	D13-200	663,93	200	D13-200	D13-300

Due to Lifting Process' Moment										
Type	Dimension	Flexural Bar					Longitudinal Bar			Stud
		As need (mm^2)	As use (mm^2)	s (mm)	n	Notation	As use (mm^2)	S (mm)	Notation	Notasi
AA1	468 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
AA2	468 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
AB1	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
AB2	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
AB3	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
AB4	448 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300
AC1	468 x 223	127	531,14	250	4	D13-250	531,14	250	D13-250	D13-300

Use reinforcement bar beforemonolith condition (flexural bar D13-100 and longitudinal bar
D13-200 with shear connector D13-300).

4.2.3.7. Shear Connector Reinforcement of Slab

In the designing process of element which it consists of precast element and *overtopping* concrete (or cast in place concrete), the need of shear connector is relly important. Because the shear connector funcionates to bind the precast element and *overtopping* concrete (or cast in place concrete). Besides, the shear connector (stud) also has function to distribute the inner forces which retained by elements into horizontal shear force on the elements' surface intersection.

The horizontal shear force, which occurs on composite element, can be divided into two conditions below (Figure 4.24)

Condition 1 : the compressive force which is produced by composite element is less than the compressive force produced by cast in place concrete element (C < Cc, Vnh = C, C = T)

Condition 2 : the compressive force which is produced by composite element is greater than the compressive force produced by cast in place concrete element (C > Cc, Vnh = C, C<T)

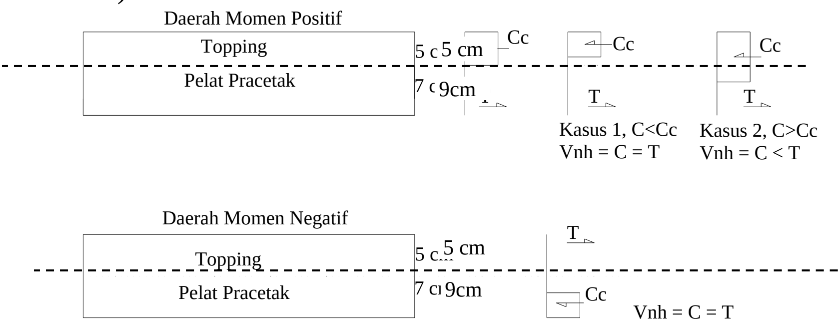


Figure 4.24. The diagram of horizontal shear force (source: Private Documentation)

<u>For roof slab (type AA1, etc.) (500 cm x 250 cm)</u>
$Cc = 0{,}85 \times f'_c \times A_{topping}$; $f'_c = 30$ MPa (> 30 days)
$A_{topping}$ is assumed as b x d in which b = 1000 mm and d is the thickness of *overtopping* concrete
Cc = 0,85x30x1000x50 mm = 1275000 N = 1275 kN
use shear connector D 13 mm
$As_{stud} = 132{,}78$ mm$^2 \rightarrow A_s$
Use condition → (Vnh = C, C = T)
$T = A_s \times f_y$ stud = 132,78 x 390 N = 51786 N = 51,786 kN
T = Vnh = C = 51,786 kN
0,55 Ac = 0,55 x b_v x d (SNI, 2013 paragraph 17.5.3.1)
Ac is assumed as b_v x d in which b_v = 1000 mm and d is the thickness of composite element (with *overtopping* concrete)
0,55 Ac = 0,55x1000x14 N = 77000 N = 77 kN
Vnh < 0,55 Ac = 51,786 kN < 77 kN (OK)
(see SNI, 2013 paragraph 17.5.3.1)
$\phi V_s = \phi(n)(A_{se} f_{ut})$ (PCI, 2004)
0,65 x 77 kN = 0,65 x n x 132,78 mm^2 x 390 N/mm^2
50050 N = 33659,73 N x n
n = 1,48 ≈ 3
s = 1000 mm/3 = 333,33 mm ≈ 300 mm
Use shear connector D13-300.

4.2.3.8. Reinforcement Bar Due to Lifting Process of Slab

When lifting process occurs, the need of reinforcement bar due to lifting process, is necessary. Because when lifting process occurs, it will produce moment force. So, the moment force must be calculated and then the reinforcement bar designed so it can bear the moment force due to lifting process. In this chapter will use four lifting points of slab (Figure 4.25).

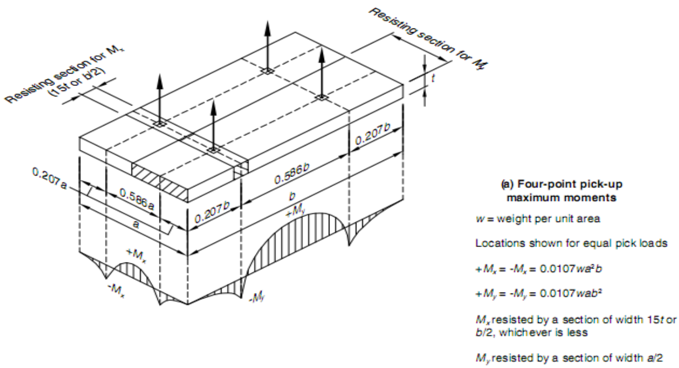


Figure 4.25. Four lifting points of roof slab
(source: PCI, 2004)

The general data for calculating lifting bar will be shown below

f'_c (14 days) = 26,4 MPa
f_y = 240 MPa = 2400 kg/cm^2
slab's type = roof, (type AA1, etc.)
dimension = 500 x 250 mm^2
n (number of points) = 4
a (slab's width) = 2230mm
b (slab's length) = 4680mm
k (shock coefficient) = 1,2
t (slab's thickness before monolith condition) = 9 cm
w (=t x 2400 kg/m^2) = 216 kg/m^2
k x DL (= slab's weight + shear connector (1% slab's weight) = 2732,16 kg
Qu (=1.4 DL x k) = 3825,032 kg
P (=Qu/n) = 956,26 kg

Designing The Lifting Bar

The designing process of lifting bar will use equation below

$$\phi_{\text{tulangamamgkat}} \geq \sqrt{\frac{n \times P}{\pi \times \sigma}}$$

in which n is the number of liting points and σ is $f_y/1,5$ = 1600 kg/cm^2 (SF = 1,5, f_y = 240 MPa).
So, the value of ϕ (diameter of lifting bar) is

$$\phi = \sqrt{\frac{4\ x\ 956{,}26\ kg}{\frac{22}{7}\ x\ 1600\ kg/cm^2}} = 0{,}87\ cm$$

use lifting bar ϕ = 10 mm instead (As = 78,5714 mm^2).

Moment Calculation Due to Lifting Process

The moment calculatin will be divided into two calculations based on slab's direction (direction a and direction b). Direction a is slab's transversal direction and direction b is slab's longitudinal direction (Figure 4.26 and Figure 4.27).

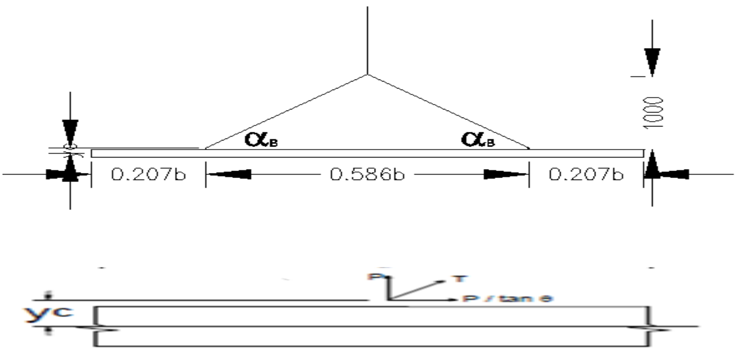


Figure 4.26. Slab's longitudinal direction (direction b) when lifting process occurs
(source: Private Documentation)

<u>direction b</u>

b	= 468 cm
n (number of point)	= 2
0,207 b	= 0,97 m
0,586 b	= 2,74 m
rope's height	= 1000 mm
$\alpha_b{}^o$	= $\tan^{-1}$ ((rope's height/(0,586 b/2))
	= $36{,}12^o$
T_b	= P x sin $\alpha_b{}^o$ = 563,78 kg
y_c	= 0,5 t + 1" = 0,0704 m
M_b	= 0,0107 x w x a^2 x b = 53,789 kgm
$M_{b'}$	= P x y_c/tan $\alpha_b{}^o$ = 92,23 kgm

(moment force due to lifting process)

M_b (total)	= 146,018 kgm

M_b is retained by area of 15t or b/4 (choose the least value between)

15t	= 135 cm
b/4	= 234 cm

use 15t = 135 cm

Z_b	= 1/6 x 15t x t^2 = 1822,5 cm^3

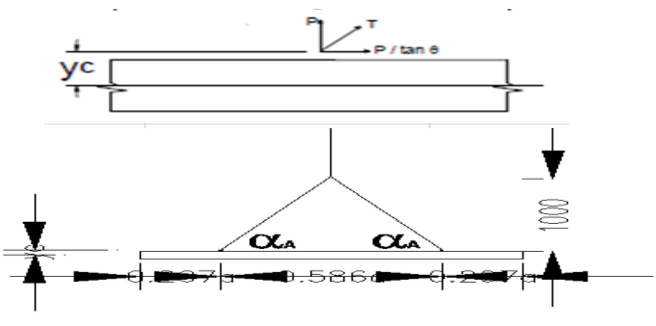


Figure 4.27. Slab's transversal direction (direction a) when lifting process occurs
(source: Private Documentation)

<u>direction a</u>

a	= 223 cm
n (number of point)	= 2
0,207 a	= 0,47 m
0,586 a	= 1,29 m
rope's height	= 1000mm
$\alpha_a{}^o$	= $\tan^{-1}$ ((rope's height/(0,586 a/2))
	= $57{,}17^o$
T_a	= P x sin $\alpha_a{}^o$ = 803,6 kg
y_c	= 0,5 t + 1" = 0,0704 m
M_a	= 0,0107 x w x a x b^2 = 112,88 kgm
$M_{a'}$	= P x y_c/tan $\alpha_a{}^o$ = 43,42 kgm

(moment force due to lifting process)

M_a (total)	= 156,306 kgm

M_a is retained by area of a/2

a/2	= 111,5 cm
Z_a	= 1/6 x a/2 x t^2 = 1505,25 cm^3

<u>Checking The Crack Factor</u>

The lifting bar will use ϕ = 10 mm plain bar.
Checking the f_{cr} value (assumed the concrete's age is 14 days)

$f_{cr} = 0{,}62\ \lambda\sqrt{f'c}$ /SF (SNI, 2013 paragraph 9.5.2.3)
(λ = 1, SNI, 2013 paragraph 8.6.1)
(safety factor = 2)

f_{cr}	= 1,593 MPa
f_b	= M_b (total)/Z_b = 0,801 MPa
f_a	= M_a (total)/Z_a = 1,038 MPa
$f_b < f_{cr}$	= 0,801 MPa < 1,593 MPa (OK)
$f_a < f_{cr}$	= 1,038 MPa < 1,593 MPa (OK)

<u>Checking The Strand Cable</u>

The strand cable for lifting process will use seven-wire strand 5/16 inch diameter with f_{pu} = 250 ksi (see PCI, 2004 Design Aid Table 11.2.3).

f_{pu}	= 250 ksi
A	= 0,058 in^2
f_{pu} x A	= 14,5 kips = 64499,2132 N
SF	= 2
Af_{pu}/SF	= 3224,96kg
T_b	= 563,78kg
T_a	= 803,60kg
$T_b < Af_{pu}/SF$	= 563,78kg < 3224,96 kg (OK)
$T_a < Af_{pu}/SF$	= 803,60kg < 3224,96 kg (OK)

4.2.3.9. Checking The Deflection of Slab

The deflection of slab is calculated using equation below

δ (deflection) = (5/384) x (q_u x L^4/EI)

$\delta_{max} = L_n/480$ (in cm) (SNI 2847:2013paragraph 9.5.3.1)

in which q_u is taken as 861,2 kg/m^2x 1 m (see previous paragraph), I is the moment of inertia of slab (I = $1/12xbxh^3$), and E equals $4700x(f'c)^{0,5}$, f'c in 14 days

Deflection (Longitudinal, L_n = 460 cm, b =L_n, h = 15 cm)

δ_b = (5/384) x (q_u x L^4/EI) = 0,197 cm

$\delta_{max} = L_n/480$ = 0,958 cm

$\delta_b < \delta_{max}$ (OK)

Deflection (Transversal, S_n = 215 cm, b =S_n, h = 15 cm)

δ_a = (5/384) x (q_u x L^4/EI) = 0,02 cm

$\delta_{max} = L_n/480$ = 0,447 cm

$\delta_a < \delta_{max}$ (OK)

4.2.3.10. Checking The Shrinkage of Slab

The shrinkage of slab is calculated using equation below (see SNI, 2013 paragraph 10.6.4)

$s = 380x280/f_s - 2{,}5\ Cc \leq 380x280/f_s$

in which $f_s = 2/3xf_y = 266{,}67$ MPa,

Cc = clear cover + 1/2xD = 36,5 mm

s = 398,99 - 91,25 = 307,74 N/mm $\leq$ 398,99 N/mm (OK)

4.2.4. Reinforcement of Precast Secondary Beam

4.2.4.1. General Data

The general data for reinforcement of secondary beam will be shown below

b (width) = 30 cm

h (height, before monolith) = 45 cm

h (height,after monolith) =45 cm + hf (floor) = 60 cm

L(axis to axis) = 5000mm

L(actual) = 4600 mm

bar's diameter (D/deform) = 22 mm

bar's diameter/stirrup(ϕ/plain) = 10 mm

f'_c(28 days) = 30 MPa

f'_c(14 days) = 0,88*30 MPa = 26,4 MPa

f_y =390 MPa

4.2.4.2. Load Calculation

The load calculation of precast secondary beam will be assumed as trapezoid shape (Figure 4.28). The loads consist of deal load and live load. The load $q_{eqivalent}$ (q_{eq}) will be obtainedd by this calculation below

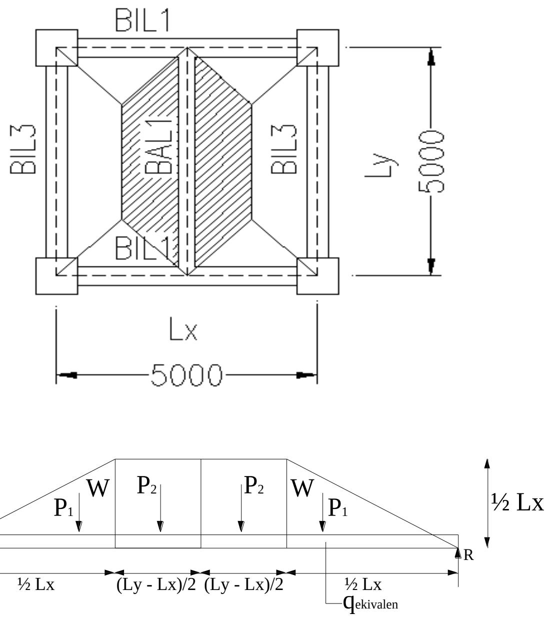


Figure 4.28. Trapezoid load of secondary beam
(source: Private Documentation)

trapezoid load
W = 1/2 x q x L_x → distributed load
P1= 1/2 x L_x x 1/2 x L_x x q/2 → area of P1(triangle)
P2= (L_y – L_x)/2 x 1/2 x L_x x q → area of P2 (rectangular)
P1 = 1/8 x q x L_x^2
P2 = 1/4 x q x L_x (L_y – L_x)
$\sum$V = 0 → 2R = 2 x (P1 + P2) → R = P1 + P2

Mmax (at middle span)
Mmax = (R x 1/2 x L_y) – (P1 x (1/2 x L_y – 1/3 x L_x)) – (P2 x (1/4 x (L_y – L_x))
Mmax = (R x 1/2 x L_y) – (P1 x1/2 x L_y)+(P1 x 1/3 x L_x)– (P2 x 1/4 x L_y)+ (P2 x 1/4 x L_x)

Mmax = (P1 x 1/2 x L_y) + (P2 x 1/2 x L_y) - (P1 x 1/2 x L_y) + (P1 x 1/3 x L_x) - (P2 x 1/4 x L_y) + (P2 x 1/4 x L_x)
Mmax = (P2 x 1/4 x L_y) + (P1 x 1/3 x L_x) + (P2 x 1/4 x L_x)
Mmax = (q x 1/16 x L_x x L_y^2) - (q x 1/16 x L_x^2 x L_y) + (q x 1/24 x L_x^3) + (q x 1/16 x L_x^2 x L_y) - (q x 1/16 x L_x^3)
Mmax = (q x 1/16 x L_x x L_y^2) - (q x 1/48 x L_x^3)
Mmax = q x 1/8 x L_y^2 x (L_x x 1/2 - L_x^3/L_y^2 x 1/6)

Meq = 1/8 x q_{eq} x L_y^2
Mmax = Meq
q x 1/8 x L_y^2 x (L_x x 1/2 - L_x^3/L_y^2 x 1/6) = 1/8 x q_{eq} xL_y^2
8 x q x 1/8 x L_y^2 x (L_x x 1/2 - L_x^3/L_y^2 x 1/6) = q_{eq} xL_y^2
q x L_y^2 x (L_x x 1/2 - L_x^3/L_y^2 x 1/6) = q_{eq} xL_y^2
q_{eq} = q x (L_x x 1/2 - L_x^3/L_y^2 x 1/6)
q_{eq} = q x L_x x 1/2 x ((1 - L_x^2/L_y^2) x 1/3))

<u>Before Monolith Condition</u>

In this condition itself will be divided into two conditions below

1. condition a

when the *overtopping* concrete is not installed yet and the load consists of working load (construction process' load), precast slab element's load, and precast secondary beam element's load

2. condition b

when the *overtopping* concrete is already installed (not yet in monolith condition) and the load consists of *overtopping* concrete's load, precast slab element's load, precast secondary beam element's load, and construction process' load

Then the most critical condition between two conditions above will be used for load calculation.

1. dead load

secondary beam BAL1 : 0,3x0,45x2400 kg/m= 324 kg/m

q of slab LA1
(condition a) : 608 kg/m^2 = 608 kg/m^2
q of slab LA1
(condition b) : 809,6 kg/m^2 = 809,6 kg/m^2
by using L_y = 500 cm – (40 cm/2 + 40 cm/2) = 460 cm and L_x = 250 cm – (40 cm/2 + 30 cm/2) = 215 cm, it is obtained the value of q_{eq} due to slab's load

(two trapezoid loads)
q_{eq} = 2 x q x 1/2 x L_x x ((1 -1/3 x $(L_x/L_y)^2$)
q (condition a) = 608 kg/m^2
q_{eq} = 1212,01 kg/m → due to slab's load
q (condition b) = 809,6 kg/m^2
q_{eq} = 1613,89 kg/m → due to slab's load
DL (condition a) = 1212,01 kg/m+ 324 kg/m
= 1536,01 kg/m
DL (k, shock coefficient = 1,2) = 1,2x1536,01 kg/m
= 1843,21 kg/m
DL (condition b) = 1613,89 kg/m +324 kg/m
= 1937,89 kg/m
DL (k, shock coefficient = 1,2) = 1,2x1937,89 kg/m
= 2325,47 kg/m

DL (total, condition a) : = 1843,21 kg/m
DL (total, condition b) : = 2325,47 kg/m

2. live load

construction load: = 200 kg/m^2
q_{eq} = 398,688 kg/m → due to live load

LL (total, condition a and b): = 398,688 kg/m

After Monolith Condition

In this condition itself will be divided only intoone condition below

1. condition c

when the *overtopping* concrete is already installed (already in monolith condition) and the load consists of*overtopping* concrete's load, precast slab element's load, precast secondary beam element's load, and live loads

1. dead load

secondary beam BAL1 : 0,3x0,60x2400 kg/m= 432 kg/m
q of slab LA1
(condition c) : 922,8 kg/m^2 = 922,8 kg/m^2

by using Ly = 500 cm – (40 cm/2 + 40 cm/2) = 460 cm and Lx = 250 cm – (40 cm/2 + 30 cm/2) = 215 cm, it is obtained the value of q_{eq} due to slab's load

(two trapezoids load)
$q_{eq} = 2 \times q \times 1/2 \times L_x \times ((1 - 1/3 \times (L_x/L_y)^2)$
q (condition c) = 922,8 kg/m^2
q_{eq} = 1839,55 kg/m → due to slab's load
DL (condition c) = 1839,55 kg/m + 432 kg/m
= 2271,55 kg/m
DL (k, shock coefficient = 1,2) = 1,2x2271.55 kg/m
= 2725,86 kg/m

DL (total, condition c) : = 2725,86 kg/m

2. live load

live load: 1,92 kN/m^2 = 192 kg/m^2

q_{eq} = 382,741 kg/m → due to live load

LL (total, condition c) : = 382,741 kg/m

4.2.4.3. Load Combination of Secondary Beam

The combination of load for secondary beam is based on SNI 2847:2013 paragraph 9.2.1. The combination itself is divided into three conditions (seeprevious pages). The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of secondary beam
condition a
(precast's age = 14 days)

Qu = 1,2×1843,21 + 1,6x398,688 = 2849,76 kg/m
condition b
(precast's age = 14 days, *overtopping*'s age = 0 day)
Qu = 1,2x2325,47 + 1,6x398,688 = 3729,06 kg/m
condition c
(precast's age > 30 days, *overtopping*'s age > 30 days)
Qu = 1,2×2725,86 + 1,6x382,741 = 3883,41 kg/m

4.2.4.4. Moment Calculation of Secondary Beam

The moment calculation of secondary beam before monolith condition (condition a and condition b) will use momentequation Mu = 1/8 x q x L^2. The moment calculation ofsecondary beam after monolith condition (condition c) will use the equations in SNI 2847:2013. For condition a and condition b, the secondary beam's support will be assumed as simple support (roller-pins). For condition c, the secondary beam's support will be assumed as fixed support. The moment calculation itself will be divided into three condition (condition a, condition b, and condition c). The equation for moment calculation for condition a and condition b will be shown below

Ml = 1/8 x qx L^2, in which L is the length of axis to axis of secondary beam

The equation for moment calculation for condition c is based on SNI (2013). According to SNI (2013), the equation of moment values for secondary beam (elastic fixed) are shown below
Ml (+) = 1/16 x qx $L^2 \rightarrow$ at field area
Mt (-) = 1/10 x qx L^2 $\rightarrow$ at support area
moment calculation of secondary beam
condition a (q = 2849,76 kg/m, L = 5 m)
Ml (+) = 8905,495 kgm
condition b (q = 3729,06 kg/m, L = 5 m)

Ml (+) = 11653,299 kgm
condition c (q = 3883,41 kg/m, L = 5 m)
Ml (+) = 6067,832 kgm
Mt(-) = 9708,532 kgm

The Mu values are taken as
condition a
Mu = 8905,495 kgm (+)
condition b
Mu = 11653,299 kgm (+)
condition c
Mul = 6067,832 kgm (+)
Mut = 9708,532 kgm (-)
and then the Mu values are divided into two conditions which it will be taken as the greatest result between condition a and condition b for before monolith condition, then Mu value in condition c is considered as Mu value for after monolith condition.

before monolith condition
Mul = 11653,299 kgm (+)
after monolith condition
Mul = 6067,832 kgm (+)
Mut = 9708,532 kgm (-)

4.2.4.5. Shear Force Calculation of Secondary Beam

The shear force calculation of secondary beam will use shear force equation Vu = 1/2 x q x L.
shear force calculation of secondary beam
condition a (q = 2849,76 kg/m, L = 5 m)
Vu = 7124,396 kg
condition b (q = 3729,06 kg/m, L = 5 m)
Vu = 9322,639 kg
condition c (q = 3883,41 kg/m, L = 5 m)
Vu = 9708,532 kg

The Vu values are taken as
condition a
Vu =7124,396 kg
condition b
Vu = 9322,639 kg
condition c
Vu =9708,532 kg
and then the Vu values are divided into two conditions which it will be taken as the greatest result between condition a and condition b for before monolith condition, then Vu value in condition c is considered as Vu value for after monolith condition.
before monolith condition →Vu = 9322,639 kg
after monolith condition →Vu = 9708,532 kg

4.2.4.6. Calculation of Reinforcement Bar

The calculation of reinforcement bar will be divided into two conditions (before monolith condition and after monolith condition). The general data for calculation of reinforcement bar of secondary beam is shown below
before monolith condition
secondary beam's dimension (precast) = 300mm × 450mm
after monolith condition
secondary beam's dimension (overall) = 300mm × 600mm
clear cover = 40 mm
bar's diameter (D) = 22 mm
bar's diameter/stirrup(ϕ/plain) = 10 mm
f'_c(28 days) = 30 Mpa
f'_c(14 days) = 0,88*30 Mpa = 26,4 MPa
f_y = 390 MPa
d'= clear cover + stirrup + D/2 = 61 mm

<u>before monolith condition</u>
d = 450 - 40 - 10 - (22/2) = 389 mm
For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 14 days = 26,4 MPa, see Table 4.7)
$\beta_1 = 0,85$
As = $\pi/4xd^2$ = 380,286 mm^2
a = $Asxf_y/(0,85xf'_cxb)$ = 22,03 mm
c = a/β_1 = 25,92 mm
ε_t = (d/c - 1)x0,003 = 0,0420 → ϕ = 0,9 (SNI, 2013 Figure S9.3.2)
m = $f_y/(0,85xf'_c)$ = 17,38
ρ_{min} = ¼ x $(f'_c)^{0,5}/f_y$ = 0,0032 (SNI, 2002 paragraph 12.5.1)

<u>after monolith condition</u>
d = 600 - 40 - 10 - (22/2) = 539 mm
For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)
$\beta_1 = 0,85 - 0,05\,\frac{(f'c-28)}{7} \geq 0,65$
$\beta_1 = 0,85 - 0,05\,\frac{(30-28)}{7} = 0,8357143 \geq 0,65$
As = $\pi/4xd^2$ = 380,286 mm^2
a = $Asxf_y/(0,85xf'_cxb)$ = 19,38 mm
c = a/β_1 = 23,19 mm
ε_t = (d/c - 1)x0,003 = 0,0667 → ϕ = 0,9 (SNI, 2013 Figure S9.3.2)
m = $f_y/(0,85xf'_c)$ = 15,29
ρ_{min} = ¼ x $(f'_c)^{0,5}/f_y$ = 0,0035 (SNI, 2002 paragraph 12.5.1)

<u>Reinforcement Bar Before Monolith Condition</u>

The general data for reinforcement bar of secondary beam before monolith condition will be shown below
secondary beam's dimension = 300x 450 mm^2
f'_c = 26,4 MPa (14 days)
clear cover = 40 mm
bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380.28 mm^2

b = 300 mm
d = 389 mm
d’ = 61 mm
ϕ = 0,9
M_{ul} = 11653,299 kgm = 116532995 Nmm

Flexural Bar
due to M_{ul}

$$Rn = \frac{Mn}{b \times d^2} = 2{,}852 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}00765$$

$\rho_{need} > \rho_{min}$ = 0,00765 > 0,0032
use $\rho = \rho_{need}$ = 0,00765
$As_{need} = \rho x b x d$ = 893,05 mm^2
n = As_{need}/As_{bar} = 893,05 mm^2/380,28 mm^2 = 2,34
use n = 3$\rightarrow$ $As_{use} = nxAs_{bar}$ = 1140,857 mm^2
$As_{use} > As_{need}$ = 1140,857 mm^2 > 893,05 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 67 mm > 25 mm (OK), use 3D22 reinforcement bar

Stirrup Bar
Use stirrup bar ϕ 10 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 9322,64 kg = 93226,4 N
Vs min = Vu/ϕ = 124302 N
A_s = 22/7 x 10 x 10/4 mm^2 = 78,57 mm^2
A_v = 2 x A_s = 157,14 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 118{,}02 \text{ mm}$$

$s_{maks} \leq d/2$ = 194,5 mm (SNI, 2013 paragraph 21.3.4.3)
use s= 100 mm

Use the space of stirrup bar, s = 100 mm, (ϕ10-100)

Development Bar's Length

ℓ_{dh} = 8xd_b = 8x22 mm = 176 mm

(SNI, 2013 paragraph 12.5.1)

ℓ_{dh} = 150 mm

(SNI, 2013 paragraph 12.5.1)

ℓ_{dh} = 0,24xfy/d_bx$(f'_c)^{0,5}$ = 21,86 mm

(SNI, 2013 paragraph 12.5.2)

So, the development bar's length will use ℓ_{dh} = 180 mm with hook 4d_b = 88 mm ≈ 100 mm.

Reinforcement Bar After Monolith Condition

The general data for reinforcement bar of seconday beam after monolith condition will be shown below

secondary beam's dimension = 300x 600 mm^2

f'_c = 30 MPa (28 days)

clear cover = 40 mm

bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2

b = 300 mm

d = 539 mm

d' = 61 mm

ϕ = 0,9

M_{ul} = 6067,832 kgm = 60678328 Nmm

M_{ut} = 9708,532 kgm = 97085325 Nmm

Flexural Bar

due to M_{ul}

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0{,}773 N/mm^2$$

$$\rho_{need} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2 \times \mathrm{m} \times \mathrm{Rn}}{\mathrm{fy}}}\right) = 0{,}002$$

$\rho_{need} < \rho_{min}$ = 0,002 < 0,0035

use $\rho = \rho_{min}$ = 0,0035
$As_{need} = \rho x b x d$ = 565,95 mm^2
n = As_{need}/As_{bar} = 565,95 mm^2/380,28 mm^2 = 1,48
use n = 3 (3D22)→ $As_{use} = nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 565,95 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 67 mm > 25 mm (OK), use 3D22 reinforcement bar

<u>Flexural Bar</u>
due to M_{ut}

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 1{,}237 N/mm^2$$

$$\rho_{need} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2 \times \mathrm{m} \times \mathrm{Rn}}{\mathrm{fy}}}\right) = 0{,}0032$$

$\rho_{need} < \rho_{min}$ = 0,0032 < 0,0035
use $\rho = \rho_{min}$ = 0,0035
$As_{need} = \rho x b x d$ = 565,95 mm^2
n = As_{need}/As_{bar} = 565,95 mm^2/380,28 mm^2 = 1,48
use n = 3 → $As_{use} = nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 565,95 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 67 mm > 25 mm (OK), use 3D22 reinforcement bar

<u>Stirrup Bar</u>
Use stirrup bar ϕ 10 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 9708,53 kg = 97085,3 N
Vs min = Vu/ϕ = 124302 N
A_s = 22/7 x 10 x 10/4 mm^2 = 78,57 mm^2
A_v = 2 x A_s = 157,14 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 163{,}53 \text{ mm}$$

$s_{maks} \leq d/2$ = 269,5 mm (SNI, 2013 paragraph 21.3.4.3)
Use the space of stirrup bar, s = 150 mm, (ϕ10-150)

Development Bar's Length
$\ell_{dh} = 8xd_b$ = 8x22 mm = 176 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_b x(f'_c)^{0,5}$ = 23,3 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 180 mm with hook $4d_b$ = 88 mm ≈ 100 mm.

Reinforcement Bar Due to Lifting Process' Moment

The general data for reinforcement bar of secondary beam due to lifting process's moment will be shown below
secondary beam's dimension = 300x 450 mm^2
f'_c = 26,4 MPa (14 days)
clear cover = 40 mm
bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2
b = 300 mm
d = 389 mm
d' = 61 mm
ϕ = 0,9
M_{ul} = 173,97 kgm = 1739753,718 Nmm
M_{ut} = 533,81 kgm = 5338170,511 Nmm
(see paragraph 4.2.4.7)

Flexural Bar
due to M_{ul}

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0{,}042 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1-\sqrt{1-\frac{2\times m\times Rn}{fy}}\right) = 0{,}000109$$

$\rho_{need} < \rho_{min}$ = 0,000109 < 0,0032
use $\rho = \rho_{min}$ = 0,0032
$As_{need} = \rho xbxd$ = 373,44 mm^2
n = As_{need}/As_{bar} = 373,44 mm^2/380,28 mm^2 = 0,98
use n = 3 → $As_{use} = nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 373,44 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 67 mm > 25 mm (OK),use 3D22 reinforcement bar

Flexural Bar
due to M_{ut}

$$Rn = \frac{Mn}{b\times d^2} = 0{,}13 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1-\sqrt{1-\frac{2\times m\times Rn}{fy}}\right) = 0{,}00033$$

$\rho_{need} < \rho_{min}$ = 0,00033 < 0,0032
use $\rho = \rho_{min}$ = 0,0032
$As_{need} = \rho xbxd$ = 373,44 mm^2
n = As_{need}/As_{bar} = 373,44 mm^2/380,28 mm^2 = 0,98
use n = 3 → $As_{use} = nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 373,44 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 67 mm > 25 mm (OK),use 3D22 reinforcement bar

Stirrup Bar
Use stirrup bar ϕ 10 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 1251,936 kg = 12519,36 N
Vs min = Vu/ϕ = 16692,48 N

A_s = 22/7 x 10 x 10/4 mm^2 = 78,57 mm^2
A_v = 2 x A_s = 157,14 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 878{,}87 \text{ mm}$$

$s_{maks} \leq d/2$ = 269,5 mm (SNI, 2013 paragraph 21.3.4.3)
Use the space of stirrup bar, s = 200 mm, (ϕ10-200)

<u>Development Bar's Length</u>
$\ell_{dh} = 8xd_b$ = 8x22 mm = 176 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_bx(f'_c)^{0,5}$ = 21,86 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 180 mm with hook $4d_b$ = 88 mm ≈ 100 mm.

For the recapitulation of reinforcement bar will be shown in Table 4.11.

Table 4.11. Recapitulation of Reinforcement Bars of Precast Secondary Beam

Before Monolith Condition									
Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar	
		As use (mm^2)	N	Code	As use (mm^2)	N	Code	S (mm)	Code
BAL1	4600	1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100
		1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100
BAL2	4400	1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100
		1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100
BAA1	4600	1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100
		1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100
BAA2	4400	1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100
		1146,94	3	3D22	1146,94	3	3D22	100	Φ10-100

After Monolith Condition									
Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar	
		As use (mm^2)	N	Code	As use (mm^2)	N	Code	s (mm)	Code
BAL1	4600	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-150
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-150
BAL2	4400	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-150
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-150
BAA1	4600	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
BAA2	4400	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200

Due to Lifting Process' Moment									
Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar	
		As use (mm^2)	N	Code	As use (mm^2)	N	Code	s (mm)	Code
BAL1	4600	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
BAL2	4400	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
BAA1	4600	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
BAA2	4400	1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200
		1140,86	3	3D22	1140,86	3	3D22	100	Φ10-200

4.2.4.7. Reinforcement Bar Due to Lifting Process of Secondary Beam

When lifting process occurs, the need of reinforcement bar due to lifting process, is necessary. Because when lifting process occurs, it will produce moment force. So, the moment force must be calculated and then the reinforcement bar designed so it can bear the moment force due to lifting process. In this chapter will use two lifting points of secondary beam (Figure 4.29).

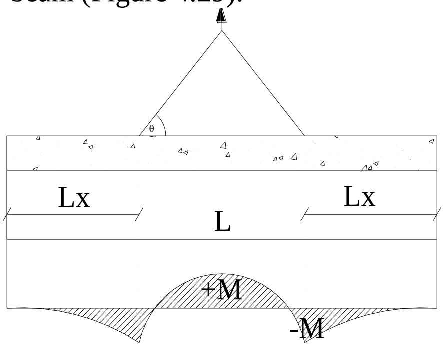


Figure 4.29. Two lifting points of secondary beam and the moment obtained due to lifting process of secondary beam (source: Private Documentation)

The general data for calculating lifting bar will be shown below

f'_c (14 days) = 26,4 MPa
f_y = 240 MPa = 2400 kg/cm^2
secondary beam's type = BAL1 (b = 300 x h = 450)
L (length) = 4600 mm
n (number of points) = 2
k (shock coefficient) = 1,2
W = 300x450x4600x2400 = DL = 1490,4 kg
Wt = 300x450x2400 = 324 kg/m

Qu (=1,4 DL x k) = 2503,87 kg
P (=Qu/n) = Tu = 1251,93 kg

Designing The Lifting Bar

The designing process of lifting bar will use equation below

$$\phi_{\text{tulangamamgkat}} \geq \sqrt{\frac{n \times P}{\pi \times \sigma}}$$

in which n is the number of liting points and σ is $f_y/1,5$ = 1600 kg/cm^2 (SF = 1,5, f_y = 240 MPa).
So, the value of ϕ (diameter of lifting bar) is

$$\phi = \sqrt{\frac{2\ x\ 1251{,}93\ \ kg}{\frac{22}{7}\ x\ 1600\ kg/cm^2}} = 0{,}706\ cm$$

use lifting bar ϕ = 10 mm instead (As = 78,5714 mm^2).

Moment Calculation Due to Lifting Process

The moment calculation due to lifting process of beam will use these equations below

$$+\mathrm{M} = \frac{\mathrm{WtL^2}}{8}\left(1 - 4\mathrm{x} + \frac{4\mathrm{yc}}{\mathrm{Ltan\theta}}\right) \rightarrow \text{ moment at field area}$$

$$-\mathrm{M} = \frac{\mathrm{WtX^2L^2}}{2} \rightarrow \text{ moment at support area}$$

$$\mathrm{X} = \frac{1 + \frac{4\mathrm{yc}}{\mathrm{Ltan\theta}}}{2\left(1 + \sqrt{1 + \frac{\mathrm{yt}}{\mathrm{yb}}\left(1 + \frac{4\mathrm{yc}}{\mathrm{Ltan\theta}}\right)}\right)}$$

$y_t = y_b$ = ½ x h = 22,5 cm
$y_c = y_t$ + 1” = 25,04 cm
rope’s height = 1500mm
θ^o = $\tan^{-1}$ ((rope’s height/(L – 2LX))
= $55{,}99^o$
tan θ^o = 1,483
X = 0,276 m → 0.28 m

LX = 1,273 m → 1.288 m
L – 2LX = 2,052 m → 2.024 m
T (cable force) = P x sin θ^o = 1037,83 kg
+M = 173,97 kgm
−M = 322,49 kgm
M’ (negative moment) = P x y_c/tan θ^o = 211,318kgm
(moment force due to lifting process)
−M (total) = 533,81 kgm
Z = 1/6 x b x h^2 = 10125 cm^3

Checking The Crack Factor

The lifting bar will use ϕ = 10 mm plain bar.
Checking the f_{cr} value (assumed the concrete’s age is 14 days)

$f_{cr} = 0.62\ \lambda\sqrt{f'c}$ /SF (SNI, 2013 paragraph 9.5.2.3)
(λ = 1, SNI, 2013 paragraph 8.6.1)
(safety factor = 2)
f_{cr} = 1,593 MPa
f (due to field moment) = +M(total)/Z = 0,172 MPa
f (due to support moment) = −M(total)/Z = 0,527 MPa
$f < f_{cr}$ = 0,172 MPa <1,593 MPa (OK)
$f < f_{cr}$ = 0,527 MPa <1,593 MPa (OK)

Checking The Strand Cable

The strand cable for lifting process will use seven-wire strand 5/16 inch diameter with f_{pu} = 250 ksi (see PCI, 2004 Design Aid Table 11.2.3).
f_{pu} = 250 ksi
A = 0,058 in^2
f_{pu} x A = 14,5 kips = 64499,2132 N
SF = 2
A x f_{pu}/SF = 3224,96kg
T = 1037,83 kg
T < Af_{pu}/SF = 1037,83 kg < 3224,96 kg (OK)

4.2.4.8. Checking The Deflection of Secondary Beam

The deflection of secondary beam is calculated using equation below

δ (deflection) = (5/384) x (q_u x L^4/EI)

δ_{max} = L_n/480 (in cm) (SNI 2847:2013paragraph 9.5.3.1)

in which q_u is taken as3883,41 kg/m(see previous paragraph), I is the moment of inertia of secondary beam (I = 1/12xbxh^3), and E equals 4700x(f'c)^{0,5}, f'c in 14 days

Deflection (L_n = 460 cm, b =30 cm, h = 45cm)

δ = (5/384) x (q_u x L^4/EI) = 0,4115 cm

δ_{max} = L_n/480 = 0,958 cm

$\delta < \delta_{max}$ (OK)

4.2.5. Reinforcement of Stair Structure

4.2.5.1. General Data

The general data for reinforcement of staircase structure will be shown below (cast in place concrete, assuming the support is roller-pins, Figure 4.30).

f'_c	= 30 MPa
f_y	= 390 MPa
height	= 4000 mm
bordes's length	= 5000 mm
bordes's width	= 1400 mm
stair's width	= 5000 mm
stair's slab thickness (t_p)	= 200 mm
bordes's slab thickness	= 200 mm
step's height (t)	= 170 mm
step's width (i)	= 300 mm
step's total (n_t)	= height/t
	= 4000 mm/170 mm
	= 23,5 = 24 steps
step's total (n_i)	= n_t -1 = 23 steps
steps to bordes	= 12 steps
bordes to steps	= 12 steps

bordes's elevation = $n_t/2$ x t
= 24/2 x 170 mm
= 2040 = 2000 mm
stair's slab length = $n_t/2$ x i
= 24/2 x 300 mm
= 3600 mm
slope (α^0) = $29{,}05^0$
→ $\tan^{-1} \alpha^0$ = bordes's elevation/stair's slab length
→ $\tan^{-1} \alpha^0$ = 2000 mm/3600 mm
→ $\alpha^0 = 29{,}05^0$
step's slab thickness (t_r) = i/2 x sin α^0
= 72,83 mm
slab's thickness overall (t_{total}) = $t_p + t_r$ = 272,83 mm

checking conditions:
steps
600 mm ≤ (2t + i) ≤ 650 mm
600 mm ≤ (2x170 + 300) ≤ 650 mm
600 mm ≤ 640 mm ≤ 650 mm (OK)

slope
$25^0 \leq \alpha^0 \leq 40^0$
$25^0 \leq 29{,}05^o \leq 40^0$ (OK)

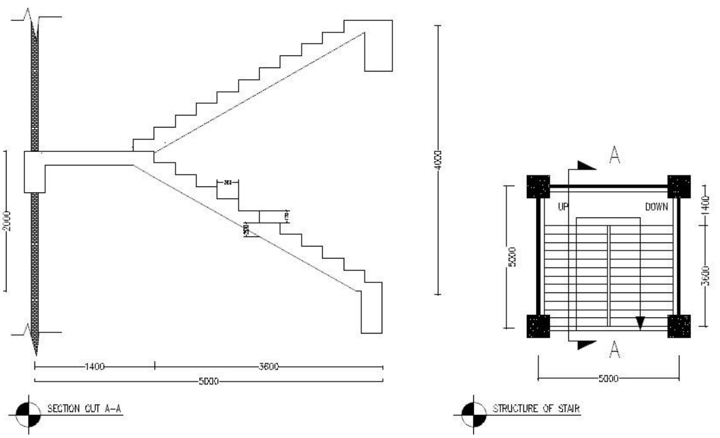


Figure 4.30. The structure of stair
(source: Private Documentation)

4.2.5.2. Load Calculation

<u>Staircase's Load</u>

1. dead load

stair's slab	: $t_{total}/\cos \alpha^0$ x2400 x 1 kg/m	= 749,1 kg/m
mortar (mixture) 2 cm		
vertical step	: 2x21 kg/m	= 42 kg/m
horizontal step	: 2x21 kg/m	= 42 kg/m
ceramic 1 cm		
vertical step	: 1x24 kg/m	= 24 kg/m
horizontal step	: 1x24 kg/m	= 24 kg/m
stair's handle	: 50 kg/m	= 50 kg/m
DL (total) :		= 931,1 kg/m

2. live load

live load	: 1x4,79 kN/m	= 479 kg/m
LL (total) :		= 479 kg/m

load combination

Qu = 1,2DL + 1,6LL.

Qu = 1,2×931,1+ 1,6x479 = 1883,72 kg/m = q_2

<u>Bordes's Load</u>

1. dead load

bordes's slab	: 0,2x1x2400 kg/m	= 480 kg/m
mortar (mixture) 2 cm : 2x21 kg/m		= 42 kg/m
ceramic 1 cm : 1x24 kg/m		= 24 kg/m
DL (total)	:	= 546 kg/m

2. live load

live load	: 1x4,79 kN/m	= 479 kg/m
LL (total)	:	= 479 kg/m

load combination

Qu = 1,2DL + 1,6LL.

Qu = 1,2×546+ 1,6x479 = 1421,6 kg/m = q_1

4.2.5.3. Moment Calculation

The moment calculation of staircase structure will be shown below (assuming the support is roller-pins, Figure 4.31)

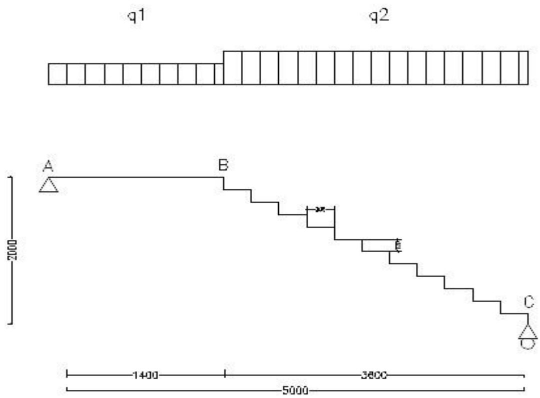


Figure 4.31. Load distribution of stair
(source: Private Documentation)

$\sum M_A = 0$
$(V_C x5) - (q_2 x3,6x(3,6/2 + 1,4)) - (q_1 x1,4x(1,4/2)) = 0$
$V_C = 4619,163$ kg

$\sum M_C = 0$
$(V_A x5) - (q_1 x1,4x(1,4/2 + 3,6)) - (q_2 x3,6x(3,6/2)) = 0$
$V_A = 4153,154$ kg

$\sum H = 0$

$\sum V = 0$
$V_A + V_C = (q_1 x1,4 \text{ m}) + (q_2 x3,6 \text{ m})$
8772,317 kg = 8772,317 kg

<u>Bordes A-B (1,4 m)</u>

a. moment force (M)

X	$= V_A/q_1$
	= 4153,154 kg/(1421,6 kg/m)
	= 2,9 m (not meet condition → X_{maz} = 1,4 m)
M_A	= 0 kgm
$M_{B\ left}$	$= V_A x\ X - ½\ x\ q_1\ x\ X^2$ → (X = 1,4 m)
	= 4421,248 kgm

b. shear force (D)

point A

D_{Aleft}	= 0 kg
D_{Aright}	$= V_A$ = 4153,154 kg

point B

D_{Bleft}	$= V_A - Q_1$
	= 4153,154 kg – 1990,24 kg
	= 2162,914 kg

c. normal force (N)

point A

N_A	= 0 kg

point B

N_B = 0 kg

Stair B-C (3,6 m)

a. moment force (M)

$X = V_C/q_2$
$= 4619{,}163$ kg/(1883,72 kg/m)
$= 2{,}45$ m (from right side)

$M_{max} = V_C x\ X - ½\ x\ q_2\ x\ X^2 \rightarrow (X = 2{,}45\ m)$
$= 5662{,}87$ kgm

$M_C = 0$ kgm

$M_{B\ right} = V_C x\ X - ½\ x\ q_2\ x\ X^2 \rightarrow (X = 3{,}6\ m)$
$= 4421{,}248$ kgm

b. shear force (D)

point C

$D_{C\ right} = 0$ kg

$D_{Cleft} = V_C x \cos \alpha^0 - (q_2 x \cos \alpha^0 x\ X) \rightarrow X = 0\ m$
$= 4037{,}875$ kg

point B

$D_{Bright} = V_C x \cos \alpha^0 - (q_2 x \cos \alpha^0 x\ X) \rightarrow X = 3{,}6\ m$
$= -1890{,}73$ kg

c. normal force (N)

point C

$N_C = -V_C x \sin\alpha^0 + (q_2 x \sin \alpha^0 x\ X) \rightarrow X = 0\ m$
$= -2243{,}26$ kg

point B

$N_B = -V_C x \sin\alpha^0 + (q_2 x \sin \alpha^0 x\ X) \rightarrow X = 3{,}6\ m$
$= 1050{,}404$ kg

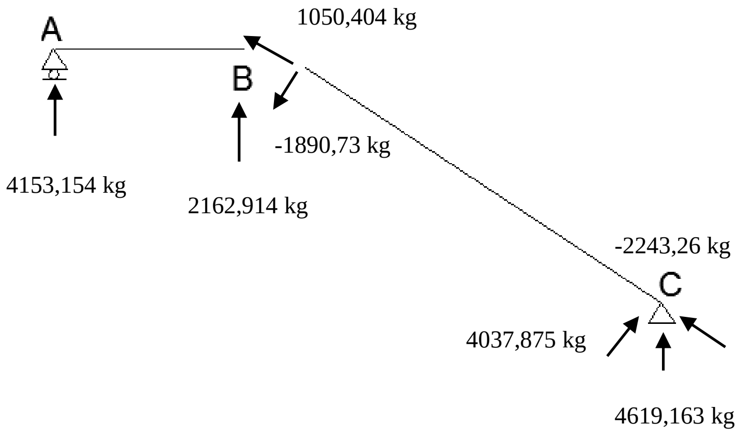


Figure 4.32. Stair's free-body diagram
(source: Private Documentation)

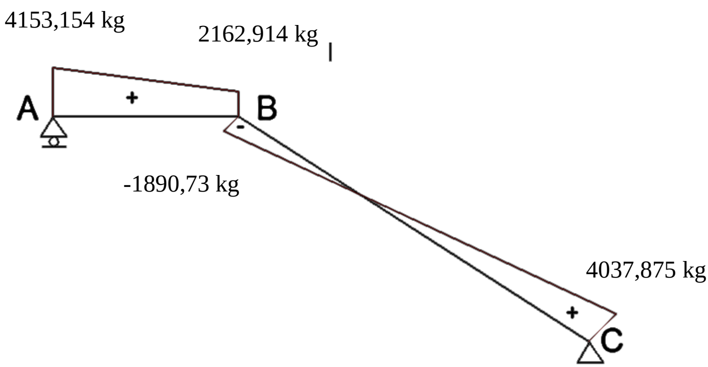


Figure 4.33. Stair's shear force diagram (D)
(source: Private Documentation)

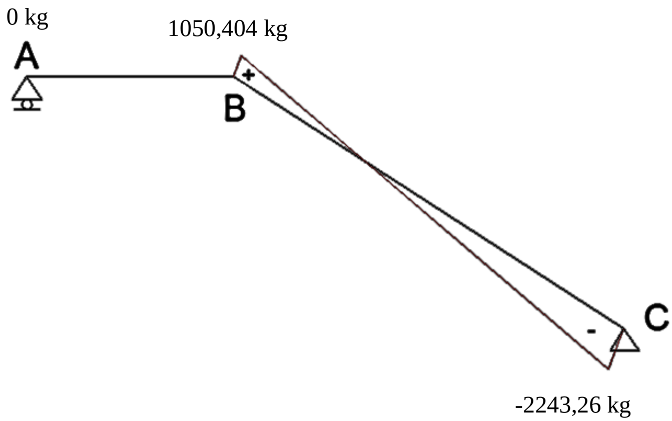


Figure 4.34. Stair's normal force diagram (N)
(source: Private Documentation)

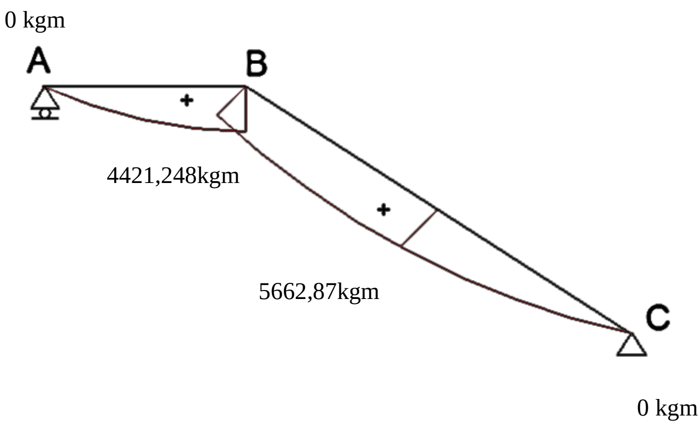


Figure 4.35. Stair's moment force diagram (M)
(source: Private Documentation)

4.2.5.4. Reinforcement of Stair's Slab

General Data

The general data for calculation of reinforcement bar of stair's slab is shown below

stair's slab thickness	= 200 mm
clear cover	= 30 mm
bar's diameter (D)	= 13 mm
f'_c	= 30 Mpa
f_y	= 390 MPa
d = 200 - 30 - 13 - (13/2)	= 150,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0,85 – 0,05\ \frac{(f'c-28)}{7} \geq 0,65$

$\beta_1 = 0,85 – 0,05\ \frac{(30-28)}{7} = 0,8357143 \geq 0,65$

As = $\pi/4xd^2$ = 132,78 mm^2

a = $Asxf_y/(0,85xf'_cxb)$ = 2,03 mm

c = a/β_1 = 2,43 mm

ε_t = (d/c - 1)x0,003 = 0,18 → ϕ = 0,9 (SNI, 2013 Figure S9.3.2)

m = $f_y/(0,85xf'_c)$ = 15,29

ρ_{min} = 0,002 (SNI, 2013 paragraph 7.12.2.1)

Reinforcement Bar of Stair's Slab

The general data for reinforcement bar of stair's slab will be shown below

stair's slab thickness		= 200 mm, f'_c = 30 MPa (28 days)
clear cover		= 30 mm
bar's diameter		= 13 mm, As = $\pi/4xd^2$ = 132,78 mm^2
b		= 1000 mm
d		= 150,5 mm
ϕ		= 0,9
M_u	= 5662,867 kgm	= 56628672,48 Nmm

<u>Main Bar/Longitudinal Bar</u>

$M_n = M_u/\phi = 62920747$ Nmm

$$\text{Rn} = \frac{\text{Mn}}{\text{b} \times d^2} = 2{,}77\,N/mm^2$$

$$\rho_{\text{need}} = \frac{1}{\text{m}}\left(1 - \sqrt{1 - \frac{2 \times \text{m} \times \text{Rn}}{\text{fy}}}\right) = 0{,}0075$$

$\rho_{need} > \rho_{min} = 0{,}0075 > 0{,}002$

use $\rho = \rho_{need} = 0{,}0075$

$As_{need} = \rho x b x d = 1137{,}7\ mm^2$

$n = As_{need}/As_{bar} = 1137{,}7\ mm^2/132{,}7857\ mm^2 = 8{,}56$

use n = 10 → $As_{use} = nxAs_{bar} = 1327{,}857\ mm^2$

$As_{use} > As_{need} = 1327{,}857\ mm^2 > 1137{,}7\ mm^2$ (OK)

s = b/n = 1000 mm/10 = 100 mm

So, the longitudinal bar will use D13-100.

<u>Shrinkage Bar/Transversal Bar</u>

$As_{need} = 0{,}002xbxd = 301\ mm^2$

$n = As_{need}/As_{bar} = 301\ mm^2/132{,}7857\ mm^2 = 2{,}26$

use n = 5 → $As_{use} = nxAs_{bar} = 663{,}93\ mm^2$

$As_{use} > As_{need} = 663{,}93\ mm^2 > 301\ mm^2$ (OK)

s = b/n = 1000 mm/5 = 200 mm

So, the transversal bar will use D13-200.

4.2.5.5. Reinforcement of Bordes's Slab

<u>General Data</u>

The general data for calculation of reinforcement bar of bordes's slab is shown below

bordes's slab thickness	= 200 mm
clear cover	= 30 mm
bar's diameter (D)	= 13 mm
f'_c	= 30 Mpa
f_y	= 390 MPa
d = 200 - 30 - 13 - (13/2)	= 150,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$

$As = \pi/4xd^2 = 132{,}78\ mm^2$

$a = Asxf_y/(0{,}85xf'_cxb) = 2{,}03\ mm$

$c = a/\beta_1 = 2{,}43\ mm$

$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}18 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)

$m = f_y/(0{,}85xf'_c) = 15{,}29$

$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1)

Reinforcement Bar of Bordes's Slab

The general data for reinforcement bar of bordes's slab will be shown below

bordes's slab thickness		= 200 mm, f'_c = 30 MPa (28 days)
clear cover		= 30 mm
bar's diameter		= 13 mm, $As = \pi/4xd^2 = 132{,}78\ mm^2$
b		= 1000 mm
d		= 150,5 mm
ϕ		= 0,9
M_u	= 4421,248 kgm	= 44212481,16 Nmm

Main Bar/Longitudinal Bar

$M_n = M_u/\phi = 49124979$ Nmm

$$Rn = \frac{Mn}{b \times d^2} = 2{,}168 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0058$$

$\rho_{need} > \rho_{min} = 0{,}0058 > 0{,}002$

use $\rho = \rho_{need} = 0{,}0058$

$As_{need} = \rho xbxd = 875{,}93\ mm^2$

$n = As_{need}/As_{bar} = 875{,}93\ mm^2/132{,}7857\ mm^2 = 6{,}59$
use n = 10 → $As_{use} = nxAs_{bar} = 1327{,}857\ mm^2$
$As_{use} > As_{need} = 1327{,}857\ mm^2 > 875{,}93\ mm^2$ (OK)
s = b/n = 1000 mm/10 = 100 mm
So, the longitudinal bar will use D13-100.

<u>Shrinkage Bar/Transversal Bar</u>
$As_{need} = 0{,}002xbxd = 301\ mm^2$
$n = As_{need}/As_{bar} = 301\ mm^2/132{,}7857\ mm^2 = 2{,}26$
use n = 5 → $As_{use} = nxAs_{bar} = 663{,}93\ mm^2$
$As_{use} > As_{need} = 663{,}93\ mm^2 > 301\ mm^2$ (OK)
s = b/n = 1000 mm/5 = 200 mm
So, the transversal bar will use D13-200.

4.2.5.6. Reinforcement of Bordes's Beam

<u>General Data</u>

The general data for calculation of reinforcement bar of bordes's beam is shown below

bordes's beam dimension	= 300mm × 600mm
clear cover	= 40 mm
bar's diameter (D)	= 22 mm
bar's diameter/stirrup(ϕ/plain)	= 10 mm
f'_c	= 30 Mpa
f_y	= 390 MPa
d' = clear cover + stirrup + D/2	= 61 mm
d = 600 - 40 - 10 - (22/2)	= 539 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$$

$$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$$

$As = \pi/4xd^2 = 380{,}28\ mm^2$
$a = Asxf_y/(0{,}85xf'_cxb) = 19{,}38\ mm$
$c = a/\beta_1 = 23{,}19\ mm$

$\varepsilon_t = (d/c - 1)x0,003 = 0,06 \rightarrow \phi = 0,9$ (SNI, 2013 Figure S9.3.2)
$m = f_y/(0,85xf'_c) = 15,29$
$\rho_{min} = \frac{1}{4} x (f'_c)^{0,5}/f_y = 0,0035$ (SNI, 2002 paragraph 12.5.1)

Load Calculation of Bordes's Beam

The load calculation for bordes's beam will be shown below

1. dead load

bordes's beam	: 0,3x0,6x2400 kg/m	= 432 kg/m
brick wall	: 5x250 kg/m	= 1250 kg/m
1,4 DL	: 1,4(432 + 1250) kg/m	= 2354,8 kg/m
bordes's slab	: 1421,6 kg/m = 1421,6 kg/m	
DL (total)	: = 3776,4 kg/m	

Load Combination of Bordes's Beam

The combination of load for bordes's beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of bordes's beam

$Qu = 1,2 \times 3776,4 + 1,6x0 = 4531,68$ kg/m

Moment Calculation of Bordes's Beam

According to SNI (2013), the equation of moment values for bordes's beamare shown below

Ml (+) $= 1/16 \text{ x qx } L^2$ → at field area
Mt (-) $= 1/10 \text{ x qx } L^2$ → at support area

moment calculation of bordes's beam
(q = 4531,68 kg/m, L = 5 m)
Ml (+) = 7080,75 kgm
Mt(-) = 11329,2 kgm

The Mu values are taken as
Mul = 7080,75 kgm(+)
Mut = 11329,2 kgm (-)

Shear Force Calculation of Bordes's Beam

The shear force calculation of bordes's beam will use shear force equation Vu = 1/2 x q x L.

shear force calculation of bordes's beam
(q = 4531,68 kg/m, L = 5 m)
Vu = 11329,2 kg

Reinforcement Bar of Bordes's Beam

The general data for reinforcement bar of bordes's beam will be shown below

bordes's beam's dimension = 300x 600 mm^2
f'_c = 30 MPa (28 days)

clear cover		= 40 mm
bar's diameter		= 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2
b		= 300 mm
d		= 539 mm
d'		= 61 mm
ϕ		= 0,9
M_{ul}	= 7080,75 kgm	= 70807500 Nmm
M_{ut}	= 11329,2 kgm	= 113292000 Nmm

Flexural Bar

due to M_{ul}

$$Rn = \frac{Mn}{b \times d^2} = 1{,}083 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0022$$

$\rho_{need} < \rho_{min}$ = 0,0022 < 0,0035
use $\rho = \rho_{min}$ = 0,0035
$As_{need} = \rho xbxd$ = 565,95 mm^2
n = As_{need}/As_{bar} = 565,95 mm^2/380,28 mm^2 = 1,48
use n = 3 → $As_{use} = nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 565,95 mm^2 (OK)

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 66,67 mm > 25 mm (OK),use 3D22 reinforcement bar

<u>Flexural Bar</u>
due to M_{ut}

$$Rn = \frac{Mn}{b \times d^2} = 1{,}733 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}00356$$

$\rho_{need} > \rho_{min}$ = 0,00356 > 0,0035
use $\rho = \rho_{min}$ = 0,00356
As_{need} = ρxbxd = 576,20 mm^2
n = As_{need}/As_{bar} = 576,20 mm^2/380,28 mm^2 = 1,51
use n = 3 → As_{use} = nxAs_{bar} = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 576,20 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 66,67 mm > 25 mm (OK),use 3D22 reinforcement bar

<u>Stirrup Bar</u>
Use stirrup bar ϕ 10 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 11329,2 kg = 113292 N
Vs min = Vu/ϕ = 151056 N
A_s = 22/7 x 10 x 10/4 mm^2 = 78,57 mm^2
A_v = 2 x A_s = 157,14 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 218{,}68 \text{ mm}$$

$s_{maks} \leq d/2$ = 269,5 mm (SNI, 2013 paragraph 21.3.4.3)
Use the space of stirrup bar, s = 150 mm, (ϕ10-150)

<u>Development Bar's Length</u>
ℓ_{dh} = 8xd_b = 8x22 mm = 176 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 0,24xfy/d_bx$(f'_c)^{0,5}$ = 23,3 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 180 mm with hook 4d_b = 88 mm ≈ 100 mm.

4.2.6. Reinforcement of Elevator's Beam
4.2.6.1. General Data

The general data for reinforcement of elevator's beam structure will be shown below (cast in place concrete, assuming the support is roller-pins)

Elevator's type:
Asia Schneider (Thailand) Co., Ltd
C300 Passenger Elevator Duplex (see Table 4.12)

capacity	= 680 kg
velocity	= 60 m/min
dimension (see Figure 4.36)	
OP	= 800 mm
CW	= 1400 mm
CD	= 1200 mm
HW	= 3850 mm
HD	= 1800 mm

machine room's reaction load	
R3	= 4900 kg
R4	= 3100 kg
elevator beam's dimension	
separator beam	= 250 mm x 650 mm
support elevator beam	= 250 mm x 650 mm

Table 4.12. Type of Elevator C300 Passenger Elevator Duplex
(source: Asia Schneider Thailand, Co., Ltd)

Speed (m/min)	Capacity		Entrance	Car Size		Dimension			Machine Room	
	Person	Load (kg)	Center Opening (mm)	Inside	Outside	In Hoistway			Reaction (kg)	
				CW*CD	SW*SD	A&A'	RD	S	R3	R4
60	6	450	800	1400x850	1450x1015	200	320	150	3600	2300
	7	550	800	1400x1000	1450x1165	200	320	150	4000	2600
	8	630	800	1400x1100	1450x1265	200	320	150	4200	2800
	9	680	800	1400x1200	1450x1365	200	320	150	4900	3100
	10	750	800	1400x1300	1450x1465	200	320	150	5300	3300
	10	800	800	1400x1350	1450x1515	200	320	150	5600	3500
	12	900	900	1600x1300	1650x1465	225	320	150	6100	3800
	13	1000	900	1600x1400	1650x1565	225	320	150	6700	4100
	Hoistway Size				Machine Room Size				Pit Reaction Load (kg)	
	Simplex		Duplex		Simplex		Duplex			
	HW	HD	HW	HD	MW	MD	MW	MD	R1	R2
	1850	1450	3850	1450	2350	3450	4350	2350	5300	4400
	1850	1600	3850	1600	2350	3600	4350	2350	6200	5100
	1850	1700	3850	1700	2350	3700	4350	2350	6700	5500
	1850	1800	3850	1800	2350	3800	4350	2350	7200	5800
	1850	1900	3850	1900	2350	3900	4350	2350	7000	6300
	1850	1950	3850	1950	2350	3950	4350	2350	8300	6700
	2100	1900	4350	1900	2600	3900	4850	2600	9200	7400
	2100	2000	4350	2000	2600	4000	4850	2600	10100	8100

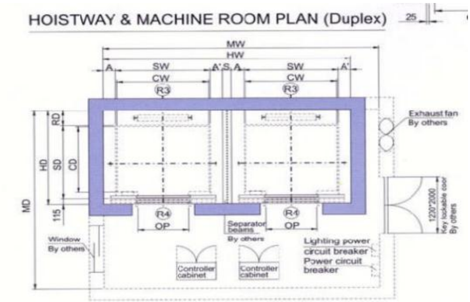


Figure 4.36. Dimension of elevator
(source: Asia Schneider Thailand, Co., Ltd)

4.2.6.2. Load Calculation

The load, P, is determined by the factor, Ψ, in which Ψ is the shock coefficient. So, the P will be calculated by this equation below

$$P = \sum R \times \Psi$$

hence

$$\Psi = (1 + k_1 k_2 v) \geq 1{,}15$$

in which

k_1 is the coefficient which its value depends on stiffenes of *keran induk*, use $k_1 = 0{,}6$

k_2 is the coefficient which its value depends on machine's behavior, use $k_2 = 1{,}3$

v is the velocity of elevator which is 60 m/min or 1 m/s

then,

$\Psi = (1 + 0{,}6 \times 1{,}3 \times 1) = 1{,}78 \geq 1{,}15$

$P = \sum R \times \Psi = (R3 + R4) \times 1{,}78 = 14240$ kg

4.2.6.3. Reinforcement of Separator Beam

General Data

The general data for calculation of reinforcement bar of separator beam is shown below

b = 200 mm
h = 650 mm
clear cover = 30 mm
bar's diameter (D) = 22 mm
bar's diameter/stirrup(ϕ/plain) = 10 mm
f'_c = 30 Mpa
f_y =390 MPa
d' = clear cover + stirrup + D/2 = 51 mm
d = 650 - 30 - 10 - (22/2) = 599 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0,85 - 0,05 \frac{(f'c-28)}{7} \geq 0,65$

$\beta_1 = 0,85 - 0,05 \frac{(30-28)}{7} = 0,8357143 \geq 0,65$

As = $\pi/4xd^2$ = 380,28 mm^2

a = $Asxf_y/(0,85xf'_cxb)$ = 29,08 mm

c = a/β_1 = 34,79 mm

ε_t = (d/c - 1)x0,003 = 0,048 → ϕ = 0,9 (SNI, 2013 Figure S9.3.2)

m = $f_y/(0,85xf'_c)$ = 15,29

$\rho_{min} = ¼ \times (f'_c)^{0,5}/f_y$ = 0,0035 (SNI, 2002 paragraph 12.5.1)

Load Calculation of Separator Beam

The load calculation for separator beam is shown below

1. dead load

beam : 0,20x0,65x2400 kg/m= 312 kg/m
slab's weight : 15 cm x 2400 kg/m^2 = 360 kg/m^2
mortar mixture (1 cm) : 1 x 21 kg/m^2 = 21 kg/m^2
ceramic (1 cm) : 1 x 24 kg/m^2 = 24 kg/m^2
q of elevator's slab : = 405 kg/m^2

by using Ly = 223 cm and Lx = 170 cm, it is obtained the value of qeq due to slab's load

(two trapezoids load)
q_{eq} = 2 x q x 1/2 x Lx x ((1 -1/3 x $(Lx/Ly)^2$)
q= 405 kg/m^2
q_{eq} = 555,126 kg/m→ due to slab's load

DL= 555,126 kg/m + 312 kg/m= 867,126 kg/m

DL (total) : = 867,126 kg/m

2. live load

live load: = 192 kg/m^2

q_{eq} = 263,1709 kg/m

LL (total) : = 263,1709 kg/m

Load Combination of Separator Beam

The combination of load for separator beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination ofseparator beam

Qu = 1,2×867,126 + 1,6x263,1709= 1461,225 kg/m

Moment Calculation of Separator Beam

According to SNI (2013), the equation of moment values forseparator beamare shown below

Mu = 1/8x qx L^2+ 1/4 x P x L

moment calculation of separator beam
(q = 1461,225 kg/m, P = 14240 kg, L = 1,7 m)
Mu = 6580,012 kgm

The Mu values are taken as
Mu = 6580,012 kgm

Shear Force Calculation of Separator Beam

The shear force calculation of separator beam will use shear force equation Vu = 1/2 x q x L + 1/2 x P
shear force calculation of separator beam
(q = 1461,225 kg/m, P = 14240 kg, L = 1,7 m)
Vu = 8749,712 kg

Reinforcement Bar of Separator Beam

The general data for reinforcement bar of separator beam will be shown below

separator beam dimension = 200x 650 mm^2
f'_c = 30 MPa (28 days)
clear cover = 30 mm
bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2
b = 200 mm
d = 599 mm
d' = 51 mm
ϕ = 0,9
M_u = 6580,012 kgm= 65800119 Nmm

Flexural Bar

due to Mu

$$Rn = \frac{Mn}{b \times d^2} = 1{,}018 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0026$$

$\rho_{need} < \rho_{min}$ = 0,0026 < 0,0035
use $\rho = \rho_{min}$ = 0,0035
$As_{need} = \rho xbxd$ = 500,65 mm^2
n = As_{need}/As_{bar} = 500,65 mm^2/380,28 mm^2 = 1,31

use n = 3 → $As_{use} = nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 500,65 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 51,1 mm > 25 mm (OK),use 3D22 reinforcement bar

<u>Stirrup Bar</u>
Use stirrup bar ϕ 10 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 8749,712 kg = 87497,12 N
Vs min = Vu/ϕ = 116662,83 N
A_s = 22/7 x 10 x 10/4 mm^2 = 78,57 mm^2
A_v = 2 x A_s = 157,14 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 193{,}6 \text{ mm}$$

$s_{maks} \leq d/2$ = 299,5 mm (SNI, 2013 paragraph 21.3.4.3)
Use the space of stirrup bar, s = 150 mm, (ϕ10-150)

<u>Development Bar's Length</u>
$\ell_{dh} = 8xd_b$ = 8x22 mm = 176 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_bx(f'_c)^{0,5}$ = 23,3 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 180 mm with hook $4d_b$ = 88 mm ≈ 100 mm.

4.2.6.4. Reinforcement of Elevator Beam
<u>General Data</u>

The general data for calculation of reinforcement bar of elevator beam is shown below

b	= 200 mm
h	= 650 mm
clear cover	= 30 mm

bar's diameter (D) = 22 mm
bar's diameter/stirrup(ϕ/plain) = 10 mm
f'_c = 30 Mpa
f_y = 390 MPa
d' = clear cover + stirrup + D/2 = 51 mm
d = 650 - 30 - 10 - (22/2) = 599 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0,85 - 0,05\,\frac{(f'c - 28)}{7} \geq 0,65$

$\beta_1 = 0,85 - 0,05\,\frac{(30-28)}{7} = 0,8357143 \geq 0,65$

As = π/4xd^2 = 380,28 mm^2
a = Asxf_y/(0,85xf'_cxb) = 29,08 mm
c = a/β_1 = 34,78 mm
ε_t = (d/c - 1)x0,003 = 0,048 → ϕ = 0,9 (SNI, 2013 Figure S9.3.2)
m = f_y/(0,85xf'_c) = 15,29
ρ_{min} = ¼ x $(f'_c)^{0,5}/f_y$ = 0,0035 (SNI, 2002 paragraph 12.5.1)

Load Calculation of Elevator Beam

The load calculation for elevator beam is shown below

1. dead load

beam : 0,20x0,65x2400 kg/m	= 312 kg/m
slab's weight : 15 cm x 2400 kg/m^2	= 360 kg/m^2
mortar mixture (1 cm) : 1 x 21 kg/m^2	= 21 kg/m^2
ceramic (1 cm) : 1 x 24 kg/m^2	= 24 kg/m^2
ducting + plumbing : 25 kg/m^2	= 25 kg/m^2
q of elevator's slab :	= 430 kg/m^2

by using Ly = 420 cm and Lx = 170 cm, it is obtained the value of qeq due to slab's load

(trapezoids load)
q_{eq} = q x 1/2 x Lx x ((1 -1/3 x (Lx/Ly)^2)
q = 430kg/m^2

q_{eq} = 294,69kg/m→ due to slab's load

DL= 294,69kg/m + 312kg/m= 606,69kg/m

DL (total) : = 606,69 kg/m

2. live load

live load: = 400 kg/m^2→ (see PPIUG, 1983 for machine's room load)

q_{eq} = 274,136 kg/m

LL (total) : = 274,136 kg/m

Load Combination of Elevator Beam

The combination of load for elevator beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination ofseparator beam

Qu = 1,2×606,69 + 1,6x274,136 = 1166,654 kg/m

Moment Calculation of Elevator Beam

According to SNI (2013), the equation of moment values forelevator beamare shown below

Mu = 1/8x qx L^2+ 1/4 x P x L

moment calculation of separator beam

(q = 1166,654 kg/m, P = 14240 kg, L = 4,2 m)

Mu = 17524,47 kgm

The Mu values are taken as

Mu= 17524,47 kgm

Shear Force Calculation of Elevator Beam

The shear force calculation of elevator beam will use shear force equation Vu = 1/2 x q x L + 1/2 x P
shear force calculation of separator beam
(q = 1166,654 kg/m, P = 14240 kg, L = 4,2 m)
Vu = 9569,974 kg

Reinforcement Bar of Elevator Beam

The general data for reinforcement bar of elevator beam will be shown below
elevator beam dimension = 200x 650 mm^2
f'_c = 30 MPa (28 days)

clear cover		= 30 mm
bar's diameter		= 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2
b		= 200 mm
d		= 599 mm
d'		= 51 mm
ϕ		= 0,9
M_u	= 17524,47 kgm	= 175244722 Nmm

<u>Flexural Bar</u>
due to Mu

$$\text{Rn} = \frac{\text{Mn}}{\text{b} \times d^2} = 2{,}173 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0073$$

$\rho_{need} < \rho_{min}$ = 0,0073 > 0,0035
use $\rho = \rho_{min}$ = 0,0073
As_{need} = ρxbxd = 884,75 mm^2
n = As_{need}/As_{bar} = 884,75 mm^2/380,28 mm^2 = 2,32
use n = 3 → As_{use} = $nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 884,75 mm^2 (OK)

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 36,1 mm > 25 mm (OK),use 3D22 reinforcement bar

Stirrup Bar

Use stirrup bar ϕ 10 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)

Vu = 9569,974 kg = 95699,74 N

Vs min = Vu/ϕ = 127599,65 N

A_s = 22/7 x 10 x 10/4 mm^2 = 78,57 mm^2

A_v = 2 x A_s = 157,14 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 177{,}04 \text{ mm}$$

$s_{maks} \leq$ d/2 = 299,5 mm (SNI, 2013 paragraph 21.3.4.3)

Use the space of stirrup bar, s = 150 mm, (ϕ10-150)

Development Bar's Length

$\ell_{dh} = 8xd_b$ = 8x22 mm = 176 mm

(SNI, 2013 paragraph 12.5.1)

ℓ_{dh} = 150 mm

(SNI, 2013 paragraph 12.5.1)

$\ell_{dh} = 0{,}24xfy/d_bx(f'_c)^{0{,}5}$ = 23,3 mm

(SNI, 2013 paragraph 12.5.2)

So, the development bar's length will use ℓ_{dh} = 180 mm with hook $4d_b$ = 88 mm ≈ 100 mm.

4.2.7. Reinforcement of Ramp Structure

4.2.7.1. General Data

The general data for reinforcement of ramp structure will be shown below (cast in place concrete, assuming the support is roller-pins, Figure 4.37).

f'_c	= 30 MPa
f_y	= 390 MPa
height	= 4000 mm
bordes's length	= 5000 mm

bordes's width = 2000 mm
ramp's width = 5000 mm
ramp's length (horizontal) = 6000 mm
ramp's slab thickness (t_p) = 300 mm
bordes's slab thickness = 300 mm
bordes's elevation = 4000 mm
ramp's slab length $= (4^2 + 6^2)^{1/2}$ m
= 7,2 m = 7200 mm
slope (α^0) $= 33{,}69^0$
→ $\tan^{-1} \alpha^0$ = bordes's elevation/ramp's length (horizontal)
→ $\tan^{-1} \alpha^0 = 4000$ mm/6000 mm
→ $\alpha^0 = 33{,}69^0$

checking conditions of slope:
$25^0 \leq \alpha^0 \leq 40^0$
$25^0 \leq 33{,}69^o \leq 40^0$ (OK)

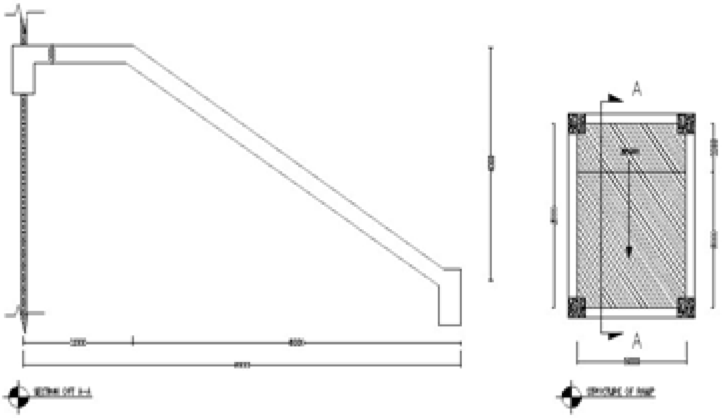


Figure 4.37. The structure of ramp
(source: Private Documentation)

4.2.7.2. Load Calculation

Ramp's Load

1. dead load

ramp's slab	: $t_p/\cos \alpha^0$ x2400x1 kg/m	= 865,3 kg/m
mortar (mixture) 2 cm	: 2x21 kg/m	= 42 kg/m
DL (total)	:	= 907,3 kg/m

2. live load

live load	: 1x800 kg/m	= 800 kg/m
LL (total)	:	= 800 kg/m

load combination

Qu = 1,2DL + 1,6LL.

Qu = 1,2×907,3 + 1,6x800= 2368,76 kg/m = q_2

Bordes's Load

1. dead load

bordes's slab	: 0,3x1x2400 kg/m	= 720 kg/m
mortar (mixture) 2 cm	: 2x21 kg/m	= 42 kg/m
DL (total)	:	= 762 kg/m

2. live load

live load	: 1x800 kg/m	= 800 kg/m
LL (total)	:	= 800 kg/m

load combination

Qu = 1,2DL + 1,6LL.

Qu = 1,2×762 + 1,6x800= 2194,4 kg/m = q_1

4.2.7.3. Moment Calculation

The moment calculation of ramp's structure will be shown below (assuming the support is roller-pins, Figure 4.38).

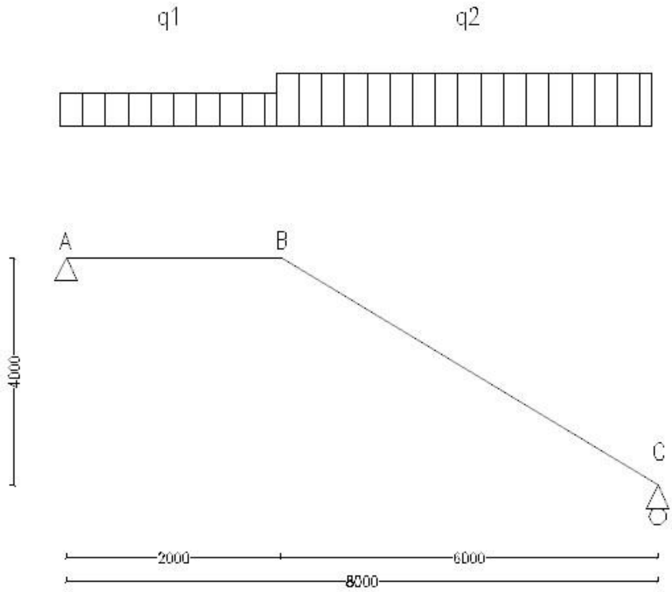


Figure 4.38. Load distribution of ramp
(source: Private Documentation)

$\sum M_A = 0$
$(V_C x8) - (q_2 x6x(6/2 + 2)) - (q_1 x2x(2/2)) = 0$
$V_C = 9432{,}21$ kg

$\sum M_C = 0$
$(V_A x8) - (q_1 x2x(2/2 + 6)) - (q_2 x6x(6/2)) = 0$
$V_A = 9170{,}36$ kg

$\sum H = 0$

$\sum V = 0$
$V_A + V_C = (q_1 x2\ m) + (q_2 x6\ m)$
18602,57 kg = 18602,57 kg

Bordes A-B (2 m)

a. moment force (M)

$X = V_A/q_1$

$= 9170{,}36$ kg/(1421,6 kg/m)

$= 4{,}17$ m (not meet condition $\rightarrow X_{max} = 2$ m)

$M_A = 0$ kgm

$M_{B\ left} = V_A x\ X - ½\ x\ q_1\ x\ X^2 \rightarrow (X = 2$ m)

$= 13951{,}93$ kgm

b. shear force (D)

point A

$D_{Aleft} = 0$ kg

$D_{Aright} = V_A = 9170{,}36$ kg

point B

$D_{Bleft} = V_A - Q_1$

$= 9170{,}36$ kg – 4388,8 kg

$= 4781{,}566$ kg

c. normal force (N)

point A

$N_A = 0$ kg

point B

$N_B = 0$ kg

Ramp B-C (6 m)

a. moment force (M)

$X = V_C/q_2$

$= 9432{,}21$ kg/(2368,76 kg/m)

$= 3{,}98$ m (from right side)

$M_{max} = V_C x\ X - ½\ x\ q_2\ x\ X^2 \rightarrow (X = 3{,}98$ m)

$= 18777{,}54$ kgm

$M_C = 0$ kgm

$M_{B\ right} = V_C x\ X - ½\ x\ q_2\ x\ X^2 \rightarrow (X = 6$ m)

$= 13951{,}93$ kgm

b. shear force (D)

point C

$D_{C\ right} = 0$ kg

D_{Cleft} $= V_C x \cos \alpha^0 - (q_2 x \cos \alpha^0 x\ X) \rightarrow X = 0$ m
$= 7848{,}073$ kg

point B

D_{Bright} $= V_C x \cos \alpha^0 - (q_2 x \cos \alpha^0 x\ X) \rightarrow X = 6$ m
$= -3978{,}5$ kg

c. normal force (N)

point C

N_C $= -V_C x \sin\alpha^0 + (q_2 x \sin \alpha^0 x\ X) \rightarrow X = 0$ m
$= -5232{,}05$ kg

point B

N_B $= -V_C x \sin\alpha^0 + (q_2 x \sin \alpha^0 x\ X) \rightarrow X = 6$ m
$= 2652{,}336$ kg

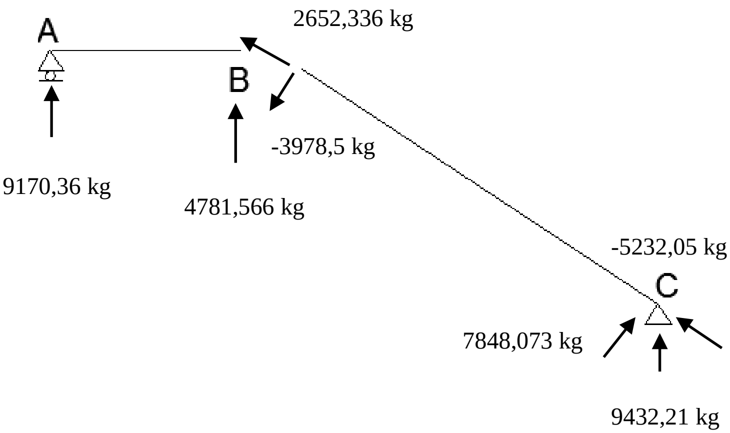


Figure 4.39. Ramp's free-body diagram
(source: Private Documentation)

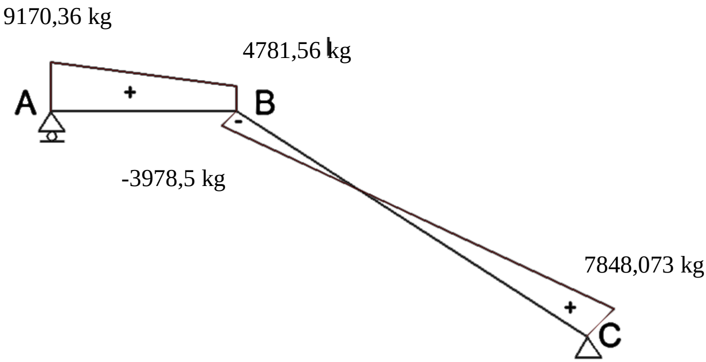


Figure 4.40. Ramp’s shear force diagram (D)
(source: Private Documentation)

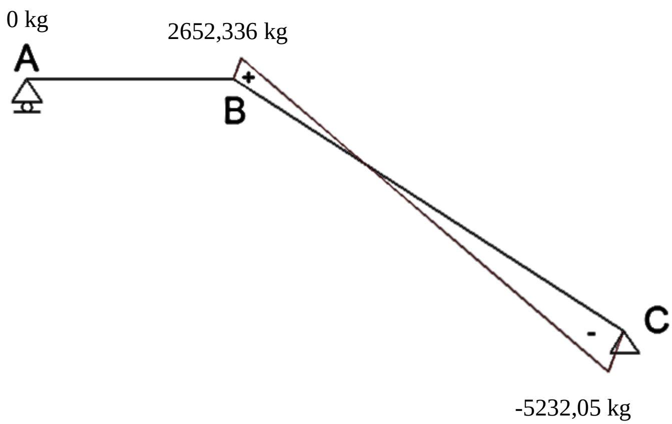


Figure 4.41. Ramp’s normal force diagram (N)
(source: Private Documentation)

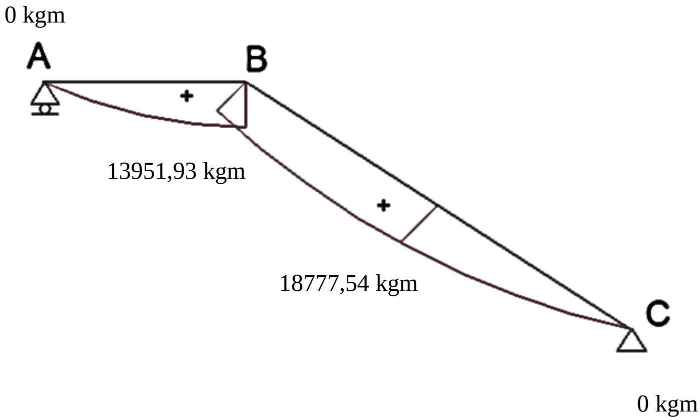


Figure 4.42. Ramp's moment force diagram (M)
(source: Private Documentation)

4.2.7.4. Reinforcement of Ramp's Slab

General Data

The general data for calculation of reinforcement bar of ramp's slab is shown below

ramp's slab thickness	= 300 mm
clear cover	= 30 mm
bar's diameter (D)	= 19 mm
f'_c	= 30 MPa
f_y	= 390 MPa
d = 300 - 30 - 19 - (19/2)	= 241,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\, \frac{(f'c-28)}{7} \geq 0{,}65$

$\beta_1 = 0{,}85 - 0{,}05\, \frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$

$As = \pi/4xd^2 = 283{,}64\ mm^2$

$a = Asxf_y/(0{,}85xf'_cxb) = 4{,}33\ mm$

$c = a/\beta_1 = 5{,}19$ mm
$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}13 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)
$m = f_y/(0{,}85xf'_c) = 15{,}29$
$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1).

<u>Reinforcement Bar of Ramp's Slab</u>

The general data for reinforcement bar of ramp's slab will be shown below

ramp's slab thickness		= 300 mm, f'_c = 30 MPa (28 days)
clear cover		= 30 mm
bar's diameter		= 19 mm, $As = \pi/4xd^2 = 283{,}64$ mm^2
b		= 1000 mm
d		= 241,5 mm
ϕ		= 0,9
M_u	= 18777,54 kgm	= 187775406 Nmm

<u>Main Bar/Longitudinal Bar</u>
$M_n = M_u/\phi = 208639340$ Nmm

$$\text{Rn} = \frac{\text{Mn}}{\text{b} \times d^2} = 3{,}57\,N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0099$$

$\rho_{need} > \rho_{min} = 0{,}0099 > 0{,}002$
use $\rho = \rho_{need} = 0{,}0099$
$As_{need} = \rho xbxd = 2397{,}16$ mm^2
$n = As_{need}/As_{bar} = 2397{,}16$ mm^2/283,64 mm^2 = 8,45
use n = 10 $\rightarrow$ $As_{use} = nxAs_{bar} = 2836{,}4$ mm^2
$As_{use} > As_{need} = 2836{,}4$ mm^2 > 2397,16 mm^2 (OK)
s = b/n = 1000 mm/10 = 100 mm
So, the longitudinal bar will use D19-100.

<u>Shrinkage Bar/Transversal Bar</u>

As_{need} = 0,002xbxd = 483 mm^2

n = As_{need}/As_{bar} = 483 mm^2/283,64 mm^2 = 1,74

use n = 5 → As_{use} = $nxAs_{bar}$ = 1418,21 mm^2

$As_{use} > As_{need}$= 1418,21 mm^2 > 483 mm^2 (OK)

s = b/n = 1000 mm/5 = 200 mm

So, the transversal bar will use D19-200.

4.2.7.5. Reinforcement of Bordes's Slab

<u>General Data</u>

The general data for calculation of reinforcement bar of bordes's slab is shown below

bordes's slab thickness	= 300 mm
clear cover	= 30 mm
bar's diameter (D)	= 19 mm
f'_c	= 30 MPa
f_y	= 390 MPa
d = 300 - 30 - 19 - (19/2)	= 241,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$

As = $\pi/4xd^2$ = 283,64 mm^2

a = $Asxf_y/(0{,}85xf'_cxb)$ = 4,33 mm

c = a/β_1 = 5,19 mm

ε_t = (d/c - 1)x0,003 = 0,13 → ϕ = 0,9 (SNI, 2013 Figure S9.3.2)

m = $f_y/(0{,}85xf'_c)$ = 15,29

ρ_{min} = 0,002 (SNI, 2013 paragraph 7.12.2.1).

Reinforcement Bar of Bordes's Slab

The general data for reinforcement bar of bordes's slab will be shown below

bordes's slab thickness = 300 mm, f'_c = 30 MPa (28 days)
clear cover = 30 mm
bar's diameter = 19 mm, As = $\pi/4xd^2$ = 283,64 mm^2
b = 1000 mm
d = 241,5 mm
ϕ = 0,9
M_u = 13951,93 kgm = 139519319,8 Nmm

Main Bar/Longitudinal Bar

$M_n = M_u/\phi$ = 155021466,4 Nmm

$$Rn = \frac{Mn}{b \times d^2} = 2{,}65 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1-\sqrt{1-\frac{2\times m\times Rn}{fy}}\right) = 0{,}0072$$

$\rho_{need} > \rho_{min}$ = 0,0072 > 0,002
use $\rho = \rho_{need}$ = 0,0072
As_{need} = ρxbxd = 1742,015 mm^2
n = As_{need}/As_{bar} = 1742,015 mm^2/283,6 mm^2 = 6,14
use n = 10 → As_{use} =nxAs_{bar} = 2836,4 mm^2
$As_{use} > As_{need}$= 2836,4 mm^2 > 1742,015 mm^2 (OK)
s = b/n = 1000 mm/10 = 100 mm
So, the longitudinal bar will use D19-100.

Shrinkage Bar/Transversal Bar

As_{need} = 0,002xbxd = 483 mm^2
n = As_{need}/As_{bar} = 483 mm^2/283,64 mm^2 = 1,74
use n = 5 → As_{use} = nxAs_{bar} = 1418,21 mm^2
$As_{use} > As_{need}$= 1418,21 mm^2 > 483 mm^2 (OK)
s = b/n = 1000 mm/5 = 200 mm
So, the transversal bar will use D19-200.

4.2.7.6. Reinforcement of Bordes's Beam

General Data

The general data for calculation of reinforcement bar of bordes's beam is shown below

bordes's beam dimension	= 400 mm × 800 mm
clear cover	= 40 mm
bar's diameter (D)	= 22 mm
bar's diameter/stirrup(ϕ/plain)	= 10 mm
f'_c	= 30 Mpa
f_y	= 390 MPa
d' = clear cover + stirrup + D/2	= 61 mm
d = 800 - 40 - 10 - (22/2)	= 739 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$

$As = \pi/4xd^2 = 380{,}28\ mm^2$

$a = Asxf_y/(0{,}85xf'_cxb) = 14{,}54$ mm

$c = a/\beta_1 = 17{,}39$ mm

$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}124 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)

$m = f_y/(0{,}85xf'_c) = 15{,}29$

$\rho_{min} = ¼ \times (f'_c)^{0{,}5}/f_y = 0{,}0035$ (SNI, 2002 paragraph 12.5.1)

Load Calculation of Bordes's Beam

The load calculation for bordes's beam will be shown below

1. dead load

bordes's beam	: 0,4x0,8x2400 kg/m	= 768 kg/m
brick wall	: 5x250 kg/m	= 1250 kg/m
1,4 DL	: 1,4(768 + 1250) kg/m = 2825,2 kg/m	
bordes's slab	: 2194,4 kg/m = 2194,4 kg/m	
DL (total)	: = 5019,6 kg/m	

<u>Load Combination of Bordes's Beam</u>

The combination of load for bordes's beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of bordes's beam

Qu = 1,2×5019,6 + 1,6x0 = 6023,52 kg/m.

<u>Moment Calculation of Bordes's Beam</u>

According to SNI (2013), the equation of moment values for bordes's beamare shown below

Ml (+) = 1/16 x qx L^2 → at field area

Mt (-) = 1/10 x qx L^2 → at support area

moment calculation of bordes's beam

(q = 6023,52 kg/m, L = 5 m)

Ml (+) = 9411,75 kgm

Mt(-) = 15058,8 kgm

The Mu values are taken as

Mul = 9411,75 kgm (+)

Mut = 15058,8 kgm (-)

<u>Shear Force Calculation of Bordes's Beam</u>

The shear force calculation of bordes's beam will use shear force equation Vu = 1/2 x q x L.

shear force calculation of bordes's beam

(q = 6023,52 kg/m, L = 5 m)

Vu = 15058,8 kg

<u>Reinforcement Bar of Bordes's Beam</u>

The general data for reinforcement bar of bordes's beam will be shown below

bordes's beam's dimension = 400 x 800 mm^2

f'_c = 30 MPa (28 days)

clear cover = 40 mm

bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2

b		= 400 mm
d		= 739 mm
d’		= 61 mm
ϕ		= 0,9
M_{ul}	= 9411,75 kgm	= 94117500 Nmm
M_{ut}	= 15058,8 kgm	= 150588000 Nmm

<u>Flexural Bar</u>
due to M_{ul}

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0{,}57 N / mm^2$$

$$\rho_{\mathrm{need}} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2 \times \mathrm{m} \times \mathrm{Rn}}{\mathrm{fy}}}\right) = 0{,}0012$$

$\rho_{need} < \rho_{min}$ = 0,0012 < 0,0035
use $\rho = \rho_{min}$ = 0,0035
As_{need} = ρxbxd = 1034,6 mm^2
n = As_{need}/As_{bar} = 1034,6 mm^2/380,28 mm^2 = 2,72
use n = 3 → As_{use} = nxAs_{bar} = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 1034,6 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 100 mm > 25 mm (OK), use 3D22 reinforcement bar

<u>Flexural Bar</u>
due to M_{ut}

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0{,}92 N / mm^2$$

$$\rho_{\mathrm{need}} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2 \times \mathrm{m} \times \mathrm{Rn}}{\mathrm{fy}}}\right) = 0{,}0018$$

$\rho_{need} < \rho_{min}$ = 0,0018 < 0,0035
use $\rho = \rho_{min}$ = 0,0035
As_{need} = ρxbxd = 1034,6 mm^2

$n = As_{need}/As_{bar} = 1034{,}6\ mm^2/380{,}28\ mm^2 = 2{,}72$
use $n = 3 \rightarrow As_{use} = nxAs_{bar} = 1140{,}86\ mm^2$
$As_{use} > As_{need} = 1140{,}86\ mm^2 > 1034{,}6\ mm^2$ (OK)
$s = b$ – 2xclear cover - 2ϕ - n x D/(n - 1) = 100 mm > 25 mm (OK), use 3D22 reinforcement bar

Stirrup Bar
Use stirrup bar ϕ 10 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 15058,8 kg = 150588 N
Vs min = Vu/ϕ = 200784 N
A_s = 22/7 x 10 x 10/4 mm^2 = 78,57 mm^2
A_v = 2 x A_s = 157,14 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 225{,}56\ mm$$

$s_{maks} \leq d/2$ = 369,5 mm (SNI, 2013 paragraph 21.3.4.3)
Use the space of stirrup bar, s = 100 mm, (ϕ10-100).

4.3. Modelling of Structure
4.3.1. Preface

The modelling of structure is based on SNI 2847:2013 and the calculation of its seismic load is based on SNI 1726:2012. The programme which is used for modelling structure is SAP 2000 v14®.

4.3.2. General Data

The general data for modelling structure of HOTEL NOVOTEL THE SAMATOR SURABAYA will be shown below

f'_c	= 30 Mpa
f_y	= 390 MPa
height	= approx. 44 meters

height:	
per storey	= 4 meters, 10 storeys
basement	= 4 meters, 1 storey
dimension:	
primary beam	= 40/80
secondary beam	= 30/60
pillar (rectangular)	= 80/80
earthquake zone	= Surabaya, low seismic load zone

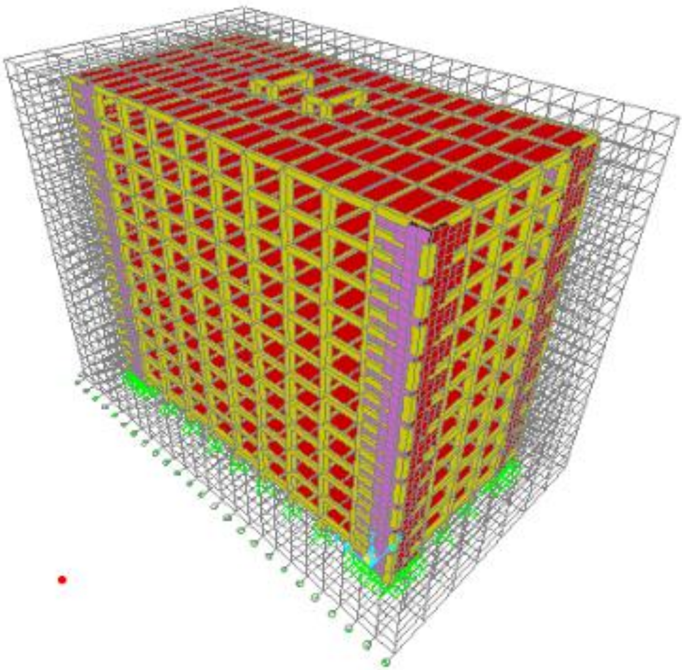


Figure 4.43. The structure's model in SAP 2000v14®
(source: Private Documentation)

4.3.3. Load Combinations

The loadcombinations for modelling structure will use combinations in SNI 1727:2013 paragraph 9.2.1 explains about load combination as the following

- U = 1,4 D
- U = 1,2 D + 1,6 L
- U = 1,2 D + 1,0 L + 1,0 E_x
- U = 1,2 D + 1,0 L + 1,0 E_y
- U = 1 D + 1 L
- U = 0,9 D + 1,0 E_x
- U = 0,9 D + 1,0 E_y

in which D is dead load, L is live load, E is seismic load, and U is ultimate load.

4.3.4. Analyzing Seismic Load

4.3.4.1. Spectrum Respons (MCE_R)

The quake zone map will be shown in Figure 4.44 and Figure 4.45. From the quake zone map, then it can be determined the value of S_s (respons spectral parameter, short periode, T = 0,2 sec.) and S_1 (respons spectral parameter, periode, T = 1 sec.). From Figure 4.46 and Figure 4.47, the value of S_S for Surabaya City is 0,7 g, and the value of S_1 for Surabaya City is 0,25 g.

Hence, the value of standard penetration which is based on SNI 1726:2012 paragraph 5.4.2 will be shown below

$$\mathrm{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} = \frac{2 \times 10}{(\frac{2}{1} + \frac{2}{1} + \frac{2}{1} + \frac{2}{1} + \frac{2}{32} + \frac{2}{35} + \frac{2}{53} + \frac{2}{60} + \frac{2}{52} + \frac{2}{54}}$$

N = 2,419 < 15 → Site Class SE (SNI, 2012 paragraph 5.3)

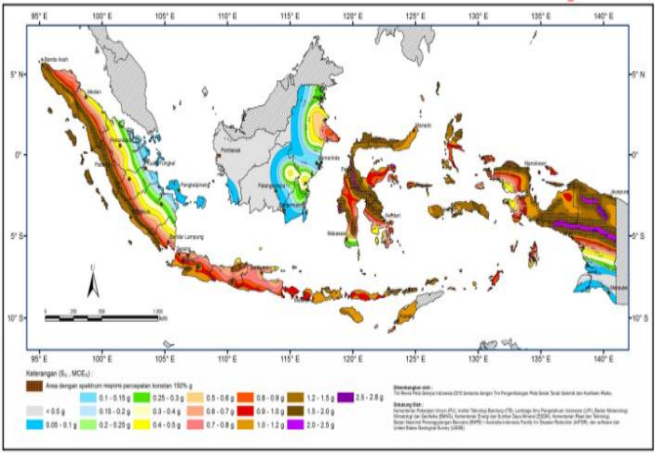


Figure 4.44. The quake zone map, for S_S
(source: SNI, 2012)

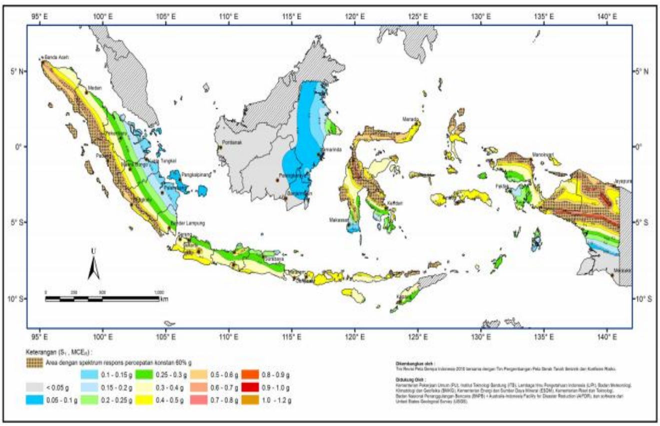


Figure 4.45. The quake zone map, for S_1
(source: SNI, 2012)

For the value of F_a (site coefficient, T = 0,2 sec.), and F_v (site coefficient, T = 1 sec.), it can be determined by looking at SNI 1726:2012 Table 4 and Table 5 (see Table 4.13 and Table 4.14).

Table 4.13. Site Coefficient, F_a
(source: SNI, 2012)

Site Class	S_S Parameter				
	$S_S \leq 0{,}25$	$S_S = 0{,}5$	$S_S = 0{,}75$	$S_S = 1$	$S_S \geq 1{,}25$
SA	0,8	0,8	0,8	0,8	0,8
SB	1	1	1	1	1
SC	1,2	1,2	1,1	1	1
SD	1,6	1,4	1,2	1,1	1
SE	2,5	1,7	1,2	0,9	0,9
SF	SS^b				

Table 4.14. Site Coefficient, F_v
(source: SNI, 2012)

Site Class	S_1 Parameter				
	$S_1 \leq 0{,}1$	$S_1 = 0{,}2$	$S_1 = 0{,}3$	$S_1 = 0{,}4$	$S_1 \geq 0{,}5$
SA	0,8	0,8	0,8	0,8	0,8
SB	1	1	1	1	1
SC	1,7	1,6	1,5	1,4	1,3
SD	2,4	2	1,8	1,6	1,5
SE	3,5	3,2	2,8	2,4	2,4
SF	SS^b				

then,

S_S = 0,7 g

S_1 = 0,25 g

F_a = 1,3 (by linear interpolation)

F_v = 3 (by linear interpolation)

S_{MS} = F_a x S_S (SNI, 2012 paragraph 6.2)

= 0,91

S_{M1} = F_v x S_1 (SNI, 2012 paragraph 6.2)

= 0,75

4.3.4.2. Spectral Respons Parameter

The spectral respons graphic will be shown in Figure 4.46.

S_{DS} = 2/3 x S_{MS} (SNI, 2012 paragraph 6.3)

= 0,61

S_{D1} = 2/3 x S_{M1} (SNI, 2012 paragraph 6.3)

= 0,5

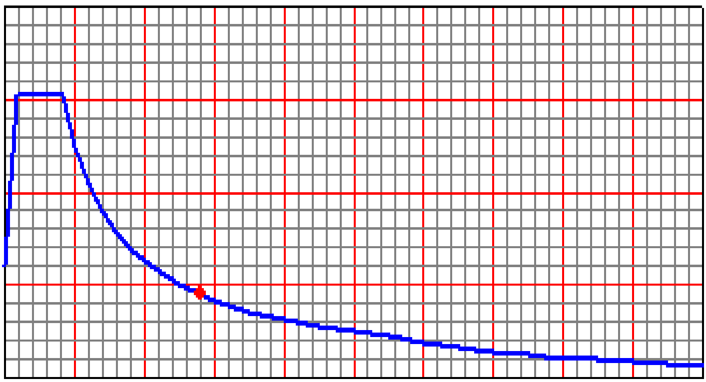


Figure 4.46. The spectral respons graphic
(source: Private Documentation)

For the seismic design category, it is based on the value of S_{DS} and S_{D1} (see Table 4.15 and Table 4.16).

Table 4.15. Seismic Design Category, Short Periode (source: SNI, 2012)

S_{DS} Value	Risk Category	
	I, or, II, or, III	IV
$S_{DS} < 0,167$	A	A
$0,167 < S_{DS} < 0,33$	B	C
$0,33 < S_{DS} < 0,50$	C	D
$0,50 \leq S_{DS}$	D	D

Table 4.16. Seismic Design Category, Short Periode (source: SNI, 2012)

S_{DS} Value	Risk Category	
	I, or, II, or, III	IV
$S_{D1} < 0,067$	A	A
$0,067 < S_{D1} < 0,133$	B	C
$0,133 < S_{D1} < 0,20$	C	D
$0,20 \leq S_{D1}$	D	D

From the table above, it can be determined that the sesismic design category for this building (risk category I) is category D. According to SNI 1726:2012 Table 9, the design criteria which will be used is dual system with special moment resisting frame.

4.3.5. Dynamic Seismic Load

4.3.5.1. Direction of Seismic Load

The direction of seismic load has to be modelled into two directions although when eartquake occurs, it never occurs in two directions. So, the seismic at X direction is modelled into 100% effectiveness when eartquake occurs, and 30% at Y direction. Hence, the seismic at Y direction is modelled into 100% effectiveness when eartquake occurs, and 30% at X direction.

4.3.5.2. Seismic Reduction Factor (R)

According to SNI 1726:2012 Table 9, the value of seismic reduction factors are

Deflection Factor Multiplier, C_d = 5,5

System Factor Value, Ω = 2,5

Respons Modiffication Coefficient, R = 7

4.3.5.3. Main Factor of Building (I)

According to SNI 1726:2012 Table 2, the value of this factor, I_e, is 1,0 (note that the risk category is I, see SNI 1726:2012 Table 1).

Table 4.17. The Value of Factor, I_e
(source: SNI, 2012)

Risk Category	Factor, I_e
I or II	1,0
III	1,25
IV	1,5

4.3.6. Design Control

4.3.6.1. Gravity Load Control

The gravity load from SAP 2000 v14® (see Table 4.19) will be compared with manual calculation. The manual calculation will be shown in Table 4.18. Note that the combination which will be used for this chapter is 1,0 DL + 1,0 LL.

Table 4.18. The Result of Manual Calculation of Gravity Load

Storey(s)	Load (N)
LG/Basement	49233040
G	32637040
1	32637040
2	32637040
3	32637040
4	32637040
5	32637040
6	32637040
7	32637040
8	32637040
9	32637040
Roof	26672800
Total	402276240

Table 4.19. The Result of Gravity Load (from SAP 2000 v14®)

Output Case	Case Type	Global FZ
Text	Text	N
1,4 DL	Combination	492487275
1,2 DL + 1,6 LL	Combination	475633895
1,2 DL + 1,0 LL + 1,0 E_x	Combination	455570660
1,2 DL + 1,0 LL + 1,0 E_y	Combination	455570672
1,0 DL + 1,0 LL	Combination	385215341
0,9 DL + 1,0 E_x	Combination	316598971
0,9 DL + 1,0 E_y	Combination	316598956

From the tables above,

- Load (from manual calculation) = 402274240N
- Load (from SAP 2000 v14®) = 385215341N
- The comparision:

= (402274240N - 385215341 N)/385215341 N x 100%
= 4,29%

The difference between the calculations is less than 10%. So, the SAP 2000 v14® result is reliable.

4.3.6.2. Fundamental Natural Periode (T)

The value of fundamental natural periode, T, is obtained by multiplying T_a by C_u. The value of T_a itself is obtained by equation in SNI 1726:2012, paragraph 7.8.2.1. Note that the value of C_u depends on S_{D1} (see SNI, 2012 Table 14).

T_a = C_t x h_n^x (SNI, 2012 paragraph 7.8.2.1)
T = T_a x C_u
Which,
C_t = 0,0466 (SNI, 2012 Table 15)
x = 0,9 (SNI, 2012 Table 15)
C_u = 1,4 (SNI, 2012 Table 14)
h_n = 44 meters

then,
T_{ax} = 0,0466 x $44^{0,9}$ = 1,40 sec.
T_{ay} = 0,0466 x $44^{0,9}$ = 1,40 sec.
T = 1,40 sec. x 1,4 = 1,96 sec.

The result, T, will be compared with SAP 2000 v14® result (see Table 4.20).

Table 4.20. Modal Periods and Frequencies (from SAP 2000 v14®)

Output Case	Step Type	Step Num	Period	Frequency	Circ Freq	Eigenvalue
Text	Text	Unitless	Sec	Cyc/sec	rad/sec	rad2/sec2
MODAL	Mode	1	1,187	0,842	5,295	28,046
MODAL	Mode	2	1,127	0,887	5,576	31,093
MODAL	Mode	3	0,840	1,190	7,477	55,909
MODAL	Mode	4	0,351	2,845	17,877	319,59
MODAL	Mode	5	0,328	3,049	19,162	367,18
MODAL	Mode	6	0,222	4,508	28,325	802,30
MODAL	Mode	7	0,202	4,933	30,999	960,91
MODAL	Mode	8	0,177	5,634	35,400	1253,2
MODAL	Mode	9	0,158	6,299	39,580	1566,6
MODAL	Mode	10	0,154	6,481	40,724	1658,5
MODAL	Mode	11	0,149	6,672	41,924	1757,6
MODAL	Mode	12	0,143	6,986	43,895	1926,7

From Table 4.20, the maximum period is 1,187 sec., which is less than T = 1,96 sec.

- $T \leq T$
- 1,187 sec. $\leq$ 1,96 sec. (OK)

4.3.6.3. Sesismic Respons Coefficient (C_s)

The value of seismic respons coefficient, C_s, is obtained by using equation in SNI 1726:2012, paragraph 7.8.1.1.

C_s = $S_{DS}/(R/I_e)$ (SNI, 2012 paragraph 7.8.1.1)
= 0,61/(7/1)
= 0,087

Hence, the value of C_s should be less than

C_s = $S_{D1}/(T \text{ x } R/I_e)$ (SNI, 2012 paragraph 7.8.1.1)
= 0,5/(1,96x7/1)
= 0,036

the value of C_s should not less than

C_s = 0,044 x S_{DS} x I_e
= 0,044 x 0,61 x 1
= 0,02684

then, the value of C_s is 0,02684.

The value of C_s will be used for the calculation of V (base shear)

V_{static} = C_s x W

in which W is 385215341 N (see Table 4.20)

V_{static} = 0,02684 x 385215341 N
= 10339179,75 N

then, the value of V_{static} will be compared with the value of $V_{dinamic}$

Table 4.21. Base Shear Due to Seismic Load (from SAP 2000 v14®)

Output Case	Global FX	Global FY
Text	N	N
E_x	11242428,6	3567154,2
E_y	3372728,6	11890513,9

then,

for X direction

$V_{dinamic} \geq 85\%$ x V_{static}

11242428,6N ≥ 85/100 x 10339179,75 N

11242428,6N ≥ 8788302,79N (OK)

for Y direction

$V_{dinamic} \geq 85\%$ x V_{static}

11890513,9N ≥ 85/100 x 10339179,75 N

11890513,9N ≥8788302,79 N (OK)

4.3.6.4. Mass Participating Ratio

The mass participating ratio of the structure should not less than 90% in both direction (X direction and Y direction).

Table 4.22. Mass Participating Ratio (from SAP 2000 v14®)

Output Case	Step Type	Step Num	Period	Sum UX	Sum UY
Text	Text	Unitless	Sec	Unitless	Unitless
MODAL	Mode	1	1,187	0,69	0
MODAL	Mode	2	1,127	0,69	0,69
MODAL	Mode	3	0,843	0,69	0,69
MODAL	Mode	4	0,351	0,82	0,69
MODAL	Mode	5	0,328	0,82	0,83
MODAL	Mode	6	0,222	0,82	0,83
MODAL	Mode	7	0,202	0,87	0,83
MODAL	Mode	8	0,177	0,87	0,88
MODAL	Mode	9	0,158	0,87	0,88
MODAL	Mode	10	0,154	0,87	0,88
MODAL	Mode	11	0,149	0,89	0,89
MODAL	Mode	12	0,143	0,90	0,90

From the table above, the maximum mass participating ratio at X direction is 90%, whereas 90% at Y direction. So, it does meet the condition in SNI 1726:2012 paragraph 7.9.1.

4.3.6.5. Load Participating Ratio

The load participating ratio (dynamic percentage) of the structure should not less than 90% in both direction (X direction and Y direction).

Table 4.23. Load Participating Ratio
(from SAP 2000 v14®)

Output Case	Item Type	Item	Static	Dynamic
Text	Text	Text	Percent	Percent
MODAL	Acceleration	UX	100	90
MODAL	Acceleration	UY	100	90

4.3.6.6. Drift

According to SNI 1726:2012 paragraph 7.8.6, the drift condition for displacement will be shown below

$\Delta_i \leq \Delta_a \rightarrow$ (SNI, 2012 paragraph 7.8.6)
in which,
$\Delta_1 = C_d \text{ x } \delta_{e1}/I_e \rightarrow$ for first floor
$\Delta_2 = C_d \text{ x } (\delta_{e2} - \delta_{e1})/I_e \rightarrow$ for second floor, etc.

hence,
$\Delta_a = 0{,}020 \text{ x } h_{sx} \rightarrow$ (SNI, 2012 paragraph 7.12.1)
in which, h_{sx} = 4 m (height per storey)
the table of displacement value of the structure (δ) will be shown below

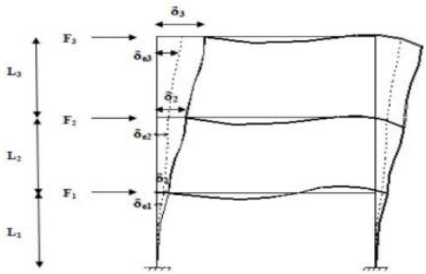


Figure 4.47. The example of drift
(source: Private Documentation)

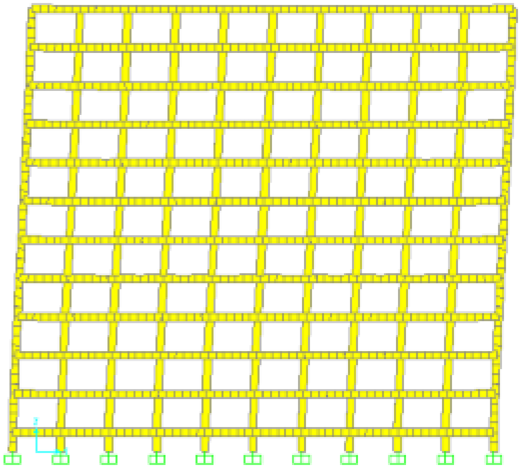


Figure 4.48. The drift (Y direction)
(source: Private Documentation)

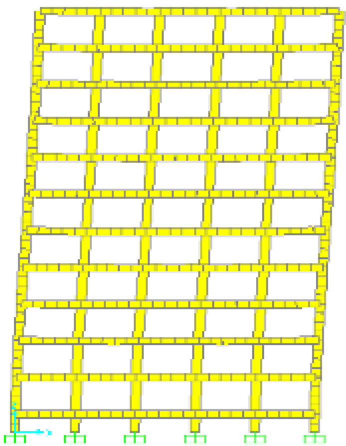


Figure 4.49. The drift (X direction)
(source: Private Documentation)

Table 4.24. Displacement (δ) from SAP 2000 v14®

Storey	Height	Due to E_x		Due to E_y	
	h_i (m)	δ (mm)		δ (mm)	
		X Direction	Y Direction	X Direction	Y Direction
Roof	40	21,132	6,039	6,340	20,123
9	36	19,467	5,590	5,840	18,633
8	32	17,712	5,115	5,314	17,047
7	28	15,814	4,594	4,744	15,302
6	24	13,773	4,024	4,132	13,396
5	20	11,615	3,414	3,485	11,355
4	16	9,385	2,776	2,816	9,224
3	12	7,147	2,129	2,145	7,063
2	8	4,983	1,496	1,496	4,953
1	4	3,002	0,909	0,902	3,004
G	0	1,356	0,413	0,407	1,365
LG	-4	0	0	0	0

Table 4.25. Control of Drift (Δ) due to E_x at X Direction

Storey	Height	Drift	Drift	Note
	h_i (m)	Δ_i (mm)	Δ_a (mm)	
Roof	40	9,154	80	OK
9	36	9,653	80	OK
8	32	10,439	80	OK
7	28	11,225	80	OK
6	24	11,868	80	OK
5	20	12,262	80	OK
4	16	12,310	80	OK
3	12	11,903	80	OK
2	8	10,892	80	OK
1	4	9,056	80	OK
G	0	7,459	80	OK
LG	-4	0	80	OK

Table 4.26. Control of Drift (Δ)due to E_x at Y Direction

Storey	Height h_i (m)	Drift Δ_i (mm)	Drift Δ_a (mm)	Note
Roof	40	2,751	80	OK
9	36	2,895	80	OK
8	32	3,130	80	OK
7	28	3,366	80	OK
6	24	3,559	80	OK
5	20	3,677	80	OK
4	16	3,691	80	OK
3	12	3,570	80	OK
2	8	3,269	80	OK
1	4	2,720	80	OK
G	0	2,241	80	OK
LG	-4	0	80	OK

Table 4.27. Control of Drift (Δ) due to E_y at X Direction

Storey	Height h_i (m)	Drift Δ_i (mm)	Drift Δ_a (mm)	Note
Roof	40	2,468	80	OK
9	36	2,610	80	OK
8	32	2,868	80	OK
7	28	3,132	80	OK
6	24	3,356	80	OK
5	20	3,506	80	OK
4	16	3,558	80	OK
3	12	3,481	80	OK
2	8	3,232	80	OK
1	4	2,728	80	OK
G	0	2,272	80	OK
LG	-4	0	80	OK

Table 4.28. Control of Drift (Δ) due to E_y at Y Direction

Storey	Height h_i (m)	Drift Δ_i (mm)	Drift Δ_a (mm)	Note
Roof	40	8,193	80	OK
9	36	8,721	80	OK
8	32	9,602	80	OK
7	28	10,482	80	OK
6	24	11,224	80	OK
5	20	11,723	80	OK
4	16	11,885	80	OK
3	12	11,601	80	OK
2	8	10,723	80	OK
1	4	9,014	80	OK
G	0	7,507	80	OK
LG	-4	0	80	OK

4.3.6.7. Shear Force Control (Shear Wall)

For the shear force control (shear wall), the seismic load should be resistedby the structure frame is approximately at least 25% (see Table 4.29).

Table 4.29. Percentage of Shear Force Which is Resisted by The Structure's System

Shear Force	X Direction	Percentage	Y Direction	Percentage
	N	%	N	%
Shear Wall	10424559	65	10587736	63
Structure Frame	5704467	35	6358208	37
Total	16129026	100	16945944	100

4.4. Reinforcement of Primary Structure

4.4.1. Preface

Reinforcement of primary structure consists of reinforcement of primary beam, reinforcement of column, and reinforcement of shearwall. Beside the reinforcement of primary structure, in this chapter will be explained about calculation of lifting point of precast element such as primary beam.

4.4.2. Reinforcement of Precast Primary Beam

4.4.2.1. Reinforcement of Precast Primary Beam Before Monolith Condition

General Data

The general data for reinforcement of primary beam will be shown below

b (width)	= 40 cm
h (height, before monolith)	= 65 cm
h (height,after monolith)	= 45 cm + hf (floor) = 80 cm
L(axis to axis)	= 5000 mm
L(actual)	= 4200 mm
bar's diameter (D/deform)	= 22 mm
bar's diameter/stirrup(D/deform)	= 13 mm
f'_c(14 days) = 0,88*30 MPa	= 26,4 MPa
f_y	= 390 MPa

Load Calculation

The load calculation of precast primary beam will be assumed as trapezoid shape. The loads consist of deal load and live load. The load $q_{eqivalent}$ (q_{eq}) will be shown below

$q_{eq} = q \times L_x \times 1/2 \times ((1 - L_x^2/L_y^2) \times 1/3))$

Before Monolith Condition

In this condition itself will be divided into two conditions below

1. condition a

when the *overtopping* concrete is not installed yet and the load consists of working load (construction process' load), precast slab element's load,precast secondary beam element's load, and precast primary beam element's load

2. condition b

when the *overtopping* concrete is already installed (not yet in monolith condition) and the load consists of *overtopping* concrete's load, precast slab element's load, precast secondary beam element's load, precast primary beam element's load, and construction process' load.

Then the most critical condition between two conditions above will be used for load calculation

1. dead load (secondary beam)

secondary beam BAL1 : 0,3x0,45x2400 kg/m= 324 kg/m

q of slab LA1

(condition a) : 608 kg/m^2 = 608 kg/m^2

q of slab LA1

(condition b) : 809,6 kg/m^2 = 809,6 kg/m^2

by using L_y = 500 cm – (40 cm/2 + 40 cm/2) = 460 cm and L_x = 250 cm – (40 cm/2 + 30 cm/2) = 215 cm, it is obtained the value of q_{eq} due to slab's load

(two trapezoid loads)

q_{eq} = 2 x q x 1/2 x L_x x ((1 -1/3 x $(L_x/L_y)^2$)

q (condition a) = 608 kg/m^2

q_{eq} = 1212,01 kg/m → due to slab's load

q (condition b) = 809,6 kg/m^2

q_{eq} = 1613,89 kg/m → due to slab's load

DL (condition a) = 1212,01 kg/m + 324 kg/m

= 1536,01 kg/m

DL (k, shock coefficient = 1,2) = 1,2x1536,01 kg/m
= 1843,21 kg/m
DL (condition b) = 1613,89 kg/m +324 kg/m
= 1937,89 kg/m
DL (k, shock coefficient = 1,2) = 1,2x1937,89 kg/m
= 2325,47 kg/m

DL (total, condition a) : = 1843,21 kg/m
DL (total, condition b) : = 2325,47 kg/m

2. live load (secondary beam)

construction load: = 200 kg/m^2
q_{eq} = 398,688 kg/m → due to live load

LL (total, condition a and b) : = 398,688 kg/m

3. dead load (primary beam)
primary beam BIL1 : 0,4x0,65x2400 kg/m= 624 kg/m
q of slab LA1
(condition a) : 608 kg/m^2 = 608 kg/m^2
q of slab LA1
(condition b) : 809,6 kg/m^2 = 809,6 kg/m^2

by using L_y = 500 cm – (80 cm/2 + 80 cm/2) = 420 cm and L_x = 250 cm – (80 cm/2 + 30 cm/2) = 195 cm, it is obtained the value of q_{eq} due to slab's load

q_{eq} = q x 1/2 x L_x x ((1 -1/3 x $(L_x/L_y)^2$)
q (condition a) = 608 kg/m^2
q_{eq} = 550,2 kg/m → due to slab's load
q (condition b) = 809,6 kg/m^2
q_{eq} = 732,6kg/m → due to slab's load
DL (condition a) = 550,2 kg/m + 624 kg/m
= 1174,21kg/m
DL (k, shock coefficient = 1,2) = 1,2x1174,21 kg/m
= 1409,04kg/m

DL (condition b) = 732,6 kg/m + 624 kg/m
= 1356,64kg/m
DL (k, shock coefficient = 1,2) = 1,2x1356,64kg/m
= 1627,96kg/m

DL (total, condition a) : = 1409,04 kg/m
DL (total, condition b) : = 1627,96 kg/m

4. live load (primary beam)

construction load: = 200 kg/m^2
q_{eq} = 180,9 kg/m → due to live load

LL (total, condition a and b) : = 180,9 kg/m

Load Combination of Primary Beam

The combination of load for primary beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of primary beam
due to secondary beam
condition a
(precast's age = 14 days)
Qu = 1,2×1843,21 + 1,6x398,688 = 2849,76 kg/m
Pu = Qu x L = 14248,79 kg

condition b
(precast's age = 14 days, *overtopping*'s age = 0 day)
Qu = 1,2x2325,47 + 1,6x398,688 = 3729,06 kg/m
Pu = Qu x L = 18645,28 kg

due to primary beam
condition a
(precast's age = 14 days)
Qu = 1,2×1409,04+ 1,6x180,9 = 1980,43 kg/m
condition b
(precast's age = 14 days, *overtopping*'s age = 0 day)
Qu = 1,2x1627,96 + 1,6x180,9 = 2243,14kg/m

Moment Calculation of Primary Beam

The moment calculation of primary beam will use momentequation

Mu = 1/8 x q x L^2+ ¼ x P x L, in which L is the length of axis to axis of primary beam

moment calculation of primary beam
condition a (q = 1980,43 kg/m, P = 14248,79 kg, L = 5 m)
Mu = 23999,85 kgm
condition b (q = 2243,14 kg/m, P = 18645,28 kg, L = 5 m)
Mu = 30316,42 kgm

The Mu values are taken as
condition a
Mu = 23999,85 kgm
condition b
Mu = 30316,42 kgm
So, the Mu value is 30316,42 kgm.

Shear Force Calculation of Primary Beam

The shear force calculation of secondary beam will use shear force equation Vu = 1/2 x q x L + ½ x P

shear force calculation of primary beam
condition a (q = 1980,43 kg/m, P = 14248,79 kg, L = 5 m)
Vu =12075,48 kg
condition b (q = 2243,14 kg/m, P = 18645,28 kg, L = 5 m)
Vu = 14930,50 kg

The Vu values are taken as
condition a
Vu =12075,48 kg
condition b
Vu = 14930,50 kg

So, the Vu value is 14930,50 kg.

Calculation of Reinforcement Bar

The general data for calculation of reinforcement bar of primary beam is shown below

primary beam's dimension (overall) = 400mm × 650mm
clear cover = 50 mm
bar's diameter (D) = 22 mm
bar's diameter/stirrup(D/deform) = 13 mm
f'_c(28 days) = 30 Mpa
f_y = 390 MPa
d' = clear cover + stirrup + D/2 = 74 mm
d = 650 - 50 - 13 - (22/2) = 576 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 14 days = 26,4 MPa, see Table 4.7)

$\beta_1 = 0,85$
As = $\pi/4xd^2 = 380,286\ mm^2$
a = $Asxf_y/(0,85xf'_cxb) = 16,52$ mm
c = $a/\beta_1 = 19,44$ mm
$\varepsilon_t = (d/c - 1)x0,003 = 0,085 \rightarrow \phi = 0,9$ (SNI, 2013 Figure S9.3.2)
m = $f_y/(0,85xf'_c) = 15,29$
$\rho_{min} = \frac{1}{4}\ x\ (f'_c)^{0,5}/f_y = 0,0032$ (SNI, 2002 paragraph 12.5.1)

Reinforcement Bar

The general data for reinforcement bar of primary beam will be shown below

secondary beam's dimension = 400x 650 mm^2
f'_c = 26,4 MPa (14 days)
clear cover = 50 mm
bar's diameter = 22 mm, As = $\pi/4xd^2 = 380,28\ mm^2$
b = 400 mm
d = 576 mm
d' = 74 mm
ϕ = 0,9

M_u = 30316,42kgm = 303164284 Nmm

Flexural Bar

due to M_u

$$Rn = \frac{Mn}{b \times d^2} = 2{,}53 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0068$$

$\rho_{need} > \rho_{min}$ = 0,0068 > 0,0032

use $\rho = \rho_{min}$ = 0,0068

As_{need} = ρxbxd = 1592,37 mm^2

n = As_{need}/As_{bar} = 1592,37 mm^2/380,28 mm^2 = 4,18

use n = 5 → As_{use} = nxAs_{bar} = 1901,43 mm^2

$As_{use} > As_{need}$ = 1901,43 mm^2 >1592,37 mm^2 (OK)

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 41mm > 25 mm (OK), use 5D22 reinforcement bar

As'_{need} = 0,5xAs_{use} = 950,7 mm^2

use n = 3 → As'_{use} = 3xAs_{bar} = 1140,86 mm^2

$As'_{use} > As'_{need}$ = 1140,86 mm^2> 950,7 mm^2 (OK)

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 104 mm > 25 mm (OK), use 3D22 reinforcement bar

4.4.2.2. Reinforcement Bar Due to Lifting Process' Moment

The general data for reinforcement bar of primary beam due to lifting process's moment will be shown below

secondary beam's dimension = 300x 650 mm^2

f'_c = 26,4 MPa (14 days)

clear cover = 50 mm

bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2

b = 400 mm

d = 576 mm

d' = 74 mm

ϕ = 0,9
M_{ul} = 173,97 kgm = 1739753,718 Nmm
M_{ut} = 533,81 kgm = 5338170,511 Nmm
(see paragraph 4.4.2.3.9)

<u>Flexural Bar</u>
due to M_{ul}

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0{,}014 N/mm^2$$

$$\rho_{need} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2 \times \mathrm{m} \times \mathrm{Rn}}{\mathrm{fy}}}\right) = 0{,}00003$$

$\rho_{need} > \rho_{min}$ = 0,00003 < 0,0032
use $\rho = \rho_{min}$ = 0,0032
As_{need} = ρxbxd = 738,56 mm^2
n = As_{need}/As_{bar} = 738,56 mm^2/380,28 mm^2 = 1,94
use n = 3 → As_{use} = nxAs_{bar} = 1140,85 mm^2
$As_{use} > As_{need}$ = 1140,85 mm^2 > 738,56 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 104 mm > 25 mm (OK), use 3D22 reinforcement bar

As'_{need} = 0,5xAs_{use} = 570,42 mm^2
use n = 3 → As'_{use} = 3xAs_{bar} = 1140,86 mm^2
$As'_{use} > As'_{need}$ = 1140,86 mm^2 > 570,42 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 104 mm > 25 mm (OK), use 3D22 reinforcement bar

<u>Flexural Bar</u>
due to M_{ut}

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0{,}044 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1-\sqrt{1-\frac{2\times m\times Rn}{fy}}\right) = 0{,}00011$$

$\rho_{need} < \rho_{min}$ = 0,00011 < 0,0032
use $\rho = \rho_{min}$ = 0,0032
$As_{need} = \rho x b x d$ = 738,56 mm^2
n = As_{need}/As_{bar} = 738,56 mm^2/380,28 mm^2 = 1,94
use n = 3 → As_{use} = $nxAs_{bar}$ = 1140,85 mm^2
$As_{use} > As_{need}$ = 1140,85 mm^2 > 738,56 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 104 mm > 25 mm (OK), use 3D22 reinforcement bar

As'_{need} = $0{,}5xAs_{use}$ = 570,42 mm^2
use n = 3 → As'_{use} = $3xAs_{bar}$ = 1140,86 mm^2
$As'_{use} > As'_{need}$ = 1140,86 mm^2 > 570,42 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 104 mm > 25 mm (OK), use 3D22 reinforcement bar

4.4.2.3. Reinforcement of Precast Primary Beam After Monolith Condition

<u>General Data</u>

The general data for reinforcement of primary beam will be shown below

b (width) = 40 cm
h (height, before monolith) = 65 cm
h (height,after monolith) = 65 cm + hf (floor) = 80 cm
L (axis to axis) = 5000mm

L (actual)	= 4200 mm
bar's diameter (D/deform)	= 22 mm
bar's diameter/stirrup(D/deform)	= 13 mm
f'_c (28 days)	= 30 MPa
f_y	= 390 MPa

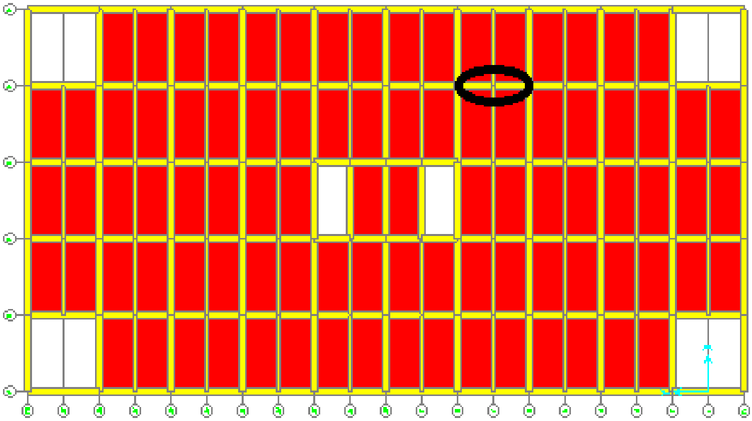


Figure 4.50. The location of precast primary beam on SAP 2000 v14®
(source: Private Documentation)

Load Calculation

The load calculation of precast primary beam will be assumed as trapezoid shape. The loads consist of deal load and live load. The load $q_{eqivalent}$ (q_{eq}) will be shown below

$q_{eq} = q \times L_x \times 1/2 \times ((1 - L_x^2/L_y^2) \times 1/3))$

After Monolith Condition

In this condition itself will be divided only intoone condition below

1. condition c

   when the *overtopping* concrete is already installed (already in monolith condition) and the load consists of *overtopping* concrete's load, precast slab element's load, precast secondary beam element's load, and live loads

1. dead load

secondary beam BAL1 : 0,3x0,60x2400 kg/m= 432 kg/m
q of slab LA1
(condition c) : 922,8 kg/m^2 = 922,8 kg/m^2

by using Ly = 500 cm – (40 cm/2 + 40 cm/2) = 460 cm and Lx = 250 cm – (40 cm/2 + 30 cm/2) = 215 cm, it is obtained the value of q_{eq} due to slab's load

(two trapezoids load)
$q_{eq} = 2 \times q \times 1/2 \times L_x \times ((1 - 1/3 \times (L_x/L_y)^2)$
q (condition c) = 922,8 kg/m^2
q_{eq} = 1839,55 kg/m → due to slab's load
DL (condition c) = 1839,55 kg/m + 432 kg/m
= 2271,55 kg/m
DL (k, shock coefficient = 1,2) = 1,2x2271,55 kg/m
= 2725,86 kg/m

DL (total, condition c) : = 2725,86 kg/m

2. live load

live load: 1,92 kN/m^2 = 192 kg/m^2

q_{eq} = 382,741 kg/m → due to live load

LL (total, condition c) : = 382,741 kg/m

3. dead load (primary beam)

primary beam BIL1 : 0,4x0,80x2400 kg/m= 768 kg/m
q of slab LA1
(condition c) : 922,8 kg/m^2 = 922,8 kg/m^2

by using L_y = 500 cm – (80 cm/2 + 80 cm/2) = 420 cm and L_x = 250 cm – (80 cm/2 + 30 cm/2) = 195 cm, it is obtained the value of q_{eq} due to slab's load

q_{eq} =q x 1/2 x L_x x ((1 -1/3 x $(L_x/L_y)^2$)

q (condition c) = 922,8 kg/m^2

q_{eq} = 835,081kg/m → due to slab's load

DL (condition c) = 835,081 kg/m + 768 kg/m

= 1603,081 kg/m

DL (k, shock coefficient = 1,2) = 1,2x1603,81 kg/m

= 1923,69kg/m

DL (total, condition c) : = 1923,69kg/m

4. live load (primary beam)

live load: = 192 kg/m^2

q_{eq} = 173,74 kg/m → due to live load

LL (total, condition c) : = 173,74 kg/m

Load Combination of Primary Beam

The combination of load for primary beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of primary beam
due to secondary beam
condition c
(precast's age > 30 days, *overtopping*'s age > 30 days)
Qu = 1,2×2725,86 + 1,6x382,741 = 3883,413 kg/m
Pu = Qu x L = 19417,07 kg

due to primary beam
condition c
(precast's age > 30 days, *overtopping*'s age > 30 days)
Qu = 1,2×1923,69 + 1,6x173,74 = 2586,435 kg/m.

Moment Calculation of Primary Beam

The moment calculation of primary beam will use moment equation

Mu = 1/8 x q x L^2+ ¼ x P x L, in which L is the length of axis to axis of primary beam

moment calculation of primary beam
condition c (q = 2586,435 kg/m, P = 19417,07 kg, L = 5 m)
Mu = 32353,94 kgm

The Mu values are taken as
condition c
Mu = 32353,94 kgm
So, the Mu value is 32353,94 kgm.

Therefore, the moment calculation of primary beam will use moment results from SAP 2000 v14®.
M (at left support area) = -357725068 Nmm
M (at right support area) = -336406667 Nmm
M (at field area) = +134026394 Nmm

Shear Force Calculation of Primary Beam

The shear force calculation of primary beam will use shear force equation Vu = 1/2 x q x L + ½ x P

shear force calculation of primary beam
(q = 2586,435 kg/m, P = 19417,07 kg, L = 5 m)
Vu =16174 kg

The Vu values are taken as
condition c
Vu =16174 kg
So, the Vu value is 16174 kg.

Therefore, the shear force results from SAP 2000 v14® will be shown below
Vu (left) = 267714,6 N, Vu (right) = 260399,58 N.

Calculation of Reinforcement Bar

The general data for calculation of reinforcement bar of primary beam is shown below
primary beam's dimension (overall) = 400mm × 800mm
clear cover = 50 mm
bar's diameter (D) = 22 mm
bar's diameter/stirrup(D/deform)= 13 mm
f'_c(28 days) = 30 MPa
f_y = 390 MPa
d' = clear cover + stirrup + D/2 = 74 mm
d = 800 - 50 - 13 - (22/2) = 726 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)
$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$
$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$
As = $\pi/4xd^2$ = 380,286 mm^2.

Reinforcement Bar (Flexural Bar)
At Support Area (Left)
(Assumed Primary Beam as Rectangular Shape)

The general data for reinforcement bar of primary beam will be shown below
Primary beam's dimension = 400x 800 mm^2
f'_c = 30 MPa (28 days)
clear cover = 50 mm
bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2
b = 400 mm
d = 726 mm
d' = 74 mm
$M_{u\ left}$ = -35772,5 kgm = -357725068Nmm

<u>Flexural Bar</u>
As (tension reinforcement) = 6D22
As = 2281,72 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 28,4 mm > 25 mm (OK)

As’ (compression reinforcement) = 6D22
As’ = 2281,72 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 28,4 mm > 25 mm (OK)

<u>Analyzing Double Reinforcement</u>

$$\rho = \frac{As}{b_w \times d} = 0{,}0078$$

$$\rho' = \frac{As'}{b_w \times d} = 0{,}0078$$

$$\rho - \rho' \geq \frac{0{,}85 \times f'c \times \beta_1 \times d'}{f_y \times d} \times \frac{600}{600 - f_y}$$

$0 \leq 0{,}0159$ (compression reinforcement not yet yield)

Thus,

$$A_s f_y = 0{,}85 f_c' \beta_1 c \cdot b + A_s' \left[600 \left(\frac{c - d'}{c} \right) - 0{,}85 f_c' \right]$$

by square equation, it is obtained the value of c which is 56,52 mm.

So,
a = β_1 x c
a = 47,19 mm

check the compression reinforcement’s condition

$$f_s' = 600 \left(\frac{c - d'}{c} \right) \leq f_y$$

f'_s = 185,49 MPa ≤ 390 MPa (compression reinforcement is in compressive condition)

Checking Nominal Moment

$$M_n = \left(As \times fy - A'_s \times f's\right) \times \left(d - \frac{a}{2}\right) + A's \times f's \times \left(d - d'\right)$$

M_n = 603712741 Nmm

$$\phi = 0{,}75 + 0{,}15\left[\left(1/\left(c/d\right)\right) - \left(5/3\right)\right] \leq 0{,}9$$

$$\phi = 2{,}67 \rightarrow \phi = 0{,}9$$

ϕM_n = 543341467 Nmm > M_u = 357725068 Nmm (OK)

At Support Area (Right)

(Assumed Primary Beam as Rectangular Shape)

The general data for reinforcement bar of primary beam will be shown below

Primary beam's dimension = 400x 800 mm^2

f'_c = 30 MPa (28 days)

clear cover = 50 mm

bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2

b = 400 mm

d = 726 mm

d' = 74 mm

$M_{u\ right}$ = -33640,6 kgm = -336406667 Nmm

Flexural Bar

As (tension reinforcement) = 6D22

As = 2281,72 mm^2

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 28,4 mm > 25 mm (OK)

As' (compression reinforcement) = 6D22

As' = 2281,72 mm^2

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 28,4 mm > 25 mm (OK)

Analyzing Double Reinforcement

$$\rho = \frac{As}{b_w \times d} = 0{,}0078$$

$$\rho' = \frac{As'}{b_w \times d} = 0{,}0078$$

$$\rho - \rho' \geq \frac{0{,}85 \times f'c \times \beta_1 \times d'}{f_y \times d} \times \frac{600}{600 - f_y}$$

$0 \leq 0{,}0159$ (compression reinforcement not yet yield)

Thus,

$$A_s f_y = 0{,}85 f_c' \beta_1 c \cdot b + A_s' \left[600 \left(\frac{c - d'}{c} \right) - 0{,}85 f_c' \right]$$

by square equation, it is obtained the value of c which is 56,52 mm.

So,

a = β_1 x c

a = 47,19 mm

check the compression reinforcement's condition

$$f_s' = 600 \left(\frac{c - d'}{c} \right) \leq f_y$$

f'_s = 185,49 MPa ≤ 390 MPa (compression reinforcement is in compressive condition)

Checking Nominal Moment

$$M_n = \left(As \times fy - A'_s \times f's \right) \times \left(d - \frac{a}{2} \right) + A's \times f's \times \left(d - d' \right)$$

M_n = 603712741 Nmm

$$\phi = 0{,}75 + 0{,}15\left[\left(1/(c/d)\right) - (5/3)\right] \le 0{,}9$$

$$\phi = 2{,}67 \rightarrow \phi = 0{,}9$$

ϕM_n = 543341467 Nmm > M_u = 336406667 Nmm (OK)

<u>At Field Area</u>

(Assumed Primary Beam as Dummy T Beam)

The general data for reinforcement bar of primary beam will be shown below

Primary beam's dimension = 400x 800 mm^2

f'_c = 30 MPa (28 days)

clear cover	= 50 mm
bar's diameter	= 22 mm, As = $\pi/4 x d^2$ = 380,28 mm^2
b	= 400 mm
d	= 726 mm
d'	= 74 mm
M_u = +13402,6 kgm	= +134026394Nmm

<u>Effective Width (b_e)</u>

be1 = ¼ x Lb = ¼ x 5000 = 1250 mm

be2 = bw + 16t_f = 400 + (16 x 150) = 2800 mm

be3 = ½ x (Lb – bw) = ½ x (5000 – 400) = 2300 mm

b = be = 1250 mm

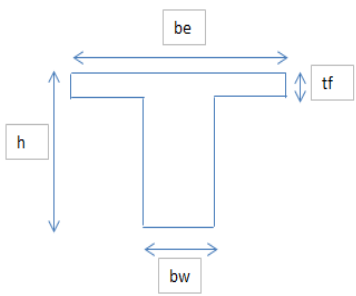


Figure 4.51. The dummy T beam dimension of primary beam (source: Private Documentation)

Flexural Bar
As (tension reinforcement) = 4D22
As = 1521,14 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 62 mm > 25 mm (OK)

As' (compression reinforcement) = 4D22
As' = 1521,14 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 62 mm > 25 mm (OK)

Analyzing Double Reinforcement

$$\rho = \frac{As}{b_w \times d} = 0{,}0052$$

$$\rho' = \frac{As'}{b_w \times d} = 0{,}0052$$

$$\rho - \rho' \geq \frac{0{,}85 \times f'c \times \beta_1 \times d'}{f_y \times d} \times \frac{600}{600 - f_y}$$

$0 \leq 0{,}0159$ (compression reinforcement not yet yield)
Thus,

$$A_s f_y = 0{,}85 f_c' \beta_1 c \cdot b + A_s' \left[600\left(\frac{c - d'}{c} \right) - 0{,}85 f_c' \right]$$

by square equation, it is obtained the value of c which is 51,46 mm.

So,
a = β_1 x c
a = 42,97 mm

check the compression reinforcement's condition

$$f_s' = 600\left(\frac{c - d'}{c} \right) \leq f_y$$

f'_s = 262,6 MPa ≤ 390 MPa (compression reinforcement is in tension condition)

Checking Nominal Moment

$$M_n = \left(As \times fy - A'_s \times f's\right) \times \left(d - \frac{a}{2}\right) + A's \times f's \times \left(d - d'\right)$$

M_n = 396966886 Nmm

$$\phi = 0{,}75 + 0{,}15\left[\left(1/\left(c/d\right)\right) - \left(5/3\right)\right] \leq 0{,}9$$

$$\phi = 2{,}86 \rightarrow \phi = 0{,}9$$

ϕM_n = 357270197 Nmm > M_u =134026394 Nmm (OK)

Checking Dummy T Beam

T = A_s x f_y
= 1521,14 mm^2 x 390 MPa
= 593245,71 N

C = 0,85 × f'c × b_e× h_f
= 0,85 x 30 MPa x 1250 mm x 150 mm
= 4781250 N

therefore C > T, then the beam is assumes as dummy T beam which treated like rectangular beam with b_e as its width,

$$a = \frac{As \times fy}{0{,}85 \times bw \times f'c} < t$$

a = 58,16 mm < t = 150 mm (OK)

$$\phi Mn = \phi \times \mathrm{As} \times \mathrm{fy} \times \left(\mathrm{d} - \frac{\mathrm{a}}{2}\right)$$

$ØM_n$> Mu
Ø → 0,9
372099964 Nmm > 134026394 Nmm (OK).

Recapitulation of the reinforcement will be shown below

- at left support
  upper reinforcement = 6D22 (As = 2281,72mm^2)
  lower reinforcement = 6D22 (As' = 2281,72mm^2)
- at right support
  upper reinforcement = 6D22 (As = 2281,72mm^2)
  lower reinforcement = 6D22 (As' = 2281,72mm^2)
- at field area
  upper reinforcement = 4D22 (As' = 1521,14 mm^2)
  lower reinforcement = 4D22 (As = 1521,14 mm^2).

Reinforcement Bar (Shear and Torsion)
Reinforcement Bar Due to Shear Force

The reinforcement bar due to shear force shall follow the conditions in SNI 2847:2013 paragraph 21.3.3. The equation for calculating shear force is shown below

$$V_u = \frac{M_{pr1} + M_{pr2}}{l_n} \pm \left(\frac{Qu \times l_n}{2} + \frac{Nu}{2} \right)$$

in which,

$$Mpr = As \times 1{,}25 \times fy \times \left(d - \frac{a}{2} \right)$$

$$a = \frac{As \times 1{,}25 \times fy}{0{,}85 \times f'c \times b}$$

the result of M_{pr} value will be shown in Table 4.30.

Table 4.30. Recapitulation of M_{pr} Value

Location			n bar	As (mm^2)	a (mm)	M_{pr} (Nmm)
At support area	Left	Upper	6	2281,72	109,05	746904222
		Lower	6	2281,72	109,05	746904222
	Right	Upper	6	2281,72	109,05	746904222
		Lower	6	2281,72	109,05	746904222

M_{pr1} = 746904222 Nmm
M_{pr2} = 746904222 Nmm
L_n = 5000 mm – (2x0,5x800) mm = 4200 mm.

<u>Analyze Vu</u>
analyzing left quake
$V_{u1} = M_{pr1} + M_{pr2}/(L_n) – (Q_u \times L_n/2) – (N_u/2)$
$V_{u2} = M_{pr1} + M_{pr2}/(L_n) + (Q_u \times L_n/2) + (N_u/2)$

in which,
Q_u = 2586,435 kg/m (see previously paragraph)
N_{uleft} = 38906,85 N (from SAP 2000 v14®)

thus,
V_{u1} = 281900,27N
V_{u2} = 429437,08 N

analyzing right quake
$V_{u1} = M_{pr1} + M_{pr2}/(L_n) – (Q_u \times L_n/2) – (N_u/2)$
$V_{u2} = M_{pr1} + M_{pr2}/(L_n) + (Q_u \times L_n/2) + (N_u/2)$

in which,
Q_u = 2586,435 kg/m (see previously paragraph)
N_{uright} = 38362,87 N (from SAP 2000 v14®)

thus,
V_{u1} = 282172,11N
V_{u2} = 429165,25 N

the value of V_u will be compared to V_u result from SAP 2000 v14®.

Table 4.31. Recapitulation of V_u Value

<table>
<tr><td colspan="2">Location</td><td>V_u
(N)</td><td>V_u
(SAP200 v14®)
(N)</td></tr>
<tr><td rowspan="4">At support area</td><td rowspan="2">Left</td><td>281900,27</td><td rowspan="2">267714,64</td></tr>
<tr><td>429437,08</td></tr>
<tr><td rowspan="2">Right</td><td>282172,11</td><td rowspan="2">260399,58</td></tr>
<tr><td>429165,25</td></tr>
</table>

From Table 4.31 above, the Vu value which will be used for calculation is 429437,08 N.

Reinforcement Bar In Plastis Area
checking condition:

1. $(M_{pr1} + M_{pr2})/L_n \geq 0,5 \times Vu$
   $= (746904222 + 746904222)/4200$ N $\geq 0,5 \times 429437,08$ N
   $= 355668,68$ N $\geq 214582,6$ N (OK)

2. $Nu \leq 0,25 \times A_g \times f'_c$
   = 38906,54 N (from SAP 2000 v14®) $\leq$ 2400000 N (OK).

According to SNI 2847:2013 paragraph 21.5.4.2, the value of Vc is assumed as 0 if both conditions above fulfilled.

Thus,
$Vs = Vu/\phi$
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 429437,08 N

Vs = 572582,78 N

use D13 reinforcement bar (Av = 265,71 mm^2),

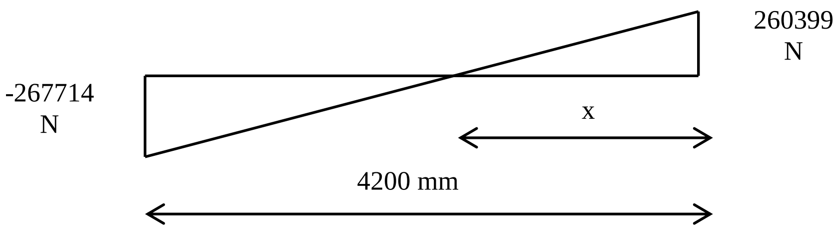
s = Av x fy x d/Vs
s = 265,71 mm^2 x 390 MPa x 726 mm/572582,78 N
s = 131,33 mm ≈ 130 mm.

According to SNI 2847:2013 paragraph 21.3.4.2, the stirrup bar should be installed along 2h = 1600 mm at both edge of beam with the space of stirrup bar must not more than the least of

1. d/4 = 726 mm/4 = 181,75 mm
2. 8D = 8 x 22 mm = 176 mm
3. 24 x ϕ = 24 x 13 mm = 312 mm
4. 300 mm.

Use stirrup bar D13-120, s (space) = 120 mm ≤ 176 mm

<u>Reinforcement Bar Outside Plastis Area</u>
checking Vu value (from SAP 2000 v14®):



Equation for finding x value

$$\frac{x}{4200 - x} = \frac{260399}{267714}$$

$$x = 2071\,\text{mm}$$

Vu at 2h = 1600 mm, is

$$\frac{2071}{2071-1600}=\frac{260399}{Vu}$$

$$Vu = 59213 \text{ N}$$

Vu = 59213 N at 1600 mm from support.

Thus,
Vs = Vu/ϕ
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 59213 N
Vs = 78951 N

use D13 reinforcement bar (Av = 265,71 mm^2),

s = Av x fy x d/Vs
s = 265,71 mm^2 x 390 MPa x 726 mm/78951 N
s = 952,41 mm ≈ 950 mm

According to SNI 2847:2013 paragraph 21.3.4.3, the stirrup bar should be installed along the beam (outside plastis area) with the space of stirrup bar must not more than

1. d/2 = 726 mm/2 = 363 mm.

Use stirrup bar D13-300, s (space) = 300 mm ≤ 363 mm.

n of stirrup bar (outside plastis area) = (L_n – 4h)/s + 1
n = (4200 mm – 4 x 800 mm)/300 mm + 1
n = 4,33≈ 5.

<u>Reinforcement Bar Due to Torsion Moment</u>

The reinforcement bar due to torsion moment shall follow the conditions in SNI 2847:2013 paragraph 11.5.1. The torsion can be neglected if the condition below fulfilled

$$T_u < \phi \frac{\sqrt{f'_c}}{12}\left(\frac{A^2{}_{cp}}{P_{cp}}\right)$$

in which,
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Tu = 16708238,3 Nmm (from SAP 2000 v14®)
A_{cp} = b x h = 320000 mm^2
P_{cp} = 2 x (b + h) = 2400 mm

Thus,
Tu = 16698745,4 Nmm ≥ 14605934,9 Nmm (not OK).
So, the torsion reinforcement bar is needed.

Longitudinal Reinforcement
The design of torsion reinforcement shall follow this condition
$\phi T_n \geq T_u$ (SNI, 2013 paragraph 11.5.3.5)
$T_n = 2A_0A_tf_{yt}\cot\theta/s$ (SNI, 2013 paragraph 11.5.3.6)
in which,
$A_0 = 0{,}85A_{h0}$ = 0,85 x (287 x 687) mm^2
A_0 = 167593,65 mm^2
$\theta = 45^0$ (non-prestress)
f_{yt} = 390 MPa
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)

then,
$A_t/s = (T_u/\phi)/(2A_0f_{yt}\cot 45^0)$
A_t/s = 0,17 mm^2/mm

according to SNI 2847:2013 paragraph 11.5.3.7, the area of torsion reinforcement should be calculated based on this equation
$A_\ell = (A_t/s) \times P_h \times f_{yt}/f_y$

in which,
P_h = 2 x (287 + 687) mm = 1948 mm

f_y = 390 MPa

then,
$A_\ell = (A_t/s) \times P_h \times f_{yt}/f_y$
A_ℓ = 331,78 mm^2

use torsion reinforcement bar 2D22 (A_ℓ = 760,57 mm^2).

checking the condition of torsion reinforcement bar according to SNI 2847:2013 paragraph 11.5.5.3

$$A_{st} + A_l \geq \frac{0{,}42\sqrt{f_c'}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right)P_h \frac{f_{yt}}{f_y}$$

in which,
A_{st} = 6D22 (2281,72mm^2)
A_ℓ = 2D22 (760,57 mm^2)
A_{cp} = 400 x 800 mm^2 = 320000 mm^2

then,
$A_{st} + A_\ell$= 3042,29 mm^2 ≥ 1555,75 mm^2 (OK)

<u>Transversal Reinforcement</u>
The need of stirrup reinforcement at plastis area:

$$\frac{A_v}{s} = \frac{V_S}{f_y d} = 2{,}022\, mm^2 / mm$$

checking stirrup (D13-120):

$$\frac{n\,\pi/4\,D^2}{s} \geq \frac{A_v}{s} + \frac{A_t}{s}$$

2,213 mm^2/mm ≥ 2,192 mm^2/mm (OK)

The need of stirrup reinforcement outside plastis area:

$$\frac{A_v}{s} = \frac{V_S}{f_y d} = 0{,}278\, mm^2 / mm$$

checking stirrup (D13-300):

$$\frac{n\pi/4\, D^2}{s} \geq \frac{A_v}{s} + \frac{A_t}{s}$$

0,885 mm^2/mm ≥ 0,449 mm^2/mm (OK).

Checking Condition of Beam (Due to Lifting Process)

When lifting process occurs, the need of reinforcement bar due to lifting process, is necessary. Because when lifting process occurs, it will produce moment force. So, the moment force must be calculated and then the reinforcement bar designed so it can bear the moment force due to lifting process. In this chapter will use two lifting points of primary beam (Figure 4.52).

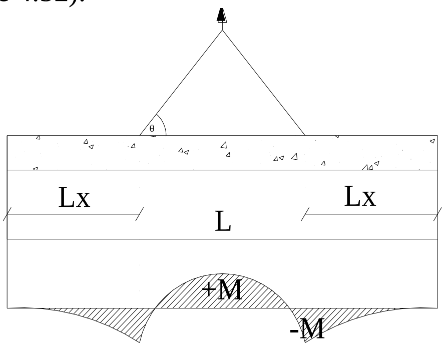


Figure 4.52. Two lifting points of primary beam and the moment obtained due to lifting process of primary beam (source: Private Documentation)

The general data for calculating lifting bar will be shown below

f'$_c$ (14 days) = 26,4 MPa
f_y = 240 MPa = 2400 kg/cm^2
primary beam's type = BIL1 (b = 300 x h = 650)
L (length) = 4200 mm
n (number of points) = 2
k (shock coefficient) = 1,2
W = 300x650x4200x2400 = DL = 2620,8 kg
Wt = 300x650x2400 = 624 kg/m
Qu (=1,4 DL x k) = 4402,94 kg
P (=Qu/n) = Tu = 2201,47 kg

Designing The Lifting Bar

The designing process of lifting bar will use equation below

$$\phi_{\text{tulangamamgkat}} \geq \sqrt{\frac{n \times P}{\pi \times \sigma}}$$

in which n is the number of liting points and σ is f_y/1,5 = 1600 kg/cm^2 (SF = 1,5, f_y = 240 MPa).

So, the value of ϕ (diameter of lifting bar) is

$$\phi = \sqrt{\frac{2\ x\ 2201{,}47\ kg}{\frac{22}{7}\ x\ 1600\ kg/cm^2}} = 0{,}93\ cm$$

use lifting bar ϕ = 12 mm instead (As = 113,14 mm^2).

Moment Calculation Due to Lifting Process

The moment calculation due to lifting process of beam will use these equations below

$+\mathrm{M} = \frac{\mathrm{WtL}^2}{8}\left(1 - 4\mathrm{x} + \frac{4\mathrm{yc}}{\mathrm{Ltan\theta}}\right) \rightarrow$ moment at field area

$-\text{M} = \frac{\text{WtX}^2\text{L}^2}{2}$ → moment at support area

$$\text{X} = \frac{1 + \frac{4\text{yc}}{\text{Ltan}\theta}}{2\left(1 + \sqrt{1 + \frac{\text{yt}}{\text{yb}}\left(1 + \frac{4\text{yc}}{\text{Ltan}\theta}\right)}\right)}$$

$y_t = y_b = ½ \times h$	= 32,5 cm
$y_c = y_t + 1"$	= 35,04 cm
rope's height	= 1500mm
θ^o	= $\tan^{-1}$ ((rope's height/(L – 2LX))
	= $55,99^o$
$\tan \theta^o$	= 1,483
X	= 0,276 m → 0,28 m
LX	= 1,273 m → 1,288 m
L – 2LX	= 2,052 m → 2,024 m
T (cable force)	= P x sin θ^o = 1037,83 kg
+M	= 173,97 kgm
−M	= 322,49 kgm
M' (negative moment)	= P x y_c/tan θ^o = 211,318kgm
(moment force due to lifting process)	
−M (total)	= 533,81 kgm
Z	= 1/6 x b x h^2 = 28166 cm^3

<u>Checking The Crack Factor</u>

The lifting bar will use ϕ = 12 mm plain bar.
Checking the f_{cr} value (assumed the concrete's age is 14 days)

$f_{cr} = 0,62\ \lambda\sqrt{f'c}$ /SF (SNI, 2013 paragraph 9.5.2.3)
(λ = 1, SNI, 2013 paragraph 8.6.1)
(safety factor = 2)

f_{cr}	= 1,593 MPa
f (due to field moment)	= +M(total)/Z = 0,172 MPa
f (due to support moment)	= −M(total)/Z = 0,527 MPa
$f < f_{cr}$	= 0,172 MPa < 1,593 MPa (OK)
$f < f_{cr}$	= 0,527 MPa < 1,593 MPa (OK)

<u>Checking The Strand Cable</u>

The strand cable for lifting process will use seven-wire strand 5/16 inch diameter with f_{pu} = 250 ksi (see PCI, 2004 Design Aid Table 11.2.3).

f_{pu}	= 250 ksi
A	= 0,058 in^2
f_{pu} x A	= 14,5 kips = 64499,2132 N
SF	= 2
Af_{pu}/SF	= 3224,96kg
T	= 1037,83 kg
T < Af_{pu}/SF	= 1037,83 kg < 3224,96 kg (OK)

<u>Checking The Deflection of Primary Beam</u>

The deflection of primary beam is calculated using equation below

δ (deflection) = (5/384) x (q_u x L^4/EI)

$\delta_{max} = L_n/480$ (in cm) (SNI 2847:2002 Paragraph 9)

in which q_u is taken as 3883,14 kg/m + 2586,435 kg/m = 6469,575 kg/m (see previous paragraph), I is the moment of inertia of secondary beam (I = $1/12xbxh^3$), and E equals $4700x(f'c)^{0,5}$, f'c in 14 days

Deflection (L_n = 420 cm, b = 40 cm, h = 80 cm)

δ = (5/384) x (q_u x L^4/EI) = 0,063 cm

$\delta_{max} = L_n/480$ = 0,875 cm

$\delta<\delta_{max}$ (OK)

<u>Checking The Crack Moment</u>

According to SNI 2847:2013 paragraph 21.3.3, the crack moment, M_{cr}, must not more than nominal moment, ϕM_n, and the M_{cr} value shall follow the the equation in SNI 2847:2013 paragraph 9.5.3.2.

$\phi M_n \geq M_{cr}$

in which,

$M_{cr} = f_{cr}$ x I_g/y_t

note that f_{cr} = 1,593 MPa,
$I_g = 1/12 \times b \times h^3$
$I_g = 1/12 \times 400 \times 650^3\ mm^4 = 9154166667\ mm^4$
y_t = 400 mm

Thus,
M_{cr} = 77716125 Nmm
ϕM_n = 357270197 Nmm
$\phi M_n \geq M_{cr}$ (OK).

The recapitulation of reinforcement bar of precast primary beam will be shown below

Table 4.32. Recapitulation of Reinforcement Bars of Precast Primary Beam

Before Monolith Condition										
Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar		Torsion
		As use (mm^2)	n	Code	As use (mm^2)	N	Code	s (mm)	Code	
BIL1	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIL2	1700	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIL3	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA1	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA2	1700	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA3	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	

After Monolith Condition										
Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar		Torsion
		As use (mm^2)	n	Code	As use (mm^2)	N	Code	s (mm)	Code	
BIL1	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIL2	1700	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIL3	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA1	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA2	1700	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA3	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	

Due to Lifting Process' Moment										
Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar		Torsion
		As use (mm^2)	n	Code	As use (mm^2)	N	Code	s (mm)	Code	
BIL1	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIL2	1700	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIL3	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA1	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA2	1700	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	
BIA3	4200	2281,72	6	6D22	1521,14	4	4D22	120	D13-120	2D22
		2281,72	6	6D22	1521,14	4	4D22	120	D13-120	

4.4.3. Reinforcement of Pillar

4.4.3.1. General Data

The general data for reinforcement of pillar will be shown below

b (width)	= 80 cm
h = b(rectangular pillar)	= 80 cm
L(axis to axis)	= 4000mm
L(actual)	= 3200 mm
bar's diameter (D/deform)	= 25 mm
bar's diameter/stirrup (D/deform)	= 19 mm
clear cover	= 50
f'_c	= 30 MPa

f_y =390 MPa

4.4.3.2. Checking Pillar's Dimension

According to SNI 2847:2013 paragraph 21.6.1, the condition for checking pillar's dimension is

Axial force (from SAP 2000 v14®) $\geq$ Ag x f'_c/10
6765773 N $\geq$ 800 mm x 800 mm x 30 MPa/10
6765773 N $\geq$ 1920000 N (OK)

then,
ratio of b/h > 0,4
800 mm/800 mm = 1 > 0,4 (OK).

4.4.3.3. Calculation of Pillar's Longitudinal Reinforcement

Before calculate the pillar's reinforcement, the result of axial force, etc. will be shown in Table 4.33.

Table 4.33. Recapitulation of Forces of Pillar (from SAP 2000 v14®)

Dimension	Axial	Shear	Torsion	Moment
mm x mm	N	N	Nmm	Nmm
800 x 800	6765773	86590	89077	210258344

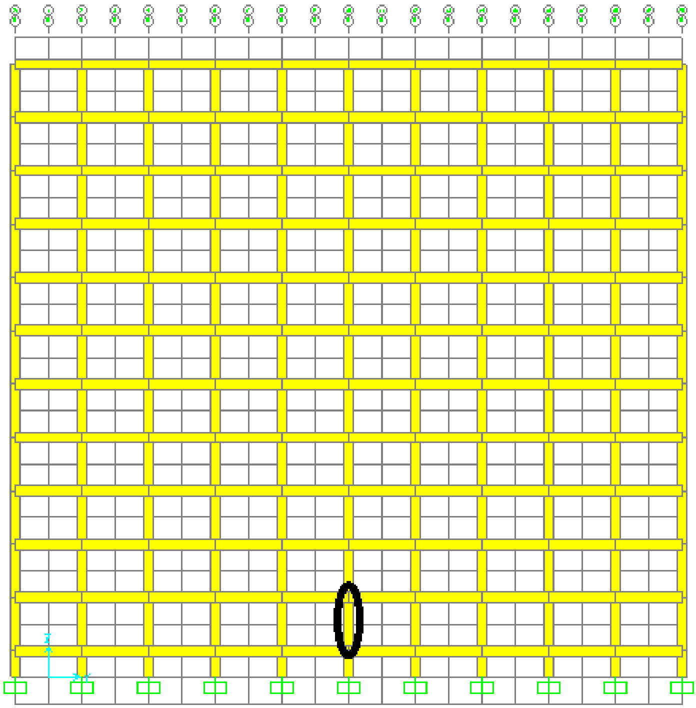


Figure 4.53. The location of pillar in SAP 2000 v14®
(source: Private Documentation)

According to SNI 2847:2013 paragraph 21.6.1, if the condition for checking pillar's dimension is fulfilled (see paragraph 4.4.5.2) then the reinforcement of pillar must follow the conditions stated in SNI 2847:2013 paragraph 21.6.4, 21.6.5, 21.7.3. The calculation of pillar's reinforcement will use SpColumn® to calculate longitudinal reinforcement bar. The graphic's result of interaction between axial force and moment in pillar will be shown below

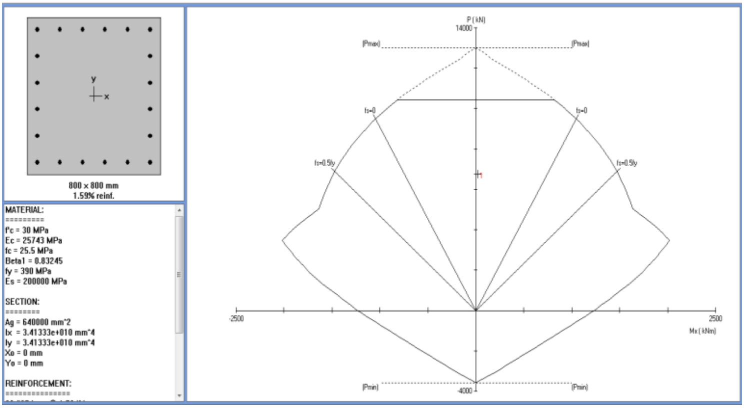


Figure 4.54. The interaction graphic between axial force and moment in pillar ($f_s = f_y$)
(source: Private Documentation)

4.4.3.4. Checking Ratio of Longitudinal Reinforcement

According to SNI 2847:2013 paragraph 21.6.3.1, the area of longitudal reinforcement bar, A_{st}, must not less than $0{,}01A_g$and must not greater than $0{,}06A_g$. From Figure 4.60, the percentage of A_{st} is $0{,}0159A_g$ (1,59% x A_g) with 20D25. So, the ratio of longitudinal reinforcement is fulfilled.

4.4.3.5. Checking Capacity of Nominal Axial Force of Pillar

According to SNI 2847:2013 paragraph 10.3.6.2, the capacity of nominal axial force's pillar must not less than axial force's pillar from computer programme (SAP 2000 v14®)

$\phi P_{n\ max} \geq P_u$
ϕ = 0,65 (from SpColumn®)
$\phi P_{n\ max} = 0{,}80 \times \phi \times [0{,}85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st}] \geq P_u$

in which,
$A_g = b \times h = 800 \times 800\ mm^2 = 640000\ mm^2$
$A_{st} = 1{,}59\% \times A_g = 9821{,}43\ mm^2$

then,
$\phi P_{n\ max} = 10347953{,}6\ N \geq P_u = 6765773N$ (OK).

<u>Checking Condition of Strong Column Weak Beam</u>

According to SNI 2847:2013 paragraph 21.6.2, the condition of strong column weak beam is
$\sum M_{nc} \geq 1{,}2 \times \sum M_{nb}$

in which,
$M_{nc} = \phi M_n/\phi$
ϕM_n = 1744,76 kNm (from SpColumn®)
ϕ = 0,65 (from SpColumn®)
M_{nb}^+ = 603712741 Nmm (nominal moment of primary beam)
M_{nb}^- = 603712741 Nmm (nominal moment of primary beam)

then,
$\sum M_{nc} = 2 \times \phi M_n/\phi$
$\sum M_{nc}$ = 5460800000Nmm
$1{,}2 \times \sum M_{nb} = 1{,}2 \times 0{,}85 \times (603712741 + 603712741)$ Nmm
$1{,}2 \times \sum M_{nb}$ = 1448910578 Nmm
$\sum M_{nc} = 5460800000\ Nmm \geq 1{,}2 \times \sum M_{nb} = 1448910578\ Nmm$ (OK).

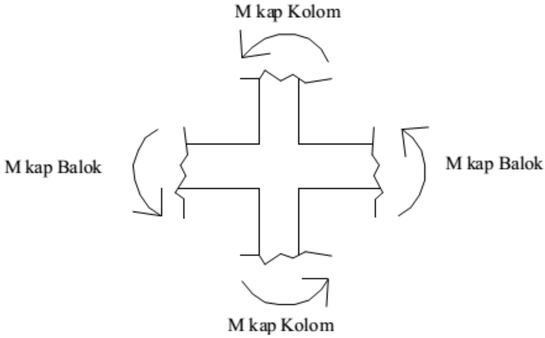


Figure 4.55. The connection between beams and pillar (source: Private Documentation)

4.4.3.6. Checking Condition of Pillar to Designed Shear Force (V_e)

According to SNI 2847:2013 paragraph 21.6.5.1, the value of V_e(shear force in pillar) is

$V_e = 2 \text{ x } M_{pr}/L_n$

in which,

$M_{pr} = \phi M_n/\phi$

ϕM_n = 1765,84kNm (from SpColumn®)

ϕ = 0,65 (from SpColumn®)

L_n = 3200 mm

note that the value of M_{pr} of column is based on assumption that $f_s \geq 1{,}25\ f_y$ = 1,25 x 390 MPa = 487,5 MPa.

then,

V_e = (2 x 1765,84 kNm/0,65)/3200 mm = 1697923,08 N

The value of V_u (shear force in beam) is

$V_u = 2 \text{ x } (M_{pr}^{+} + M_{pr}^{-})/L_1 \text{ x } [L_1/(L_1 + L_2)]$

in which,

M_{pr}^{+} = 746904222 Nmm (probable moment of primary beam)
M_{pr}^{-} = 7469042220 Nmm (probable moment of primary beam)
L_1 = 4000 mm (pillar's height, first floor)
L_2 = 4000 mm (pillar's height, second floor)

then,
V_u = 2 x (746904222 Nmm + 746904222Nmm)/4000 mm x [4000 mm/(400 mm + 4000 mm)]
V_u = 373452,11 N

V_e = 1697923,08 N ≥ V_u = 373452,11 N (OK)

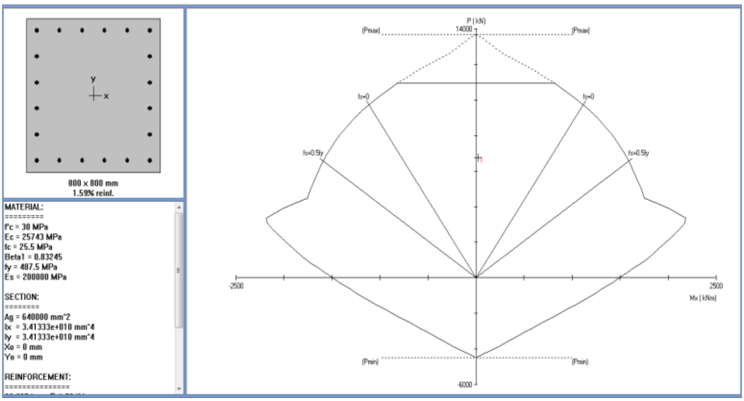


Figure 4.56. The interaction graphic between axial force and moment in pillar (f_s = 1,25f_y)
(source: Private Documentation)

4.4.3.7. Calculation of Pillar's Confinement Reinforcement

According to SNI 2847:2013 paragraph 21.6.4.1, the value of ℓ_0 must not less than the greatest value below

1. h = 800 mm
2. 1/6 x L_n = 1/6 x 3200 mm = 533,33 mm
3. 450 mm

whilst according to SNI 2847:2013 paragraph 21.6.4.3, the value of s_0 must not more than the least value below

1. ¼ x b = ¼ x 800 mm = 200 mm
2. 6D = 6 x 25 mm = 150 mm
3. $s_0 = 100 + (350 - h_x)/3$

   h_x = 0,5 x (b – 2 x (clear cover + stirrup/2))

   h_x = 0,5 x (800 – 2 x (50 + 19/2))

   h_x = 340,5 mm

   s_0 = 100 + (350 – 340,5)/3 = 103,2 mm

in which s_0 value is not more than 150 mm and not less than 100 mm.
use s = 100 mm (2D19 – 100).

According to SNI 2847:2013 paragraph 21.6.4.4, the value of A_{sh} must not less than the greatest value below

1. A_{sh} = 0,3 x s x b_c x f'_c/f_{yt} x ((A_g/A_{ch}) - 1)
2. A_{sh} = 0,09 x s x b_c x f'_c/f_{yt}

in which,
b_c = b – 2D – clear cover
b_c = 800 – 2 x 19 – 50 mm = 712 mm
A_g = 800 x 800 mm^2 = 640000 mm^2
A_{ch} = (800 – 50)^2 mm^2 = 562500 mm^2

then,

1. A_{sh} = 147,14 mm^2
2. A_{sh} = 320,4 mm^2

Use A_{sh} = 320,4 mm^2.

then use confinement reinforcement bar 2D19 (A_s = 567,28 mm^2 > A_{sh} = 320,4 mm^2).

According to SNI 2847:2013 paragraph 11.2.1.2, the value of V_c is

$$V_c = 0{,}17\left(1 + \frac{N_u}{14A_g}\right)\lambda\sqrt{f_c'}b_w d$$

in which,
N_u = 6765773 N (from SAP 2000 v14®)
A_g = 640000 mm^2
λ = 1 (SNI, 2013 paragraph 8.6.1)
d = b – clear cover – stirrup – ½ D
d = 800 – 50 – 19 – 0,5 x 25 mm = 718,5 mm
V_c = 757333,17N

the value of V_s is

$$V_s = \frac{A_s \times f_y \times d}{s}$$

in which,
A_s = 567,28 mm^2
d = 718,5 mm
s = 100 mm
V_s = 1589619,66 N

checking the condition
$\phi V_n \geq V_u$
$V_n = V_c + V_s$
V_n = 757333,17 N + 1589619,66 N = 2346952,83 N
V_u = 86590N (from SAP 2000 v14®)
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
ϕV_n = 1760214,63 N $\geq$ V_u = 86590N (OK)

According to SNI 2847:2013 paragraph 21.5.3.2, the space of reinforcement bar must not greater than the least of

1. ¼ x d = ¼ x 718,5 mm = 179,625 mm
2. 6D = 6 x 25 mm = 150 mm
3. 150 mm

use s = 150 mm (2D19 – 150).

Calculation of Pillar's Development Bar

According to SNI 2847:2013 paragraph 12.2.2, the length of development bar which is located in the middle of pillar for pillar's connection must fulfill this equation below

$$l_d = \left(\frac{f_y}{1{,}1\lambda\sqrt{f_c'}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) \times d_b$$

in which,
Ψ_t = 1, Ψ_t = 1, Ψ_t =1 (SNI, 2013 paragraph 12.2.4)
λ = 1 (SNI, 2013 paragraph 12.2.4)
K_{tr} = 0 (SNI, 2013 paragraph 12.2.3)
c_b = clear cover + D (stirrup) + ½ x D_b
c_b = 50 mm + 19 mm + ½ x 25 mm = 81,5 mm
ℓ_d = 615,7 mm ≈ 500 mm

According to SNI 2847:2013 paragraph 12.7.2, the value of ℓ_d must not less than 200 mm

ℓ_d = 615,7 mm ≈ 500 mm ≥ 200 mm (OK)

use ℓ_d = 650 mm.

The recapitulation of reinforcement bar pillar will be shown below

Table 4.34. Recapitulation of Pillar's Reinforcement

Pillar's Type	Dimension (mm^2)	Longitudinal Reinforcement	Confinement Reinforcement	
			along l_o	outside l_o
K1	800 x 800	20D25	2D19-100	2D19-150
K2	800 x 800	16D25	2D19-100	2D19-150

4.4.4. Reinforcement of Shear Wall

4.4.4.1. General Data

The general data for reinforcement of shear wall will be shown below

section cut of shear wall (SCUT1, from SAP 2000 v14®)

t (thickness)	= 40 cm
L(length)	= 5000mm
H(total height)	= 44m
bar's diameter (D/deform)	= 22 mm
clear cover	= 40 mm
f'_c	= 30 MPa
f_y	= 390 MPa

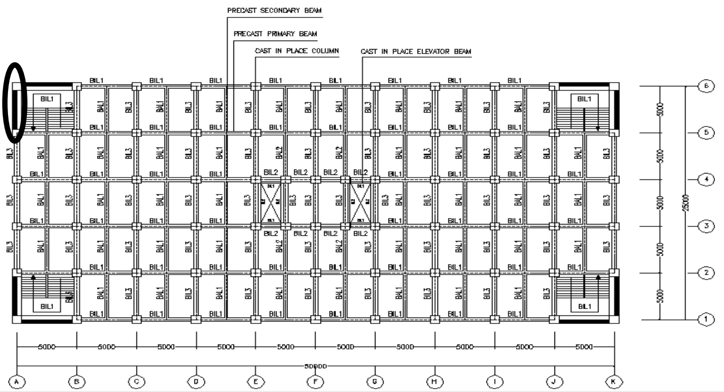


Figure 4.57. The location of shearwall
(source: Private Documentation)

4.4.4.2. Checking The Shear Force of Shear Wall

The result of shear wall's shear force will be show below

Table 4.35. Recapitulation of Shear Wall's Forces (from SAP 2000 v14®)

Combination	Axial	Shear	Moment
Unitless	N	N	Nmm
1,2 DL + 1,0 LL + 1,0 E_x	258602,78	816424,52	2906581896
1,2 DL + 1,0 LL + 1,0 E_y	862009,28	244927,36	9688606319

4.4.4.3. Checking Planned Axial Force

According to SNI 2847:2013 paragraph 14.5.2, the planned axial force should follow this equation

$$\phi P_n = 0{,}55\phi f'c.Ag\left[1-\left(\frac{k.\ell_c}{32h}\right)^2\right]$$

in which,
A_g = 5000 mm x 400 mm = 2000000 mm^2
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
ℓ_c = 4000 mm (pillar's length)
k = 0,8 (SNI, 2013 paragraph 14.5.2)
h = 400 mm (shear wall's thickness)

then,
for X direction
ϕP_n = 232031250N $\geq$ P_u = 258602,78 N (OK)

for Y direction
ϕP_n = 232031250N $\geq$ P_u = 862009,28 N (OK).

4.4.4.4. Checking Shear Wall's Thickness

According to SNI 2847:2013 paragraph 11.9.3, the thickness of shear wall should follow this condition

$$\phi V_n = \phi 0{,}83\sqrt{f'c}.h.d \geq V_u$$

in which,
ℓ_w = 5000 mm (shear wall’s length)
d = 0,8ℓ_w (SNI, 2013 paragraph 11.9.4)
d = 0,8 x 5000 mm = 4000 mm
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
h = 400 mm (shear wall’s thickness)

then,
for X direction
ϕV_n = 54553166,7N ≥ V_u = 816424,52 N (OK)

for Y direction
ϕV_n = 54553166,7N ≥ V_u = 244927,36N (OK).

4.4.4.5. Calculation Concrete’s Shear Force

According to SNI 2847:2013 paragraph 11.9.6, the shear force which is given by concrete is

$$Vc = 0{,}27\lambda\sqrt{f'c} \times h \times d + \frac{Nu \times d}{4 \times \ell_w}$$

in which,
ℓ_w = 5000 mm (shear wall’s length)
d = 0,8ℓ_w (SNI, 2013 paragraph 11.9.4)
d = 0,8 x 5000 mm = 4000 mm
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
h = 400 mm (shear wall’s thickness)
λ = 1 (SNI, 2013 paragraph 8.6.1)

then,
for X direction
N_u = 244927,36N(from SAP 2000 v14®)
V_c= 23824899,39N

then,
for Y direction
N_u = 244927,36N(from SAP 2000 v14®)
V_c = 23710599,96 N

4.4.4.6. Reinforcement of Shear Wall

According to SNI 2847:2013 paragraph 21.9.2.2, the reinforcement of shear wall should be two layers reinforcement if
$V_u \geq 0{,}17 \times A_{cv} \times f'c^{0,5} \times \lambda$

in which,
A_{cv} = 5000 mm x 400 mm = 2000000 mm^2
λ = 1 (SNI, 2013 paragraph 8.6.1)

then,
for X direction
V_u = 816424,52 N (from SAP 2000 v14®)
$0{,}17 \times A_{cv} \times f'c^{0,5} \times \lambda$ = 1862256,69 N
$V_u \leq 0{,}17 \times A_{cv} \times f'c^{0,5} \times \lambda$ (OK)

then,
for Y direction
V_u = 244927,36N (from SAP 2000 v14®)
$0{,}17 \times A_{cv} \times f'c^{0,5} \times \lambda$ = 1862256,69 N
$V_u \leq 0{,}17 \times A_{cv} \times f'c^{0,5} \times \lambda$ (OK)

According to SNI 2847:2013 paragraph 11.9.9, the ratio between horizontal shear reinforcement bar and gross area of concrete should not less than 0,0025

for X direction
use D22 reinforcement bar (A_s = 380,28 mm^2)

the space of reinforcement bar should not more than the least of

1. $\ell_w/5$ = 5000/5 mm = 1000 mm
2. 3h = 3 x 400 mm = 1200 mm
3. 450 mm

use s = 100 mm

$$\rho_t = \frac{A_s}{h \times s}$$

ρ_t = 0,0095> $\rho_{t\ min}$ = 0,0025 (OK)

the value of V_s is

$$V_s = \frac{A_v \times f_y \times d}{s}$$

in which,
ℓ_w = 5000 mm (shear wall's length)
d = 0,8ℓ_w (SNI, 2013 paragraph 11.9.4)
d = 0,8 x 5000 mm = 4000 mm
A_v = 760,57 mm^2
s = 100 mm
V_s = 11864914,29 N

checking the condition
$\phi V_n \geq V_u$
$V_n = V_c + V_s$
V_n = 23824899,39 N + 11864914,29 N = 35689813,67 N
V_u = 816424,52 N(from SAP 2000 v14®)
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)

ϕV_n = 26767360,26 N $\geq$ V_u = 816424,52 N (OK)

use D22 – 100 with s = 100 mm.

for Y direction

use D22 reinforcement bar (A_s = 380,28 mm^2)
the space of reinforcement bar should not more than the least of

1. ℓ_w/5 = 5000/5 mm = 1000 mm
2. 3h = 3 x 400 mm = 1200 mm
3. 450 mm

use s = 100 mm

$$\rho_t = \frac{A_s}{h \times s}$$

ρ_t = 0,0095> $\rho_{t\,min}$ = 0,0025 (OK)

the value of V_s is

$$V_s = \frac{A_v \times f_y \times d}{s}$$

in which,
ℓ_w = 5000 mm (shear wall's length)
d = 0,8ℓ_w (SNI, 2013 paragraph 11.9.4)
d = 0,8 x 5000 mm = 4000 mm
A_v = 760,57 mm^2
s = 100 mm
V_s = 11864914,29 N
checking the condition
$\phi V_n \geq V_u$
$V_n = V_c + V_s$
V_n = 23710599,96 N + 11864914,29 N = 35575514,24N
V_u = 244927,36N(from SAP 2000 v14®)
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)

ϕV_n = 26681635,68 N $\geq$ V_u = 244927,36N (OK).

use D22 – 100 with s = 100 mm.

4.5. Designing The Connection

4.5.1. Preface

The connection funcionate todistribute the forces that retained by structure's element to be trasported to other structure's element. Those forces then distributed to foundation. Besides that, the connection is made so that the stability of structure can be achived. The connection is expected to transfer all those forces simultaneously.

The wet connection in construction process is tend to be easier to be done than dry connection (non-topping) like *mechanical connection* and *welding connection* which are more complicated process. For the wet connection in the joint area, it is needed to give the additional reinforcement which is calculated based on the development bar. Besides that, it is needed to calculate the friction shear force between precast concrete with *overtopping*. In the construction process it is usual to use *shear connector* which funcionate as shear bearer and bending agent between precast slab and *overtopping* so that the slab can be monolith, and the integrity of structure can be achived.

In the construction process of precast concrete, a strong connection is always considered from economical and practical consideration. Besides that, it is needed to considerate about *serviceability*, strengthess, and production. The connection is expected to bear the loads and the moment which is dead load, live load, seismic load, and the combination of them.

The wet coonection is already used in many construction process as the solution for precast concrete.

According to SNI 2847:2013 paragraph 16.6.2.2, the length of support in support

- D = L_n/180, in which L_n is the span of precast element
- for *hollow-core* structure: 50 mm
- for *stemmed* structure: 75 mm

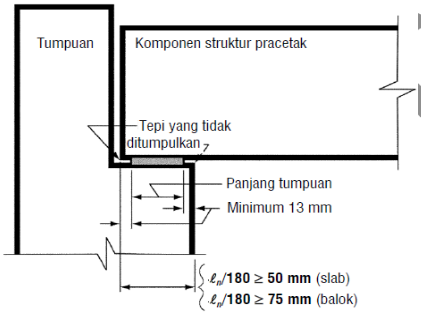


Figure 4.58. The support's length in support area
(source: SNI, 2013)

4.5.2. Concept of Connection's Design

4.5.2.1. Mechanism of Load Transfer

The function of connection is to transfer the loads from one structure's element to another structure's element. The mechanism of load trasfer will be explained below (see Figure 4.59).

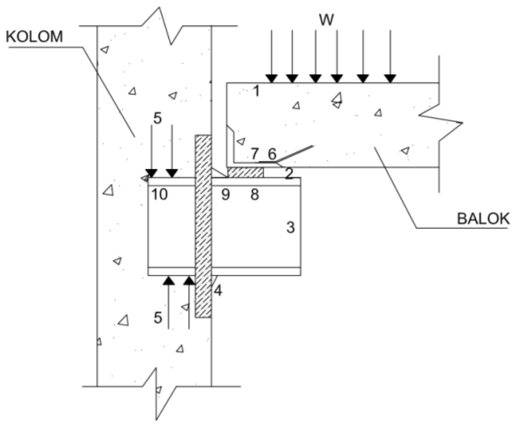


Figure 4.59. The mechanism of load transfer
(source: Private Documentation)

1. the load is absorbed by slab and transfered to support by shear strength
2. hen from support, transfered to *haunch* through *pads*’s compression force
3. *haunch* absorbs the vertical force from support by shear strength and flexural from steel profile
4. the vertical shear force and flexural transfer to steel plate through weld point
5. the concrete pillar gives the reaction to steel profile which is attached

The mechanism of load transfer due to shrinkage can be explained below

1. the concrete beam to reinforce bar
2. steel reinforce on the edge of the beam is attached by weld
3. steel on the edge of beam to *haunch* through friction on *bearing pads* and under *bearing pads,* partially the force due to volume change is reduced with deformation on *pads*
4. partially of the force due to the change of volume change is transfered through weld to steel plate
5. the force is retained by support and transfer by *stud* to concrete pillar.

4.5.2.2. Classification of System and Connection

The precast system is defined in two categories which are the location of its connection and type of connector

1. the location of connection

   the ductile portal can be defined according to its connection’s location and its location which expected to yield. The terms below are used for identifying the behaviour and character of connection

   - *strong*, the connection of precast elements which are strong and not yield due to quake

- *pins*, the precast elements connection behaves like pins if it considered from moment due to quake lateral force
- *ductile*, the precast elements connection should be ductile
- the location of plastis pins

2. the type of connector
    - *precast shell* with its core is casted by concrete
    - *cold joint* which is reinforced
    - *cold joint* which is partially reinforced and the joint *grouted*
    - *cold joint* which is partially reinforced and the joint not *grouted*
    - the mechanical connection

4.5.2.3. The Deformation Patterns

The example of deformation patterns can be seen on Figure 4.60.

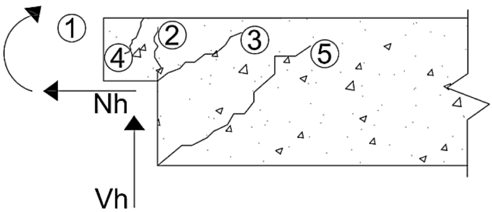


Figure 4.60. The example of deformation patterns (source: PCI, 2004)

PCI Design Handbook gives 5 patterns of deformation which are have to be investigated when designing *dapped-end* from beam

1. flexural and axial tension force on the edge
2. diagonal tension which is from edge corner
3. immediate shear force between bulge and beam
4. diagonal tension on the end of edge
5. support on the edge

in this final project the connection between precast's elements will use short console (see Figure 4.61)

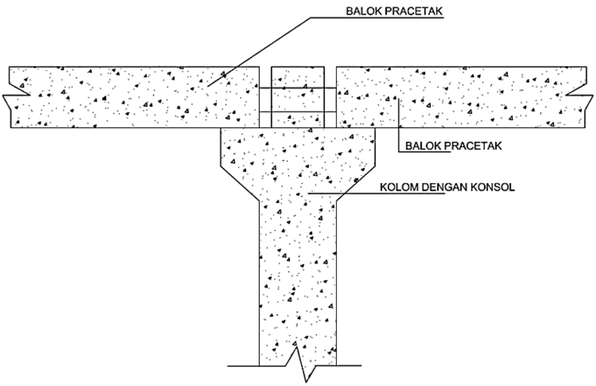


Figure 4.61. The model of connection between precast elements
(source: Private Documentation)

4.5.3. The Use of Overtopping Concrete

The function of *overtopping* concrete will be shown below

1. to guarantee that the precast concrete slab can work as a unity of horizontal diafragma which is stiff enough
2. to make sure that the distribution of vertical live load of precast component smoother
3. to smoothener the concrete's surface.

The thickness of topping is usually appproximately 50 mm to 100 mm. The transfer of shear force due to lateral load at structure's component shall work efficiently if the horizontal shear stress is not more than 5,50 kg/m^2. If the shear stress is exceeded then the topping shall not be considered as monolith structure, but it shall be considered as dead load which retained by precast element. The need of reinforcement steel bar due to *overtopping* in order to resist horizontal shear force can be designed by *shear friction concept*

$$A_{vf} = \frac{V_n}{fy \times \mu} \geq A_{vf} \ \min$$

in which

A_{vf} = the area of shear friction reinforcement
V_n = the area of nominal shear < 0,2 f'_c A_c (Newton)
< 5,5 A_c (Newton)
A_c = the area of concrete's section
f_y = the yield strength of reinforcement bar
μ = friction coefficient (use μ = 1)
A_{vfmin} = 0,018 A_c for f_y< 400 MPa
= 0,018 x 400/f_y for f_y> 400 MPa, $A_{vf\ min}$ shall not less than 0,0014 A_c.

4.5.4. Designing of Connection Between Beam and Pillar

4.5.4.1. Designing Console of Pillar

The connection between beam and pillar will use short console. The beam will be located at pillar's console then assembled into structure's unity. The designing of pillar's console will use condition in SNI 2847:2013 paragraph 11.8.

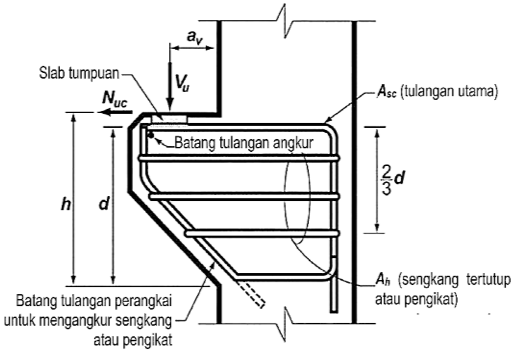


Figure 4.62. The model of short console on pillar (source: SNI, 2013)

According to SNI 2847:2013 paragraph 11.8 about the designing of short console, shall follow these conditions

1. designing the short console, the ratio between shear span to the height of $\mathbf{a_v/d}$is not more than 1, and subjected by factored horizontal tensile force, N_{uc}, should not more than Vu. The effective height, *d*, should be determined from front of support
2. the height at the outer edge of support shall not less than *0,5d*
3. the section which is located at the front of support shall be designed to resist $V_u$of a factored moment V_{ua} + N_{uc} (h-d), and factored horizontal tensile force, N_{uc}, simultaneously
   1) the design's calculation must follow conditions in SNI 2847:2013 paragraph 11.8, the ϕ value should be 0,75
   2) the design of shear friction's reinforcement, A_{vf}, for resisting V_u shall follow the condition in SNI 2847:2013 paragraph 11.6

a) for normal concrete, Vn should not more than the least of 0,2 x f'c x bw x d, (3,3 + 0,08 f'c) x bw x d, and 11 x bw x d
b) for light concrete or light sand, *Vn* should not more than the least of (0,2 – 0,07 a/d) x f'c x bw x d and (5,5 – 1,9 a/d) x bw x d
c) the A_f reinforcement to bear factored value of $[V_u a_v + N_{uc} (h-d)]$ should be calculated based on SNI 247:2013 paragraph 10.2 and paragraph 10.3
d) the $A_n$reinforcement for bearing factored tension shear strength, Nuc, should be determined by the value ofϕAn x fy $\geq N_{uc}$. The value of factored tension shear strength, Nuc, should at least or more than $0{,}2V_u$ except if the particular conditions are made to avoid the tension strength. Nuc should be considered as live load even if the tension which produced by creep, shrinkage, or temperature difference
e) the area of main tension reinforcement bar *Asc* should not less than the biggest value of (Af + An) and ($2A_{vf}/3$ + An)

4. the total area of Ah, the closed stirrup or parallel confinement, compared to main tension reinforcement bar shall not less than $0{,}5(A_{sc} - A_n)$, distributed Ah equally on (2/3) d next to main tension reinforcement
5. A_{sc}/bd should not less than 0,04 f'_c/f_y
6. on the front of short console, the main tension reinforcement *As* must be anchored by one of these
   1) the welding of structure on transversal reinforement bar with slightly same size; the welding is designed for expanding fy main tension reinforcement bar
   2) the bending of main tension reinforcement bar to form horizontal closed shape; or
   3) the other anchoring method

7. the area of support on short console shall not have more bulge exceed the part of *As*, or transversal anchored reinforcement bar.

4.5.4.2. Calculation Console on Pillar

The V_u value from primary structure's calculation

V_u = 149305 N

<u>General Data</u>

primary beam dimension = 40/80

console dimension:

bw = 600 mm

h = 500 mm

d = h - clear cover - D

= 500 - 40 - 25 = 435 mm

f'c = 30 MPa

fy = 390 MPa

a = 400 mm.

The condition which is used for designing the short console shall follow condition in SNI 2847:2013 paragraph 11.8. The geometry of short console shall followconditions in SNI 2847:2013 paragraph 11.8.1.

a/d = 400 mm/435 mm = 0,92 < 1 (OK)

$N_{uc} \leq V_u$

N_{uc} = 0,2 x 149305 N $\leq$ 149305 N

N_{uc} = 29861 N $\leq$ 149305 N (OK)

according to SNI 2847:2013 paragraph 11.8.3.1, the condition of V_n for normal concrete is

$V_n = V_u/\phi$

= 149305 N/0,75 = 199073,333 N

<u>The area of friction shear reinforcement</u>

according to SNI 2847:2013 paragraph 11.8.3.2 (a), for normal concrete, the value of V_n shall not more than

0,2 x f'c x bw x d = 0,2 x 30 x 600 x 435 N

= 1566000 N > V_n = 199073,333 N (OK)

11 x bw x d = 11 x 600 x 435 N
= 2871000 N > V_n = 199073,333 N (OK)
A_{vf} = $V_n/(f_y \times \mu)$
= 199073,33 N/(390 MPa x 1)
= 510,44 mm^2.

<u>The area of flexural reinforcement</u>
the support which will be used for short console is roller-pins, and so it let the deformation both lateral and horizontal, so the horizontal force due to long-term shrinkage and deformation of beam framework should not happen. According to SNI 2847:2013 paragraph 11.8.3.4, the minimum N_{uc} will be used.

Mu = V_{ua} x a + N_{uc} (h-d)
= (149305 x 400) + 29861 x (500 - 435)
= 61662965Nmm
m = $f_y/(0,85 \times f'_c)$
= 390 MPa/(0,85 x 30 MPa)
= 15,294
Rn = $M_u/(0,85 \times b \times d_x^2)$
= 61662965 Nmm/(0,85 x 600 mm x 435^2 mm^2)
= 0,638 N/mm^2
ρ_{need} = $1/m\, [1 – (1 – 2 \times m \times Rn/f_y)^{0,5}]$
= 0,0016
ρ_{min} = $¼ \times (f'_c)^{0,5}/f_y$
= 0,0035
use $\rho = \rho_{min}$ = 0,0035
A_{f1} = $M_u/(0,85 \times \phi \times f_y \times d)$
= 61662965/(0,85 x 0,75 x 390 x 435)
= 570,15 mm^2
A_{f2} = $\rho \times b \times d$
= 0,0035 x 600 x 435
= 913,5 mm^2
use $A_f = A_{f2}$ = 913,5 mm^2.

<u>Main reinforcement, As</u>

$A_n = N_{uc}/(\phi \times f_y)$
$= 29861/(0,75 \times 390)$
$= 102,08\ mm^2$.

according to SNI 2847:2013 paragraph 11.8.3.5

$A_s = A_f + A_n$
$= 913,5\ mm^2 + 102,08\ mm^2$
$= 1015,58 mm^2$

$A_s = (2 \times A_{vf}/3 + A_n)$
$= (2 \times 510,44/3 + 102,08)\ mm^2$
$= 442,373\ mm^2$

according to SNI 2847:2013 paragraph 11.8.5

$A_{s\ min} = 0,04 \times f'_c/f_y \times b \times d$
$= 0,04 \times 30/390 \times 600 \times 435\ mm^2$
$= 803,07\ mm^2$

use $A_{s\ min} = 1015,58\ mm^2$

according to SNI 2847:2013 paragraph 11.8.3.4

$A_h = 0,5\ (A_s - A_n)$
$= 0,5\ (1015,58 – 102,08)\ mm^2$
$= 456,75\ mm^2$

use reinforcement bar 4D25 ($A_s = 1964,28\ mm^2$)
installed along $2/3 \times d = 2/3 \times 235$ mm = 156,67 ≈ 200 mm
installed stirrup D13 with spacing s = 50 mm.

<u>The area of elastomer</u>

$V_u = \phi \times 0,85 \times f'_c \times A_l$
$A_l = V_u/(\phi \times 0,85 \times f'_c)$
$= 149305/(0,75 \times 0,85 \times 30)\ mm^2$
$= 7806,79\ mm^2$

use base plate150 x 150 mm^2 = 22500 mm^2 (thickness = 15 mm).

4.5.4.3. Calculation of Connection on Beam and Pillar

The system of connection between beam and pillar will use development bar's length with beam's reinforcement bar, especially reinforcement on bottom which will be anchored. The length of reinforcement bar is assumed to bear compression and tension, so in the designing process it'll be calculated into two conditions, tension and compression.

d_b = 25mm

A_s need = 1015,58 mm^2

A_s use = 1964,28 mm^2

<u>The length of development bar in compression condition</u>

according to SNI 2847:2013 paragraph 12.3.2,

$\ell_{dc} \geq (0,24 \times f_y/((f'_c{}^{0,5} \times \lambda)) \times d_b$

$\ell_{dc} \geq (0,24 \times 390/((30^{0,5} \times 1)) \times 25$ mm

$\ell_{dc} \geq 427,22$ mm ≈ 430 mm

$\ell_{dc} \geq (0,043 \times f_y) \times d_b$

$\ell_{dc} \geq (0,043 \times 390) \times 25$ mm

$\ell_{dc} \geq 419,25$ mm ≈ 420 mm

use ℓ_{dc} = 430 mm.

<u>The length of development bar in tension condition</u>

according to SNI 2847:2013 paragraph 12.5.1,

$\ell_{dh} = 8xd_b = 8x25$ mm = 200 mm

(SNI, 2013 paragraph 12.5.1)

ℓ_{dh} = 150 mm

(SNI, 2013 paragraph 12.5.1)

$\ell_{dh} = 0,24xfy/d_b x(f'_c)^{0,5}$ = 20,5 mm

(SNI, 2013 paragraph 12.5.2)

So, the development bar's length will use ℓ_{dh} = 200 mm

<u>The standard length of development hook bar</u>
acccording to SNI 2847:2013 paragraph 12.5.1,
$\ell_{dh} \geq 8 \times d_b = 8 \times 25 = 200$ mm
$\ell_{dh} \geq 150$ mm
$\ell_{dh} = (0{,}24\psi_e f_y/\lambda\ f'_c{}^{0,5})/d_b$
$= (0{,}24 \times 1 \times 390/1 \times 30^{0,5})/25$ mm
$= 20{,}5$ mm
use ℓ_{dh} = 200 mm with minimum standard hook 90^0 as 12 x d_b = 12 x 25 mm = 300 mm.

4.5.5. Designing of Connection Between Primary Beam and Secondary Beam

In the designing process of connection between primary beam and secondary beam will use short console. The secondary beam will be put on console which attached on primary beam then assemblied. The designing of console on the primary beam shall follow condition in SNI 2847:2013 paragraph 11.8.

4.5.5.1. Designing Console on Primary Beam

The V_u value is from secondary structure's calculation,
V_u = 97085,32 N
<u>General Data</u>
secondary beam dimension = 30/60
console dimension:
bw = 400 mm
h = 200 mm
d = h - clear cover - D
= 200 - 40 - 22 = 138 mm
f'c = 30 MPa
fy = 390 MPa
a = 100 mm.
a/d = 100 mm/138 mm
= 0,724

V_n = 97085,32 N/0,75
= 129447,09 N

according to SNI 2847:2013 paragraph 11.8.3.2 (a), for normal concrete, the value of V_n shall not more than

0,2 x f'c x bw x d = 0,2 x 30 x 300 x 138 N
= 248400 N > V_n = 129447,09 N (OK)

11 x bw x d = 11 x 300 x 138 N
= 455400 N > V_n = 129447,09 N (OK)

<u>The area of friction shear reinforcement</u>

according to SNI 2847:2013 paragraph 11.8.3.2, the value of A_{vf} is determined below

A_{vf} = $V_n/(f_y \times \mu)$ (SNI, 2013 paragraph 11.8.3.2)
= 129447,09 N/(390 MPa x 1)
= 331,915 mm^2.

<u>The area of flexural reinforcement</u>

the support which will be used for short console is roller-pins, and so it let the deformation both lateral and horizontal, so the horizontal force due to long-term shrinkage and deformation of beam framework should not happen. According to SNI 2847:2013 paragraph 11.8.3.4, the minimum N_{uc} will be used.

N_{uc} = 0,2 x V_u
= 0,2 x 97085,32 N
= 19417,06 N

Mu = V_{ua} x a + N_{uc} (h-d)
= (97085,32 x 100) + 19417,06 x (200 - 138)
= 10912389,72 Nmm

m = $f_y/(0,85 \times f'_c)$
= 390 MPa/(0,85 x 30 MPa)
= 15,294

Rn = $M_u/(0,85 \times b \times d_x^2)$
= 10912389,72 Nmm/(0,85 x 300 mm x 138^2 mm^2)
= 2,247

ρ_{need} = 1/m [1 – (1 – 2 x m x Rn/$f_y)^{0,5}$]
= 0,0061
ρ_{min} = ¼ x (f'$_c)^{0,5}$/f_y
= 0,0035
use ρ= ρ_{need} = 0,0061.
A_{f1} = M_u/(0,85 x ϕ x f_y x d)
= 10912389,72/(0,85 x 0,75 x 390 x 138)
= 318,05 mm^2
A_{f2} = ρ x b x d
= 0,0061 x 300 x 138
= 252,54 mm^2
use A_f = A_{f1} = 318,05 mm^2.

<u>Main reinforcement, As</u>
A_n = N_{uc}/(ϕ x f_y)
= 19417,06/(0,75 x 390)
= 66,383 mm^2.
according to SNI 2847:2013 paragraph 11.8.3.5
A_s = A_f + A_n
= 318,05 mm^2 + 66,383 mm^2
= 384,43 mm^2
A_s = (2 x A_{vf}/3 + A_n)
= (2 x 331,92/3 + 66,383) mm^2
= 287,663 mm^2

according to SNI 2847:2013 paragraph 11.8.5
$A_{s\ min}$ = 0,04 x f'$_c$/f_y x b x d
= 0,04 x 30/390 x 300 x 138 mm^2
= 127,38 mm^2
use $A_{s\ min}$ = 384,43 mm^2

according to SNI 2847:2013 paragraph 11.8.3.4
A_h = 0,5 (A_s - A_n)
= 0,5 (384,43– 66,383) mm^2
= 159,02 mm^2

use reinforcement bar 4D22 (A_s = 1521,14 mm^2)
installed along 2/3 x d = 2/3 x 138 mm = 92 ≈ 100 mm
installed stirrup D13 with spacing s = 50 mm.

<u>The area of elastomer</u>

V_u = ϕ x 0,85 x f'_c x A_l
A_l = $V_u/(\phi$ x 0,85 x $f'_c)$
= 97085,32/(0,75 x 0,85 x 30) mm^2
= 5076,36 mm^2

use elastomer 100 x 100 mm^2 = 10000 mm^2 (thickness = 15 mm).

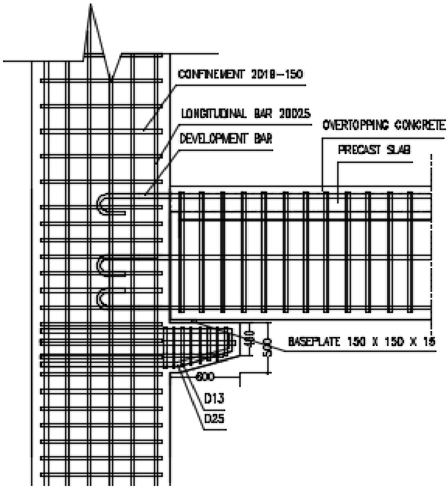


Figure 4.63. The connection between pillar and precast primary beam
(source: Private Documentation)

4.5.5.2. Calculation of Connection on Primary Beam and Secondary Beam

The system of connection between primary beam and secondary beam will use development bar's length with beam's reinforcement bar, especially reinforcement on bottom which will be anchored. The length of reinforcement bar is assumed to bear compression and tension, so in the designing process it'll be calculated into two conditions, tension and compression.

d_b = 22 mm
A_s need = 384,43 mm^2
A_s use = 1521,14 mm^2

<u>The length of development bar in compression condition</u>
according to SNI 2847:2013 paragraph 12.3.2,
$\ell_{dc} \geq (0,24 \text{ x } f_y/((f'_c{}^{0,5} \text{ x } \lambda)) \text{ x } d_b$
$\ell_{dc} \geq (0,24 \text{ x } 390/((30^{0,5} \text{ x } 1)) \text{ x } 22$ mm
$\ell_{dc} \geq 375,95$ mm $\approx$ 400 mm

$\ell_{dc} \geq (0,043 \text{ x } f_y) \text{ x } d_b$
$\ell_{dc} \geq (0,043 \text{ x } 390) \text{ x } 22$ mm
$\ell_{dc} \geq 368,94$ mm $\approx$ 370 mm
use ℓ_{dc} = 400 mm.

<u>The length of development bar in tension condition</u>
according to SNI 2847:2013 paragraph 12.5.1,
$\ell_{dh} = 8xd_b = 8x22$ mm = 176 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0,24xfy/d_b x(f'_c)^{0,5}$ = 21,86 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 180 mm.

<u>The standard length of development hook bar</u>
according to SNI 2847:2013 paragraph 12.5.1,
$\ell_{dh} \geq 8 \text{ x } d_b = 8 \text{ x } 22 = 176$ mm
$\ell_{dh} \geq 150$ mm

$\ell_{dh} = (0,24\psi_e f_y/\lambda\ f'_c{}^{0,5})/d_b$
$= (0,24 \text{ x } 1 \text{ x } 390/1 \text{ x } 30^{0,5})/22$ mm
$= 23,3$ mm
use ℓ_{dh} = 200 mm with minimum standard hook as
4 x d_b = 4 x 22 mm = 88 mm ≈ 90 mm≥ 65 mm.

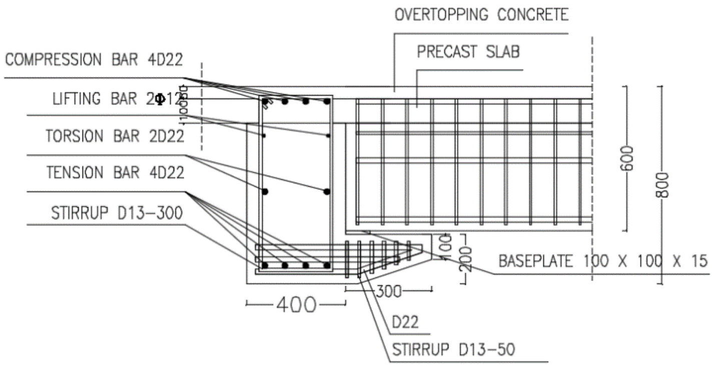


Figure 4.64. The connection between precast primary beam and precast secondary beam
(source: Private Documentation)

4.5.6. Designing of Connection Between Secondary Beam and Slab

The connection between secondary beam and slab will use the support bar which is installed along and perpendicular to beam (connecting the slab's shear connector). Then the precast slab will be given *overtopping* concrete in order to become monolith structure.

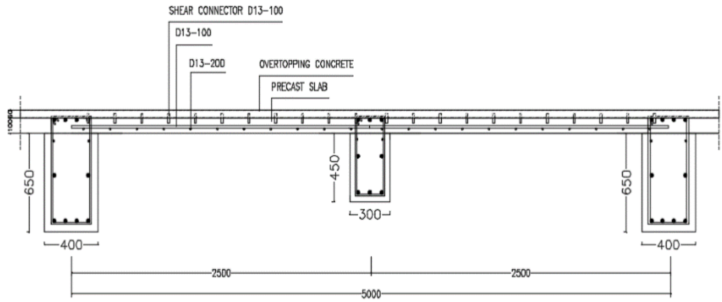


Figure 4.65. The connection between precast secondary beam and precast slab
(source: Private Documentation)

4.5.6.1. Designing The Development Bar of Precast Slab

According to the previously calculation, the result of reinforcement calculation of precast slab type is shown below,
d_b = 13 mm

flexural bar (X direction)
A_s need = 1109,528mm^2
A_s use = 1327,857 mm^2

shrinkage bar (Y direction)
A_s need = 227 mm^2
A_s use = 663,93 mm^2

<u>The length of development bar of flexural bar in compression condition</u>
according to SNI 2847:2013 paragraph 12.3.2,
$\ell_{dc} \geq (0{,}24 \text{ x } f_y/((f'_c{}^{0,5} \text{ x } \lambda)) \text{ x } d_b$
$\ell_{dc} \geq (0{,}24 \text{ x } 390/((30^{0,5} \text{ x } 1)) \text{ x } 13 \text{ mm}$
$\ell_{dc} \geq 222{,}15 \text{ mm} \approx 230 \text{ mm}$

$\ell_{dc} \geq (0{,}043 \text{ x } f_y) \text{ x } d_b$
$\ell_{dc} \geq (0{,}043 \text{ x } 390) \text{ x } 13 \text{ mm}$
$\ell_{dc} \geq 218{,}01 \text{ mm} \approx 220 \text{ mm}$
use $\ell_{dc} = 220$ mm.

<u>The length of development bar of flexural bar in tension condition</u>
$\ell_{dh} = 8xd_b = 8x13 \text{ mm} = 104 \text{ mm}$
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 150$ mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_b x(f'_c)^{0,5} = 37 \text{ mm}$
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use $\ell_{dh} = 150$ mm.

<u>The length of development bar of shrinkage bar in compression condition</u>
according to SNI 2847:2013 paragraph 12.3.2,
$\ell_{dc} \geq (0{,}24 \text{ x } f_y/((f'_c{}^{0,5} \text{ x } \lambda)) \text{ x } d_b$
$\ell_{dc} \geq (0{,}24 \text{ x } 390/((30^{0,5} \text{ x } 1)) \text{ x } 13 \text{ mm}$
$\ell_{dc} \geq 222{,}15 \text{ mm} \approx 230 \text{ mm}$
$\ell_{dc} \geq (0{,}043 \text{ x } f_y) \text{ x } d_b$
$\ell_{dc} \geq (0{,}043 \text{ x } 390) \text{ x } 13 \text{ mm}$
$\ell_{dc} \geq 218{,}01 \text{ mm} \approx 220 \text{ mm}$
use $\ell_{dc} = 220$ mm.

<u>The length of development bar of shrinkage bar in tension condition</u>
$\ell_{dh} = 8xd_b = 8x13 \text{ mm} = 104 \text{ mm}$
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 150$ mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_b x(f'_c)^{0,5} = 37 \text{ mm}$
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use $\ell_{dh} = 150$ mm.

4.6. Designing The Basement's Structure

4.6.1. Reinforcement of Basement's Wall

4.6.1.1. Preface

The basement's wall will use cast in situ concrete, the calculation of shear force and moment force will be shown below

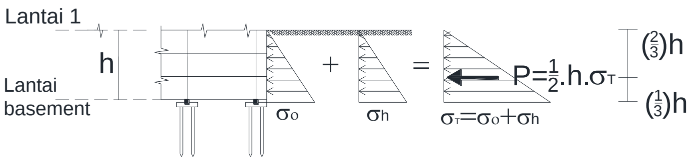


Figure 4.66. The diagram force at basement's wall
(source: Private Documentation)

the general soil data for calculation of basement's wall will be shown below,

(source: Laboratory of Soil Engineeering ITS Surabaya, 2016)
$c = 5$ kN/m^2
$\phi = 0$ kN/m^2
$\gamma_{soil} = 16$ kN/m^3 (saturated)
$\gamma_w = 10$ kN/m^3
$\beta = 0^0$

by using Rankine formula, the value of Ka is

$$Ka = \frac{\text{Cos } \beta*((\text{Cos } \beta - (\text{Cos}^2 \beta - \text{Cos}^2 \Phi)\text{^}0.5))}{((\text{Cos } \beta + (\text{Cos}^2\beta - \text{Cos}^2 \Phi)\text{^}0.5))}$$

Kp = 1/Ka
Ka water = 1
Kp water = 1
$\sigma_{ha} = \sigma_v$ x Ka – ((2.c.(Ka)^0,5)) → active soil pressure
$\sigma_{hp} = \sigma_v$ x Kp – ((2.c.(Kp)^0,5)) → passive soil pressure.

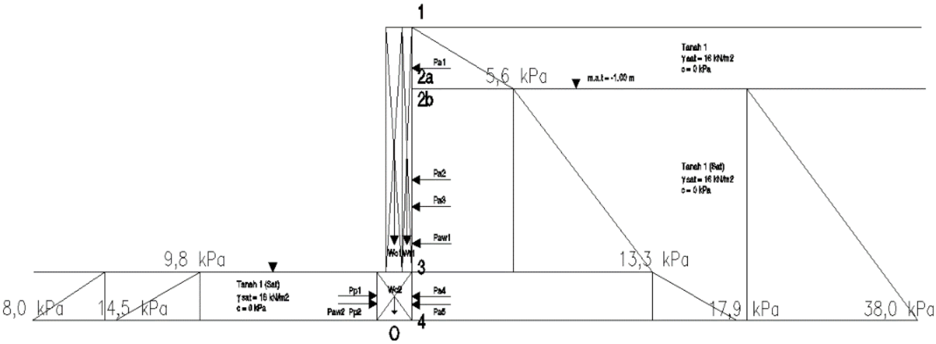


Figure 4.67. The soil and water pressures of basement's wall (source: Private Documentation)

The recapitulation of horizontal soil pressure and soil pressure's force will be shown in Table 4.36.

Table 4.36. Recapitulation of Horizontal Soil Pressure

No.	Point	Soil	C (kPa)	ϕ^0	Ka	Kp	σ_v (kPa)	σ_{ha} (kPa)	σ_{hp} (kPa)
1	1	1	5	0	0,96	1,1	0	0	-
2	2a	1	5	0	0,96	1,1	16	5,62	-
3	2b	1	5	0	0,96	1,1	16	5,62	-
4	3	1	5	0	0,96	1,1	24	13,35	-
5	4	1	5	0	0,96	1,1	28,8	17,9	-
6	3	1	5	0	0,96	1,1	0	-	9,83
7	4	1	5	0	0,96	1,1	4,8	-	14,46

Table 4.37. Recapitulation of Soil Pressure's Force

No.	Section	Vertical (kN)	Distance to O point (m)	Horizontal (kN)	Distance to O point (m)
1	Wt1	4,2	0,24	-	-
2	Wc1	28,8	0	-	-
3	Wc2	12,48	0	-	-
4	Pa1	-	-	45,013	4,14
5	Pa2	-	-	101,279	2,3
6	Pa3	-	-	69,54	1,86
7	Pa4	-	-	64,099	0,4
8	Pa5	-	-	11,127	0,26
9	Paw1	-	-	722	1,27
10	Pp1	-	-	-47,175	0,41
11	Pp2	-	-	-22,255	0,28
12	Paw2	-	-	-32	0,28
Total	45,48 kN		Total	911,636 Kn	

Table 4.38. Recapitulation of Moment Result

No.	Vertical (kN)	d (m)	Moment (kNm)	Horizontal (kN)	h (m)	Moment (kNm)
1	4,2	0,24	-1,008	-	-	-
2	28,8	0	0	-	-	-
3	12,48	0	0	-	-	-
4	-	-	-	45,013	4,14	186,35
5	-	-	-	101,279	2,3	232,94
6	-	-	-	69,54	1,86	129,36
7	-	-	-	64,099	0,4	25,639
8	-	-	-	11,127	0,26	2,893
9	-	-	-	722	1,27	916,94
10	-	-	-	-47,175	0,41	-19,34
11	-	-	-	-22,255	0,28	-6,231
12	-	-	-	-32	0,28	-8,96
Total		-1,008 kNm		Total	1459,595 kNm	
Grand Total		1458,58 kNm				

total moment = 1458,58 kNm
total vertical force = 45,48 kNm
total horizontal force = 911,636 kNm.

4.6.1.2. General Data

The general data for reinforcement bar of basement's wall will be shown below

dimension of wall		= 5000 x 4000 mm^2
f'_c		= 30 MPa
f_y		= 390 MPa
clear cover		= 40 mm
bar's diameter		= 22 mm, As = $\pi/4xd^2$ = 380,28 mm^2
thickness		= 300 mm
b		= 5000 mm
h		= 4000 mm
d		= 300 – 40 – D/2 – D = 227 mm
ϕ		= 0,9
M_u	= 1458,58 kNm	= 1458580000 Nmm

<u>Flexural Bar</u>
due to M_u

$$\text{Rn} = \frac{\text{Mn}}{\text{b} \times d^2} = 6{,}29 N/mm^2$$

$$\rho_{\text{need}} = \frac{1}{\text{m}}\left(1 - \sqrt{1 - \frac{2 \times \text{m} \times \text{Rn}}{\text{fy}}}\right) = 0{,}0188$$

$\rho_{need} < \rho_{min}$ = 0,002 < 0,0035
use $\rho = \rho_{need}$ = 0,0188
$As_{need} = \rho xbxd$ = 21338 mm^2
n = As_{need}/As_{bar} = 21338 mm^2/380,28 mm^2 = 56,11
use n = 60 → $As_{use} = nxAs_{bar}$ = 22816,8 mm^2
$As_{use} > As_{need}$ = 22816,8 mm^2 > 21338 mm^2 (OK)
s_{max} = (5000 – 2 x 40)/(60 – 1) = 83,38 mm > 25 mm (OK)
use D22 – 80.

Checking The Minimum Thickness of Wall

According to SNI 2847:2013 paragraph 15.5.3.2 which is stated that the thickness of basement wall shall not less than 190 mm. Meanwhile, h = 300 mm > 190 mm (OK).

Checking The Ratio of Reinforcement

According to SNI 2847:2013 paragraph 14.3.3 which is stated the value of ρ_t shall more than 0,0025.

$\rho_t = A_s$ x n/(b x h)

$\rho_t = 380{,}28\ mm^2$ x 60/(5000 mm x 300 mm)

$\rho_t = 0{,}0152 > 0{,}0025$ (OK).

4.6.2. Reinforcement of Basement's Slab

4.6.2.1. General

The early design of slab BBA consists of 20 cm thickness of cast in place concrete.The calculation will use the conditions based on SNI 2847:2013.

4.6.2.2. General Data

The general data for reinforcement of slab will be shown below

slab's thickness (overall)	= 20 cm
slab type	= BBA (500 cm x 250 cm)
dimension	= 500 x 250 x 20 cm^3
diameter of bar (D)	= 13 mm
L_y(clear span)	= 4350mm
L_x(clear span)	= 1925 mm
L_y/L_x	= 2,25
f'_c	= 30 MPa
f_y	= 390 MPa

4.6.2.3. Load Calculation

The load calculation for slab will be shown below

1. dead load

slab (overall) : (0,20+0,02)x2400 kg/m^2= 528 kg/m^2

plumbing and ducting	: 25 kg/m^2	= 25 kg/m^2
sanitary	: 20 kg/m^2	= 20 kg/m^2
mortar mixture (2 cm)	: 2x21 kg/m^2	= 42 kg/m
asphalt (1 cm)	: 14 kg/m^2	= 14 kg/m^2
DL (total)	:	= 629 kg/m^2
2. live load		
live load	: 800 kg/m^2	= 800 kg/m^2
LL (total)	:	= 800kg/m^2

4.6.2.4. Load Combination of Slab

The combination of load for slab is based on SNI 2847:2013 paragraph9.2.1.The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of slab

(concrete's age > 30 days)

Qu = 1,2×629 + 1,6×800 = 2034,8 kg/m^2

4.6.2.5. Moment Calculation of Slab

The equation for moment calculation isbased on PBBI (1971). According to PBBI (1971), the X values are determined by L_y/L_x value which as 2,25. The X values are X1 = 41, X2 = 18, X3 = 83, X4 = 57. The equation of moment values are show below

Mlx (+) = 0,001 x qx Lx^{2}x X1
Mly (+) = 0,001 x qx Lx^{2}x X2
Mtx (-) = 0,001 x qx Lx^{2}x X3
Mty (-) = 0,001 x qx Lx^{2}x X4

moment calculation of slab
(q = 2034,8 kg/m^2, L_x = 192,5 cm)
Mlx (+) = 309,148 kgm
Mly (+) = 135,723 kgm
Mtx (-) = 625,837 kgm

Mty (-) = 429,792 kgm

The Mu values are taken as
Mu = 625,837 kgm (-)

4.6.2.6. Calculation of Reinforcement Bar

The general data for calculation of reinforcement bar of slab is shown below

slab's dimension (actual)	= 4350mm × 1925 mm
slab's thickness (cast in place)	= 200 mm
clear cover	= 50 mm
bar's diameter (D)	= 13 mm
f'_c (28 days)	= 30 MPa
f_y	= 390 MPa
$\beta_1 = L_y/L_x$	= 2,259 (one way slab)
dx = 200 - 50 - (13/2)	= 143,5 mm
dy = 200 - 50 - 13 - (13/2)	= 130,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$

$As = \pi/4xd^2 = 132{,}78\ mm^2$

$a = Asxf_y/(0{,}85xf'_cxb) = 2{,}03\ mm$

$c = a/\beta_1 = 2{,}43\ mm$

$\varepsilon_t = (d/c - 1)x0{,}003 = 0{,}158 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)

$m = f_y/(0{,}85xf'_c) = 15{,}3$

$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1)

If the slab is determined as one way slab, then the main reinforcement bar of slab is transversal reinforcement bar only and the longitudinal reinforcement bar will funcionate astemperature and shrinkage bearer. Both of them will use spiral bar D13 (diameter = 13 mm, As = 132,7857 mm^2).

4.6.2.7. Reinforcement Bar

The general data for reinforcement bar of slab will be shown below

slab's thickness	= 200 mm, f'_c = 30 MPa (> 30 days)
clear cover	= 30 mm
bar's diameter	= 13 mm, As = $\pi/4xd^2$ = 132,78 mm^2
b	= 1000 mm
d	= 143,5 mm
ϕ	= 0,9

M_u = 625,837 kgm = 6258370,8 Nmm

<u>Main Bar/Transversal Bar</u>

$M_n = M_u/\phi$ = 6953745 Nmm

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b}\times d^2} = 0{,}337\, N/mm^2$$

$$\rho_{\mathrm{need}} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2\times\mathrm{m}\times\mathrm{Rn}}{\mathrm{fy}}}\right) = 0{,}000872$$

$\rho_{need} < \rho_{min}$ = 0,000872 < 0,002

use $\rho = \rho_{min}$ = 0,002

As_{need} = ρxbxd = 287 mm^2

n = As_{need}/As_{bar} = 287 mm^2/132,7857 mm^2 = 2,16

use n = 5 → As_{use} = nxAs_{bar} = 663,928 mm^2

$As_{use} > As_{need}$= 663,92857 mm^2 > 287 mm^2 (OK)

s = b/n = 1000 mm/5 = 200 mm

So, the transversal bar will use D13-200.

<u>Shrinkage Bar/Longitudinal Bar</u>

As_{need} = 0,002xbxd = 287 mm^2

n = As_{need}/As_{bar} = 287 mm^2/132,7857 mm^2 =2,16

use n = 5 → As_{use} = nxAs_{bar} = 663,92857 mm^2

$As_{use} > As_{need}$= 663,92857 mm^2 >287 mm^2 (OK)

s = b/n = 1000 mm/5 = 200 mm

So, the longitudinal bar will use D13-200.

Development Bar's Length
ℓ_{dh} = 8xd_b = 8x13 mm = 104 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 0,24xfy/d_bx$(f'_c)^{0,5}$ = 39,43 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 150 mm.

4.6.2.8. Checking The Deflection of Slab

The deflection of slab is calculated using equation below
δ (deflection) = (5/384) x (q_u x L^4/EI)
δ_{max} = L_n/480 (in cm) (SNI 2847:2013paragraph 9.5.3.1)
in which q_u is taken as 2034,8 kg/m^2x 1 m (see previous paragraph), I is the moment of inertia of slab (I = 1/12xbxh^3), and E equals 4700x$(f'c)^{0,5}$, f'c in 28 days

Deflection (Longitudinal, L_n = 435 cm, b =L_n, h = 20 cm)
δ_b = (5/384) x (q_u x L^4/EI) = 0,127 cm
δ_{max} = L_n/480 = 0,906 cm
$\delta_b < \delta_{max}$ (OK)

Deflection (Transversal, S_n = 192,5 cm, b =S_n, h = 20 cm)
δ_a = (5/384) x (q_u x L^4/EI) = 0,0110 cm
δ_{max} = L_n/480 = 0,401 cm
$\delta_a < \delta_{max}$ (OK)

4.6.2.9. Checking The Shrinkage of Slab

The shrinkage of slab is calculated using equation below (see SNI, 2013 paragraph 10.6.4)
s = 380x280/f_s- 2,5 Cc $\leq$ 380x280/f_s
in which f_s = 2/3xf_y = 266,67 MPa,
Cc = clear cover + 1/2xD = 56,5 mm
s = 398,99 - 141,25 = 257,74 N/mm $\leq$ 398,99 N/mm (OK)

4.6.2.10. Load Calculation (Uplift)

The load calculation for slab will be shown below

1. dead load

slab (overall)	: (0,20+0,02)x2400 kg/m^2	= 528 kg/m^2
plumbing and ducting	: 25 kg/m^2	= 25 kg/m^2
sanitary	: 20 kg/m^2	= 20 kg/m^2
mortar mixture (2 cm)	: 2x21 kg/m^2	= 42 kg/m
asphalt (1 cm)	: 14 kg/m^2	= 14 kg/m^2
DL (total)	:	= 629 kg/m^2

2. live load → due to uplift force

uplift force	: 1000 kg/m^3 x 4 m	= 4000 kg/m^2
LL (total)	:	= 4000kg/m^2

4.6.2.11. Load Combination of Slab

The combination of load for slab is based on SNI 2847:2013 paragraph 9.2.1.The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of slab

(concrete's age > 30 days)

Qu = 1,2×629 + 1,6×4000 = 7154,8 kg/m^2

4.6.2.12. Moment Calculation of Slab

The equation for moment calculation isbased on PBBI (1971). According to PBBI (1971), the X values are determined by L_y/L_x value which as 2,25. The X values are X1 = 41, X2 = 18, X3 = 83, X4 = 57. The equation of moment values are show below

Mlx (+) = 0,001 x qx Lx^2x X1

Mly (+) = 0,001 x qx Lx^2x X2

Mtx (-) = 0,001 x qx Lx^2x X3

Mty (-) = 0,001 x qx Lx^2x X4

moment calculation of slab
(q = 7154,8 kg/m^2, L_x = 192,5 cm)

Mlx (+)	= 1087,033 kgm
Mly (+)	= 477,234 kgm
Mtx (-)	= 2200,579 kgm
Mty (-)	= 1511,241 kgm

The Mu values are taken as
Mu = 2200,579 kgm (-)

4.6.2.13. Calculation of Reinforcement Bar

The general data for calculation of reinforcement bar of slab is shown below

slab's dimension (actual)	= 4350 mm × 1925 mm
slab's thickness (cast in place)	= 200 mm
clear cover	= 30 mm
bar's diameter (D)	= 13 mm
f'_c	= 30 MPa
f_y	= 390 MPa
dx = 200 - 50 - (13/2)	= 143,5 mm
dy = 200 - 50 - 13 - (13/2)	= 130,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$

$As = \pi/4 x d^2 = 132{,}78\ mm^2$

$a = As x f_y/(0{,}85 x f'_c x b) = 2{,}03$ mm

$c = a/\beta_1 = 2{,}43$ mm

$\varepsilon_t = (d/c - 1) x 0{,}003 = 0{,}158 \rightarrow \phi = 0{,}9$ (SNI, 2013 Figure S9.3.2)

$m = f_y/(0{,}85 x f'_c) = 15{,}3$

$\rho_{min} = 0{,}002$ (SNI, 2013 paragraph 7.12.2.1)

If the slab is determined as one way slab, then the main reinforcement bar of slab is transversal reinforcement bar only and the longitudinal reinforcement bar will funcionate as temperature and shrinkage bearer. Both of them will use spiral bar D13 (diameter = 13 mm, As = 132,7857 mm^2).

4.6.2.14. Reinforcement Bar

The general data for reinforcement bar of slab will be shown below

slab's thickness		= 200 mm, f'_c = 30 MPa (> 30 days)
clear cover		= 30 mm
bar's diameter		= 13 mm, As = $\pi/4 x d^2$ = 132,78 mm^2
b		= 1000 mm
d		= 143,5 mm
ϕ		= 0,9
M_u	= 2200,579 kgm	= 22005795 Nmm

<u>Main Bar/Transversal Bar</u>

$M_n = M_u/\phi$ = 24450883 Nmm

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 1{,}187\, N/mm^2$$

$$\rho_{need} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2 \times \mathrm{m} \times \mathrm{Rn}}{\mathrm{fy}}}\right) = 0{,}0032$$

$\rho_{need} > \rho_{min}$ = 0,0032 > 0,002

use $\rho = \rho_{nedd}$ = 0,0032

$As_{need} = \rho x b x d$ = 447,57 mm^2

n = As_{need}/As_{bar} = 447,57 mm^2/132,7857 mm^2 = 3,37

use n = 5 → $As_{use} = n x As_{bar}$ = 663,928 mm^2

$As_{use} > As_{need}$= 663,92857 mm^2 > 436,381 mm^2 (OK)

s = b/n = 1000 mm/5 = 200 mm

So, the transversal bar will use D13-200.

<u>Shrinkage Bar/Longitudinal Bar</u>
As_{need} = 0,002xbxd = 287 mm^2
n = As_{need}/As_{bar} = 287 mm^2/132,7857 mm^2 =2,16
use n = 5 → As_{use} = n x As_{bar} = 663,92857 mm^2
$As_{us\ e}$> As_{need}= 663,92857 mm^2 >287 mm^2 (OK)
s = b/n = 1000 mm/5 = 200 mm
So, the longitudinal bar will use D13-200.

<u>Development Bar's Length</u>
ℓ_{dh} = 8xd_b = 8x13 mm = 104 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 0,24xfy/d_bx$(f'_c)^{0,5}$ = 39,43 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 150 mm.

4.6.2.15. Checking The Deflection of Slab

The deflection of slab is calculated using equation below
δ (deflection) = (5/384) x (q_u x L^4/EI)
δ_{max} = L_n/480 (in cm) (SNI 2847:2013paragraph 9.5.3.1)
in which q_u is taken as 7154,8 kg/m^2x 1 m (see previous paragraph), I is the moment of inertia of slab (I = 1/12xbxh^3), and E equals 4700x$(f'c)^{0,5}$, f'c in 28 days
Deflection (Longitudinal, L_n = 435 cm, b =L_n, h = 20 cm)
δ_b = (5/384) x (q_u x L^4/EI) = 0,446 cm
δ_{max} = L_n/480 = 0,906 cm
$\delta_b < \delta_{max}$ (OK)
Deflection (Transversal, S_n = 192,5 cm, b =S_n, h = 20 cm)
δ_a = (5/384) x (q_u x L^4/EI) = 0,0387 cm
δ_{max} = L_n/480 = 0,401 cm
$\delta_a < \delta_{max}$ (OK)

4.6.2.16. Checking The Shrinkage of Slab

The shrinkage of slab is calculated using equation below (see SNI, 2013 paragraph 10.6.4)

$s = 380x280/f_s - 2{,}5\ Cc \leq 380x280/f_s$

in which $f_s = 2/3xf_y = 266{,}67$ MPa,

cc = clear cover + 1/2xD = 56,5 mm

s = 398,99 - 141,25 = 257,74 N/mm $\leq$ 398,99 N/mm (OK)

For the recapitulation of reinforcement bar will be shown in Table 4.39.

Table 4.39. Recapitulation of Reinforcement Bars of Basement's Slab

Type	Dimension	Flexural Bar					Longitudinal Bar		
		As need (mm^2)	As use (mm^2)	s (mm)	n	Notation	As use (mm^2)	s (mm)	Notation
BBA	435 x 192,5	436,381	663,928	200	5	D13-200	663,93	200	D13-200
BBB	445 x 192,5	436,381	663,928	200	5	D13-200	663,93	200	D13-200
BBC	435 x 192,5	436,381	663,928	200	5	D13-200	663,93	200	D13-200
BBD	435 x 200,2	436,381	663,928	200	5	D13-200	663,93	200	D13-200
BBE	435 x 197,5	436,381	663,928	200	5	D13-200	663,93	200	D13-200

4.6.3. Reinforcement of Basement's Secondary Beam

4.6.3.1. General Data

The general data for reinforcement of secondary beam will be shown below

b (width)	= 50 cm
h (height)	= 70 cm
L(axis to axis)	= 5000mm
L(actual)	= 4350 mm
bar's diameter (D/deform)	= 22 mm
bar's diameter/stirrup(ϕ/plain)	= 12 mm
f'_c	= 30 MPa
f_y	= 390 MPa

4.6.3.2. Load Calculation

The load calculation for secondary beam will be shown below

1. dead load

secondary beam BAB1 : 0,5x0,7x2400 kg/m= 840 kg/m

q of slab BBA : 2034,8 kg/m^2

by using Ly = 500 cm – (65 cm/2 + 65 cm/2) = 435 cm and Lx = 250 cm – (65 cm/2 + 50 cm/2) = 192,5 cm, it is obtained the value of qeq due to slab's load

(two trapezoids load)

q_{eq} = 2 x q x 1/2 x Lx x ((1 -1/3 x $(Lx/Ly)^2$)

q= 2034,8 kg/m^2

q_{eq} = 3661,30 kg/m→ due to slab's load

DL= 3661,30 kg/m + 840 kg/m= 4501,3 kg/m

DL (k, shock coefficient = 1,2) = 1,2x4501,3 kg/m

= 5401,56 kg/m

DL (total) : = 5401,56 kg/m

2. live load

live load for *basement*: = 800 kg/m^2

q_{eq} = 1439,473 kg/m → due to live load for *basement*

LL (total) : = 1439,473 kg/m^2

4.6.3.3. Load Combination of Secondary Beam

The combination of load for secondary beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of secondary beam

Qu = 1,2×5401,56 + 1,6x1439,473 = 8785,029 kg/m

4.6.3.4. Moment Calculation of Secondary Beam

According to SNI (2013), the equation of moment values for secondary beamare shown below

Ml (+) = 1/16 x qx L^2 → at field area

Mt (-) = 1/10 x qx L^2 → at support area

moment calculation of secondary beam

(q = 8785,029 kg/m, L = 5 m)

Ml (+) = 13726,607 kgm

Mt(-) = 21962,571 kgm

The Mu values are taken as

Mul = 13726,607 kgm(+)

Mut = 21962,571 kgm (-)

4.6.3.5. Shear Force Calculation of Secondary Beam

The shear force calculation of secondary beam will use shear force equation Vu = 1/2 x q x L.

shear force calculation of secondary beam

(q = 8785,029 kg/m, L = 5 m)

Vu = 21962,57 kg.

4.6.3.6. Calculation of Reinforcement Bar

The calculation of reinforcement bar will be divided into two conditions (before monolith condition and after monolith condition). The general data for calculation of reinforcement bar of secondary beam is shown below

secondary beam's dimension (overall) = 500mm × 700mm
clear cover = 50 mm
bar's diameter (D) = 22 mm
bar's diameter/stirrup(ϕ/plain) = 12 mm
f'_c = 30 MPa
f_y = 390 MPa
d' = clear cover + stirrup + D/2 = 73 mm
d = 700 - 50 - 12 - (22/2) = 627 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(f'c-28)}{7} \geq 0{,}65$

$\beta_1 = 0{,}85 - 0{,}05\,\frac{(30-28)}{7} = 0{,}8357143 \geq 0{,}65$

As = $\pi/4xd^2$ = 380,286 mm^2

a = $Asxf_y/(0{,}85xf'_cxb)$ = 11,63 mm

c = a/β_1 = 13,92 mm

ε_t = (d/c - 1)x0,003 = 0,129 → ϕ = 0,9 (SNI, 2013 Figure S9.3.2)

m = $f_y/(0{,}85xf'_c)$ = 15,29

$\rho_{min} = ¼ \text{ x } (f'_c)^{0{,}5}/f_y$ = 0,0035 (SNI, 2002 paragraph 12.5.1)

4.6.3.7. Reinforcement Bar

The general data for reinforcement bar of seconday beam after monolith condition will be shown below

secondary beam's dimension = 500x 700 mm^2
f'_c = 30 MPa (28 days)
clear cover = 50 mm
bar's diameter = 22 mm, As = $\pi/4xd^2$ = 380.28 mm^2
b = 500 mm

d = 627 mm
d' = 73 mm
ϕ = 0,9
M_{ul} = 13726,607 kgm= 137266071,2 Nmm
M_{ut} = 21962,571 kgm= 219625713,9 Nmm

<u>Flexural Bar</u>
due to M_{ul}

$$Rn = \frac{Mn}{b \times d^2} = 0{,}776 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}00202$$

$\rho_{need} < \rho_{min}$ = 0,00202 < 0,0035
use $\rho = \rho_{min}$ = 0,0035
$As_{need} = \rho x b x d$ = 1097,25 mm^2
n = As_{need}/As_{bar} = 1097,25 mm^2/380,28 mm^2 = 2,88
use n = 3 → $As_{use} = nxAs_{bar}$ = 1140,86 mm^2
$As_{use} > As_{need}$ = 1140,86 mm^2 > 1097,25 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 155 mm > 25 mm (OK), use reinforcement 3D22

<u>Flexural Bar</u>
due to M_{ut}

$$Rn = \frac{Mn}{b \times d^2} = 1{,}24 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}0032$$

$\rho_{need} < \rho_{min}$ = 0,0032 < 0,0035
use $\rho = \rho_{nmin}$ = 0,0035
$As_{need} = \rho x b x d$ = 1097,25 mm^2
n = As_{need}/As_{bar} = 1097,25 mm^2/380,28 mm^2 = 2,88

use n = 3 → $As_{use} = nxAs_{bar} = 1140,86\ mm^2$
$As_{use} > As_{need} = 1140,86\ mm^2 > 1097,25\ mm^2$ (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 155 mm > 25 mm (OK), use reinforcement 3D22

Stirrup Bar
Use stirrup bar ϕ 12 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu =21962,57 kg= 219625,7 N
Vs min = Vu/ϕ = 292834,26 N
A_s = 22/7 x 12 x 12/4 mm^2 = 113,14 mm^2
A_v = 2 x A_s = 226,28 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 116{,}28\ mm$$

$s_{maks} \leq d/2$ = 313,5 mm (SNI, 2013 paragraph 21.3.4.3)
Use the space of stirrup bar, s = 100 mm, (ϕ12-100)

Development Bar's Length
$\ell_{dh} = 8xd_b$ = 8x22 mm = 176 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_b x(f'_c)^{0,5}$ = 23,30 mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 180 mm with hook $4d_b$ = 88 mm ≈ 90 mm.

4.6.3.8. Checking The Deflection of Secondary Beam

The deflection of secondary beam is calculated using equation below
δ (deflection) = (5/384) x (q_u x L^4/EI)
$\delta_{max} = L_n/480$ (in cm) (SNI 2847:2002 Paragraph 9)

in which q_u is taken as 8785,029 kg/m (see previous paragraph), I is the moment of inertia of secondary beam (I = $1/12xbxh^3$), and E equals $4700x(f'c)^{0,5}$, f'c in 14 days
Deflection (L_n = 435 cm, b =50 cm, h = 70cm)
$\delta = (5/384) \times (q_u \times L^4/EI) = 0{,}112$ cm
$\delta_{max} = L_n/480 = 0{,}906$ cm
$\delta < \delta_{max}$ (OK)

4.6.3.9. Load Calculation (Uplift)

The load calculation for secondary beam will be shown below

1. dead load
   secondary beam BAB1 : 0,5x0,7x2400 kg/m= 840 kg/m

   q of slab BBA : 7154,8 kg/m^2 → due to uplift force

by using Ly = 500 cm – (65 cm/2 + 65 cm/2) = 435 cm and Lx = 250 cm – (65 cm/2 + 50 cm/2) = 192,5 cm, it is obtained the value of qeq due to slab's load

(two trapezoids load)
$q_{eq} = 2 \times q \times 1/2 \times Lx \times ((1 - 1/3 \times (Lx/Ly)^2)$
q= 7154,8 kg/m^2
q_{eq} = 12873,93 kg/m → due to slab's load
DL= 12873,93 kg/m + 840 kg/m= 13713,93 kg/m
DL (k, shock coefficient = 1,2) = 1,2x13713,93 kg/m
= 16456,71 kg/m

DL (total) : = 16456,71 kg/m

4.6.3.10. Load Combination of Secondary Beam

The combination of load for secondary beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of secondary beam
Qu = 1,2×16456,71 + 1,6x0 = 19748,06 kg/m

4.6.3.11. Moment Calculation of Secondary Beam

According to SNI (2013), the equation of moment values for secondary beamare shown below

Ml (+) = 1/16 x qx L^2→ at field area
Mt (-) = 1/10 x qx L^2 → at support area

moment calculation of secondary beam
(q = 19748,06 kg/m, L = 5 m)
Ml (+) = 30856,336 kgm
Mt(-) = 49370,138 kgm

The Mu values are taken as
Mul = 30856,336 kgm(+)
Mut = 49370,138 kgm (-)

4.6.3.12. Shear Force Calculation of Secondary Beam

The shear force calculation of secondary beam will use shear force equation Vu = 1/2 x q x L.
shear force calculation of secondary beam
(q = 19748,06 kg/m, L = 5 m)
Vu =49370,138 kg

4.6.3.13. Calculation of Reinforcement Bar

The calculation of reinforcement bar will be divided into two conditions (before monolith condition and after monolith condition). The general data for calculation of reinforcement bar of secondary beam is shown below

secondary beam's dimension (overall) = 500mm × 700mm
clear cover = 50 mm
bar's diameter (D) = 22 mm
bar's diameter/stirrup(ϕ/plain) = 12 mm
f'_c = 30 Mpa
f_y = 390 MPa
d' = clear cover + stirrup + D/2 = 73 mm
d = 700 - 50 - 12 - (22/2) = 627 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0,85 – 0,05\,\frac{(f'c-28)}{7} \geq 0,65$

$\beta_1 = 0,85 – 0,05\,\frac{(30-28)}{7} = 0,8357143 \geq 0,65$

$As = \pi/4xd^2 = 380,286\ mm^2$

$a = Asxf_y/(0,85xf'_cxb) = 11,63$ mm

$c = a/\beta_1 = 13,92$ mm

$\varepsilon_t = (d/c - 1)x0,003 = 0,132 \rightarrow \phi = 0,9$ (SNI, 2013 Figure S9.3.2)

$m = f_y/(0,85xf'_c) = 15,29$

$\rho_{min} = ¼ \text{ x } (f'_c)^{0,5}/f_y = 0,0035$

4.6.3.14. Reinforcement Bar

The general data for reinforcement bar of seconday beam after monolith condition will be shown below

secondary beam's dimension = 500x 700 mm^2

f'_c = 30 MPa (28 days)

clear cover = 50 mm

bar's diameter = 22 mm, $As = \pi/4xd^2 = 380,28\ mm^2$

b = 500 mm

d = 627 mm

d' = 71 mm

ϕ = 0,9

M_{ul} = 30856,336 kgm= 308563366,5Nmm

M_{ut} = 49370,138kgm = 493701386,4 Nmm

<u>Flexural Bar</u>

due to M_{ul}

$$Rn = \frac{Mn}{b \times d^2} = 1{,}73 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0{,}00460$$

$\rho_{need} > \rho_{min}$ = 0,00460 > 0,0035
use $\rho = \rho_{need}$ = 0,00460
$As_{need} = \rho x b x d$ = 1448,42 mm^2
n = As_{need}/As_{bar} = 1448,42 mm^2/380,28 mm^2 = 3,8
use n = 5 → $As_{use} = nxAs_{bar}$ = 1901,428 mm^2
$As_{use} > As_{need}$ = 1901,428 mm^2 > 1448,42 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 66,5 mm > 25 mm (OK), use reinforcement 5D22

Flexural Bar
due to M_{ut}

$$Rn = \frac{Mn}{b \times d^2} = 2,79 N/mm^2$$

$$\rho_{need} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0,0075$$

$\rho_{need} > \rho_{min}$ = 0,0075 > 0,0035
use $\rho = \rho_{need}$ = 0,0075
$As_{need} = \rho x b x d$ = 2381,6 mm^2
n = As_{need}/As_{bar} = 2381,6 mm^2/380,28 mm^2 = 6,2
use n = 7 → $As_{use} = nxAs_{bar}$ = 2662 mm^2
$As_{use} > As_{need}$ = 2662 mm^2 > 2381,6 mm^2 (OK)
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 37 mm > 25 mm (OK), use reinforcement 7D22

Stirrup Bar
Use stirrup bar ϕ 12 mm, f_y = 240 MPa, ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 49370,1 kg= 493701 N
Vs min = Vu/ϕ = 658268N
A_s = 22/7 x 12 x 12/4 mm^2 = 113,14 mm^2
A_v = 2 x A_s = 226,28 mm^2

$$s_{maks} = \frac{A_v \times f_y \times d}{V_s} = 51{,}72 \text{ mm}$$

$s_{maks} \leq d/2 = 313{,}5$ mm (SNI, 2013 paragraph 21.3.4.3)
Use the space of stirrup bar, s = 50 mm, (ϕ12-50)

<u>Development Bar's Length</u>
$\ell_{dh} = 8xd_b = 8x22$ mm = 176 mm
(SNI, 2013 paragraph 12.5.1)
ℓ_{dh} = 150 mm
(SNI, 2013 paragraph 12.5.1)
$\ell_{dh} = 0{,}24xfy/d_bx(f'_c)^{0,5} = 23{,}3$ mm
(SNI, 2013 paragraph 12.5.2)
So, the development bar's length will use ℓ_{dh} = 180 mm with hook $4d_b$ = 88 mm ≈ 90 mm.

4.6.3.15. Checking The Deflection of Secondary Beam

The deflection of secondary beam is calculated using equation below
δ (deflection) = (5/384) x (q_u x L^4/EI)
$\delta_{max} = L_n/480$ (in cm) (SNI 2847:2002 Paragraph 9)
in which q_u is taken as 19748,06 kg/m (see previous paragraph), I is the moment of inertia of secondary beam (I = $1/12xbxh^3$), and E equals $4700x(f'c)^{0,5}$, f'c in 14 days
Deflection (L_n = 435 cm, b = 50 cm, h = 70cm)
δ = (5/384) x (q_u x L^4/EI) = 0,25 cm
$\delta_{max} = L_n/480$ = 0,9065 cm
$\delta < \delta_{max}$ (OK).

For the recapitulation of reinforcement bar will be shown in Table 4.40.

Table 4.40. Recapitulation of Reinforcement Bars of Basement's Secondary Beam

Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar	
		As use (mm^2)	n	Code	As use (mm^2)	n	Code	s (mm)	Code
BAB1	4350	2662	7	7D22	1140,86	3	3D22	50	Φ12-50
		1901,43	5	5D22	1901,43	5	5D22	50	Φ12-50
BAB2	4150	2662	7	7D22	1140,86	3	3D22	50	Φ12-50
		1901,43	5	5D22	1901,43	5	5D22	50	Φ12-50

4.6.4. Reinforcement of Basement's Primary Beam

4.6.4.1. General Data

The general data for reinforcement of primary beam will be shown below

b (width)	= 65 cm
h (height)	= 80 cm
L(axis to axis)	= 5000mm
L(actual)	= 4200 mm
bar's diameter (D/deform)	= 25 mm
bar's diameter/stirrup(D/deform)	= 19 mm
f'_c	= 30 MPa
f_y	= 390 MPa

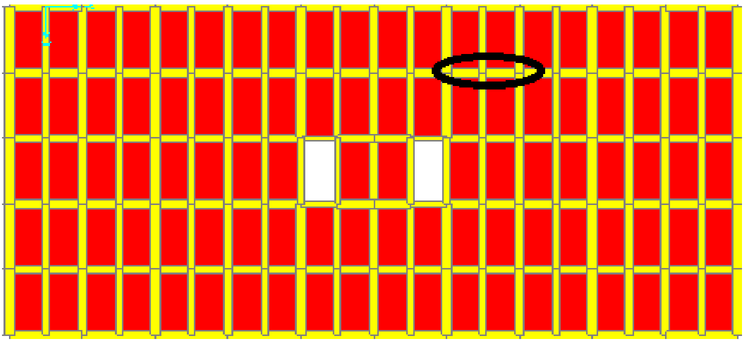


Figure 4.68. The location of basement's primary beam on SAP 2000 v14®
(source: Private Documentation)

4.6.4.2. Load Calculation

The load calculation for primary beam will be shown below

1. dead load

secondary beam BAB1 : 0,5x0,7x2400 kg/m= 840 kg/m

q of slab BBA : 7154,8 kg/m^2→ due to uplift force by using Ly = 500 cm – (65 cm/2 + 65 cm/2) = 435 cm and Lx = 250 cm – (65 cm/2 + 50 cm/2) = 192,5 cm, it is obtained the value of qeq due to slab's load

(two trapezoids load)
q_{eq} = 2 x q x 1/2 x Lx x ((1 -1/3 x $(Lx/Ly)^2$)
q= 7154,8 kg/m^2
q_{eq} = 12873,93 kg/m→ due to slab's load
DL= 12873,93 kg/m + 840 kg/m
= 13713,93kg/m

DL (total) : = 13713,93kg/m

2. dead load (primary beam)

secondary beam BIB1 : 0,65x0,8x2400 kg/m= 1248 kg/m

q of slab BBA : 7154,8 kg/m^2→ due to uplift force

by using Ly = 500 cm – (65 cm/2 + 65 cm/2) = 435 cm and Lx = 250 cm – (65 cm/2 + 50 cm/2) = 192,5 cm, it is obtained the value of qeq due to slab's load

(two trapezoids load)
q_{eq} = 2 x q x 1/2 x Lx x ((1 -1/3 x $(Lx/Ly)^2$)
q= 7154,8 kg/m^2
q_{eq} = 12873,93 kg/m→ due to slab's load
DL= 12873,93 kg/m + 1248 kg/m
= 14121,93 kg/m

DL (total) : = 14121,93 kg/m

4.6.4.3. Load Combination of Primary Beam

The combination of load for primary beam is based on SNI 2847:2013 paragraph 9.2.1. The load combination will use the ultimate load, Qu, as 1,2DL + 1,6LL.

load combination of primary beam
due to secondary beam
Qu = 1,2×13713,93+ 1,6x0 = 16456,72 kg/m
Pu = Qu x L = 82283,6 kg

due to primary beam
Qu = 1,2×14121,93+ 1,6x0= 16946,32 kg/m.

4.6.4.4. Moment Calculation of Primary Beam

The moment calculation of primary beam will use momentequation

Mu = 1/8 x q x L^2+ ¼ x P x L, in which L is the length of axis to axis of primary beam

moment calculation of primary beam
(q = 16946,32 kg/m, P = 82283,6 kg, L = 5 m)
Mu = 155811,75 kgm

The Mu values are taken as
Mu = 155811,75 kgm
So, the Mu value is 155811,75kgm.

Therefore, the moment calculation of primary beam will use moment results from SAP 2000 v14®.
M (at left support area) = -1037111748 Nmm
M (at right support area) = -1035241435 Nmm
M (at field area) = +544119344 Nmm

4.6.4.5. Shear Force Calculation of Primary Beam

The shear force calculation of primary beam will use shear force equation Vu = 1/2 x q x L + ½ x P
shear force calculation of primary beam
(q = 16946,32 kg/m, P = 82283,6 kg, L = 5 m)
Vu = 83507,6 kg

The Vu values are taken as
condition c
Vu =83507,6 kg
So, the Vu value is 83507,6kg.

Therefore, the shear force results from SAP 2000 v14® will be shown below
Vu (left) = 1218335,5 N, Vu (right) = 1215960,53 N

4.6.4.6. Calculation of Reinforcement Bar

The general data for calculation of reinforcement bar of primary beam is shown below
primary beam's dimension (overall) = 650mm × 800mm
clear cover = 50 mm
bar's diameter (D) = 25 mm
bar's diameter/stirrup(D/deform) = 19 mm
f'_c = 30 MPa
f_y = 390 MPa
d' = clear cover + stirrup + D/2 = 81,5 mm
d = 800 - 50 - 19 - (25/2) = 718,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)
$\beta_1 = 0,85 - 0,05\,\frac{(f'c-28)}{7} \geq 0,65$
$\beta_1 = 0,85 - 0,05\,\frac{(30-28)}{7} = 0,8357143 \geq 0,65$
As = $\pi/4 x d^2$ = 491,07 mm^2.

4.6.4.7. Reinforcement Bar (Flexural Bar)
<u>At Support Area (Left)</u>
(Assumed Primary Beam as Rectangular Shape)

The general data for reinforcement bar of primary beam will be shown below
Primary beam's dimension = 650x 800 mm^2
f'_c = 30 MPa (28 days)
clear cover = 50 mm

bar's diameter	= 25 mm, As = $\pi/4xd^2$ = 491,07 mm^2
b	= 650 mm
d	= 718,5 mm
d'	= 81,5 mm

$M_{u\ left}$ = -103711,17 kgm = -1037111748 Nmm

Flexural Bar

As (tension reinforcement) = 10D25

As = 4910,72 mm^2

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 29,11 mm > 25 mm (OK)

As' (compression reinforcement) = 10D25

As' = 4910,72 mm^2

s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 29,11 mm > 25 mm (OK)

Analyzing Double Reinforcement

$$\rho = \frac{As}{b_w \times d} = 0{,}011$$

$$\rho' = \frac{As'}{b_w \times d} = 0{,}011$$

$$\rho - \rho' \geq \frac{0{,}85 \times f'c \times \beta_1 \times d'}{f_y \times d} \times \frac{600}{600 - f_y}$$

$0 \leq 0{,}0176$ (compression reinforcement not yet yield)

Thus,

$$A_s f_y = 0{,}85 f_c' \beta_1 c \cdot b + A_s' \left[600 \left(\frac{c - d'}{c} \right) - 0{,}85 f_c' \right]$$

by square equation, it is obtained the value of c which is 64,62 mm.

So,
$a = \beta_1 \times c$
a = 53,96 mm

check the compression reinforcement's condition

$$f_s' = 600\left(\frac{c-d'}{c}\right) \leq f_y$$

f'_s = 156,64 MPa ≤ 390 MPa (compression reinforcement is in compressive condition)

Checking Nominal Moment

$$M_n = (As \times fy - A'_s \times f's) \times \left(d - \frac{a}{2}\right) + A's \times f's \times (d - d')$$

M_n = 1282443561 Nmm

$$\phi = 0{,}75 + 0{,}15\left[(1/(c/d)) - (5/3)\right] \leq 0{,}9$$

$$\phi = 2{,}42 \rightarrow \phi = 0{,}9$$

ϕM_n = 1154199205 Nmm > M_u = 1037111748 Nmm (OK)

At Support Area (Right)
(Assumed Primary Beam as Rectangular Shape)

The general data for reinforcement bar of primary beam will be shown below

Primary beam's dimension = 650x 800 mm^2
f'_c = 30 MPa (28 days)
clear cover = 50 mm
bar's diameter = 25 mm, As = $\pi/4 x d^2$ = 491,07 mm^2
b = 650 mm
d = 718,5 mm
d' = 81,5 mm
$M_{u\ left}$ = -103524,1 kgm = -1035241435 Nmm

<u>Flexural Bar</u>
As (tension reinforcement) = 10D25
As = 4910,72 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 29,11 mm > 25 mm (OK)

As’ (compression reinforcement) = 10D25
As’ = 4910,72 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 29,11 mm > 25 mm (OK)

<u>Analyzing Double Reinforcement</u>

$$\rho = \frac{As}{b_w \times d} = 0{,}011$$

$$\rho' = \frac{As'}{b_w \times d} = 0{,}011$$

$$\rho - \rho' \geq \frac{0{,}85 \times f'c \times \beta_1 \times d'}{f_y \times d} \times \frac{600}{600 - f_y}$$

$0 \leq 0{,}0176$ (compression reinforcement not yet yield)

Thus,

$$A_s f_y = 0{,}85 f_c' \beta_1 c \cdot b + A_s' \left[600 \left(\frac{c - d'}{c} \right) - 0{,}85 f_c' \right]$$

by square equation, it is obtained the value of c which is 64,62 mm.

So,
a = β_1 x c
a = 53,96 mm

check the compression reinforcement’s condition

$$f_s' = 600 \left(\frac{c - d'}{c} \right) \leq f_y$$

f'_s = 156,64 MPa ≤ 390 MPa (compression reinforcement is in compressive condition)

Checking Nominal Moment

$$M_n = \left(As \times fy - A'_s \times f's\right) \times \left(d - \frac{a}{2}\right) + A's \times f's \times \left(d - d'\right)$$

M_n = 1282443561 Nmm

$$\phi = 0,75 + 0,15\left[\left(1/\left(c/d\right)\right) - \left(5/3\right)\right] \leq 0,9$$

$$\phi = 2,42 \rightarrow \phi = 0,9$$

ϕM_n = 1154199205 Nmm > M_u = 1035241435 Nmm (OK)

At Field Area

(Assumed Primary Beam as Dummy T Beam)

The general data for reinforcement bar of primary beam will be shown below

Primary beam's dimension = 650x 800 mm^2

f'_c = 30 MPa (28 days)

clear cover = 50 mm

bar's diameter = 25 mm, As = $\pi/4xd^2$ = 491,07 mm^2

b = 650 mm

d = 718,5 mm

d' = 81,5 mm

M_u = +54411,93 kgm = +544119344Nmm

Effective Width (b_e)

be1 = ¼ x Lb = ¼ x 5000 = 1250 mm

be2 = bw + $16t_f$ = 400 + (16 x 200) = 3850 mm

be3 = ½ x (Lb – bw) = ½ x (5000 – 650) = 2175 mm

b = be = 1250 mm

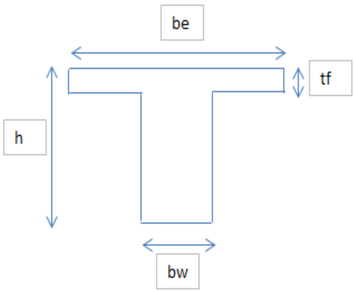


Figure 4.69. The dummy T beam dimension of basement's primary beam
(source: Private Documentation)

<u>Flexural Bar</u>
As (tension reinforcement) = 6D25
As = 2946,43 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 72,4 mm > 25 mm (OK)

As' (compression reinforcement) = 6D25
As' = 2946,43 mm^2
s = b – 2xclear cover - 2ϕ - n x D/(n - 1) = 72,4 mm > 25 mm (OK)

<u>Analyzing Double Reinforcement</u>

$$\rho = \frac{As}{b_w \times d} = 0{,}0063$$

$$\rho' = \frac{As'}{b_w \times d} = 0{,}0063$$

$$\rho - \rho' \geq \frac{0{,}85 \times f'c \times \beta_1 \times d'}{f_y \times d} \times \frac{600}{600 - f_y}$$

$$0 \leq 0{,}017 \text{ (compression reinforcement not yet yield)}$$

Thus,

$$A_s f_y = 0{,}85 f_c' \beta_1 c \cdot b + A_s' \left[600 \left(\frac{c - d'}{c} \right) - 0{,}85 f_c' \right]$$

by square equation, it is obtained the value of c which is 57,80 mm.

So,
$a = \beta_1 \times c$
a = 48,26 mm

check the compression reinforcement's condition

$$f_s' = 600 \left(\frac{c - d'}{c} \right) \leq f_y$$

f'_s = 246,01 MPa ≤ 390 MPa (compression reinforcement is in tension condition)

<u>Checking Nominal Moment</u>

$$M_n = \left(As \times fy - A'_s \times f's \right) \times \left(d - \frac{a}{2} \right) + A's \times f's \times \left(d - d' \right)$$

M_n = 756320686,2 Nmm

$$\phi = 0{,}75 + 0{,}15 \left[\left(1 / \left(c / d \right) \right) - \left(5 / 3 \right) \right] \leq 0{,}9$$

$$\phi = 2{,}62 \rightarrow \phi = 0{,}9$$

ϕM_n = 680688617,6 Nmm > M_u = 544119344 Nmm (OK)

<u>Checking Dummy T Beam</u>
$T = A_s \times f_y$
= 2946,43 mm^2 x 390 MPa
= 1149107,14 N

$C = 0{,}85 \times f'c \times b_e \times h_f$
= 0,85 x 30 MPa x 1250 mm x 200 mm
= 6375000 N

therefore C > T, then the beam is assumes as dummy T beam which treated like rectangular beam with b_e as its width,

$$a = \frac{As \times fy}{0{,}85 \times bw \times f'c} < \mathrm{t}$$

a = 69,32 mm < t = 200 mm (OK)

$$\phi Mn = \phi \times \mathrm{As} \times \mathrm{fy} \times \left(\mathrm{d} - \frac{\mathrm{a}}{2} \right)$$

$ØM_n$> Mu

Ø → 0,9

707220888Nmm >544119344 Nmm (OK).

Recapitulation of the reinforcement will be shown below

- at left support
  upper reinforcement = 10D25 (As = 4910,72 mm^2)
  lower reinforcement = 10D25 (As’ = 4910,72 mm^2)
- at right support
  upper reinforcement = 10D25 (As = 4910,72 mm^2)
  lower reinforcement = 10D25 (As’ = 4910,72 mm^2)
- at field area
  upper reinforcement = 6D25 (As’ = 2946,43 mm^2)
  lower reinforcement = 6D25 (As = 2946,43 mm^2)

4.6.4.8. Reinforcement Bar (Shear and Torsion)

<u>Reinforcement Bar Due to Shear Force</u>

The reinforcement bar due to shear force shall follow the conditions in SNI 2847:2013 paragraph 21.3.3. The equation for calculating shear force is shown below

$$V_u = \frac{M_{pr1} + M_{pr2}}{l_n} \pm \left(\frac{Qu \times l_n}{2} + \frac{Nu}{2} \right)$$

in which,

$$Mpr = As \times 1{,}25 \times fy \times \left(d - \frac{a}{2} \right)$$

$$a = \frac{As \times 1{,}25 \times fy}{0{,}85 \times f'c \times b}$$

the result of M_{pr} value will be shown in Table 4.41.

Table 4.41. Recapitulation of M_{pr} Value

Location			n bar	As (mm^2)	a (mm)	M_{pr} (Nmm)
At support area	Left	Upper	10	4910,72	144,43	1547185659
		Lower	10	4910,72	144,43	1547185659
	Right	Upper	10	4910,72	144,43	1547185659
		Lower	10	4910,72	144,43	1547185659

M_{pr1} = 1547185659 Nmm
M_{pr2} = 1547185659 Nmm
L_n = 5000 mm – (2x0,5x800) mm = 4200 mm.

Analyze Vu
analyzing left quake
$V_{u1} = M_{pr1} + M_{pr2}/(L_n) - (Q_u \times L_n/2) - (N_u/2)$
$V_{u2} = M_{pr1} + M_{pr2}/(L_n) + (Q_u \times L_n/2) + (N_u/2)$

in which,
Q_u = 16946 kg/m (see previously paragraph)
N_{uleft} = 26168 N (from SAP 2000 v14®)

thus,
V_{u1} = 367805,07 N
V_{u2} = 1105705,1 N

analyzing right quake
$V_{u1} = M_{pr1} + M_{pr2}/(L_n) – (Q_u \text{ x } L_n/2) – (N_u/2)$
$V_{u2} = M_{pr1} + M_{pr2}/(L_n) + (Q_u \text{ x } L_n/2) + (N_u/2)$

in which,
Q_u = 16946 kg/m (see previously paragraph)
N_{uright} = 22066 N (from SAP 2000 v14®)

thus,
V_{u1} = 369856,07 N
V_{u2} = 1103654,1 N

the value of V_u will be compared to V_u result from SAP 2000 v14®.

Table 4.42. Recapitulation of V_u Value

Location		V_u (N)	V_u (SAP200 v14®) (N)
At support area	Left	367805,07	1218335,5
		1105705,1	
	Right	369856,07	1215960,5
		1103654,1	

From Table 4.46 above, the Vu value which will be used for calculation is 1218335,5 N.

<u>Reinforcement Bar In Plastis Area</u>
checking condition:

1. $(M_{pr1} + M_{pr2})/L_n \geq 0,5 \text{ x Vu}$
   = (1547185659 + 1547185659)/4200 N
   ≥ 0,5 x 1218335,5 N
   = 736755,1 N ≥ 551827 N (OK)

2. $Nu \leq 0{,}25 \times A_g \times f'_c$
$= 26168$ N (from SAP 2000 v14®) ≤ 3900000 N (OK).

According to SNI 2847:2013 paragraph 21.5.4.2, the value of Vc is assumed as 0 if both conditions above fulfilled.

Thus,
$Vs = Vu/\phi$
$\phi = 0{,}75$ (SNI, 2013 paragraph 9.3.2.3)
$Vu = 1218335{,}5$ N
$Vs = 1624446{,}7$ N

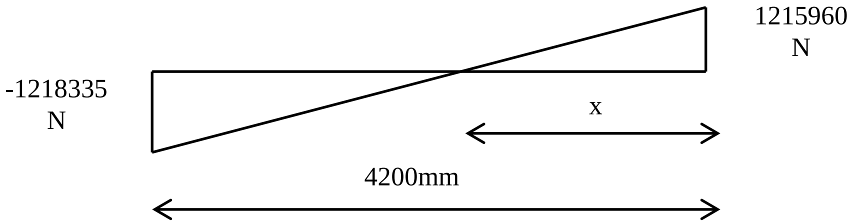
use 2D19 reinforcement bar ($Av = 567{,}28\ mm^2$),
$s = Av \times fy \times d/Vs$
$s = 567{,}28\ mm^2 \times 390\ MPa \times 718{,}5\ mm/1624446{,}7\ N$
$s = 97{,}5\ mm \approx 100\ mm$.

According to SNI 2847:2013 paragraph 21.3.4.2, the stirrup bar should be installed along 2h = 1600 mm at both edge of beam with the space of stirrup bar must not more than the least of

1. $d/4 = 718{,}5\ mm/4 = 179{,}63\ mm$
2. $8D = 8 \times 25\ mm = 200\ mm$
3. $24 \times \phi = 24 \times 19\ mm = 456\ mm$
4. 300 mm.

Use stirrup bar 2D19-100, s (space) = 100 mm $\leq$ 179,6 mm

<u>Reinforcement Bar Outside Plastis Area</u>
checking Vu value (from SAP 2000 v14®):



Equation for finding x value

$$\frac{x}{4200-x}=\frac{1215960}{1218335}$$

$$x = 2097 \text{ mm}$$

Vu at 2h = 1600 mm, is

$$\frac{2097}{2097-1600}=\frac{1215960}{Vu}$$

$$Vu = 288609{,}5 \text{ N}$$

Vu = 288609,5 N at 1600 mm from support.
Thus,
Vs = Vu/ϕ
ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)
Vu = 288609,5 N
Vs = 384812,7 N

use 2D19 reinforcement bar (Av = 567,28 mm^2),

s = Av x fy x d/Vs
s = 567,28 mm^2 x 390 MPa x 718,5 mm/384812,7 N
s = 413,1 mm ≈ 300 mm

According to SNI 2847:2013 paragraph 21.3.4.3, the stirrup bar should be installed along the beam (outside plastis area) with the space of stirrup bar must not more than

1. d/2 = 718,5 mm/2 = 359,3 mm.

Use stirrup bar 2D19-300, s (space) = 300 mm ≤ 359,3 mm.

n of stirrup bar (outside plastis area) = (L_n – 4h)/s + 1
n = (4200 mm – 4 x 800 mm)/300 mm + 1
n = 4,33 ≈ 5.

<u>Reinforcement Bar Due to Torsion Moment</u>

The reinforcement bar due to torsion moment shall follow the conditions in SNI 2847:2013 paragraph 11.5.1. The torsion can be neglected if the condition below fulfilled

$$T_u < \phi \frac{\sqrt{f'_c}}{12} \left(\frac{A^2{}_{cp}}{P_{cp}} \right)$$

in which,

ϕ = 0,75 (SNI, 2013 paragraph 9.3.2.3)

Tu = 25886055,6Nmm (from SAP 2000 v14®)

A_{cp} = b x h = 520000 mm^2

P_{cp} = 2 x (b + h) = 2900 mm

Thus,

Tu = 25886055,6 Nmm ≤ 31919004,2 Nmm (OK).

So, the torsion reinforcement bar is not needed.

4.6.4.9. Checking The Condition of Beam

<u>Checking The Deflection of Primary Beam</u>

The deflection of primary beam is calculated using equation below

δ (deflection) = (5/384) x (q_u x L^4/EI)

δ_{max} = L_n/480 (in cm) (SNI 2847:2002 Paragraph 9)

in which q_u is taken as 16946,32 kg/m (see previous paragraph), I is the moment of inertia of secondary beam (I = 1/12xbxh^3), and E equals 4700x$(f'c)^{0,5}$, f'c in 14 days

Deflection (L_n = 420 cm, b = 65 cm, h = 80 cm)

δ = (5/384) x (q_u x L^4/EI) = 0,096cm

δ_{max} = L_n/480 = 0,875 cm

$\delta < \delta_{max}$ (OK)

<u>Checking The Crack Moment</u>

According to SNI 2847:2013 paragraph 21.3.3, the crack moment, M_{cr}, must not more than nominal moment, ϕM_n, and the M_{cr} value shall follow the the equation in SNI 2847:2013 paragraph 9.5.3.2.

$\phi M_n \geq M_{cr}$

in which,

$M_{cr} = f_{cr} \times I_g/y_t$

$f_{cr} = 0{,}62\ \lambda\sqrt{f'c}$ /SF (SNI, 2013 paragraph 9.5.2.3)

$f_{cr} = 0{,}62 \times 1 \times (30^{0,5})/2 = 1{,}69$ MPa

$I_g = 1/12 \times b \times h^3$

$I_g = 1/12 \times 650 \times 800^3\ mm^4 = 2{,}773 \times 10^{10}\ mm^4$

$y_t = 650$ mm

Thus,

$M_{cr} = 72098000$ Nmm

$\phi M_n = 707220888$Nmm

$\phi M_n \geq M_{cr}$ (OK)

For the recapitulation of reinforcement bar will be shown in Table 4.43.

Table 4.43. Recapitulation of Reinforcement Bars of Basement's Primary Beam

Type	Length (mm)	Flexural Bar in support area			Flexural Bar in field area			Stirrup Bar		Additional
		As use (mm^2)	n	Code	As use (mm^2)	n	Code	s (mm)	Code	
BIB1	4200	4910,71	10	10D25	2946,43	6	6D25	100	2D19-100	2D25
		4910,71	10	10D25	2946,43	6	6D25	100	2D19-100	
BIB2	1600	4910,71	10	10D25	2946,43	6	6D25	100	2D19-100	2D25
		4910,71	10	10D25	2946,43	6	6D25	100	2D19-100	
BIB3	4200	4910,71	10	10D25	2946,43	6	6D25	100	2D19-100	2D22
		4910,71	10	10D25	2946,43	6	6D25	100	2D19-100	
BIB4	1600	2455,35	5	5D25	2455,35	5	5D25	100	2D19-100	2D25
		2455,35	5	5D25	2455,35	5	5D25	100	2D19-100	
BIB5	4200	2455,35	5	5D25	2455,35	5	5D25	100	2D19-100	2D25
		2455,35	5	5D25	2455,35	5	5D25	100	2D19-100	

4.7. Designing The Lower Structure
4.7.1. Designing The Foundation
4.7.1.1. Preface

The foundation funcionate as the loads bearer from upper structure and transform them soil. The foundation in this final project will use *Prestressed Concrete Spun Pile* from ICP PILES Co, Ltd. The data of pile will be shown below

foundation's type	: *Prestressed Concrete Spun Pile*
class	: C
effective prestress	: 7 N/mm^2
size (diameter)	: 60 cm
thickness	: 10 cm
cross section	: 157080 mm^2
bending moment	
crack	: 19,6 tm
break	: 36,5 tm
allowable compression	: 292 ton
length of pile	: 6-30 meter (three segmental)

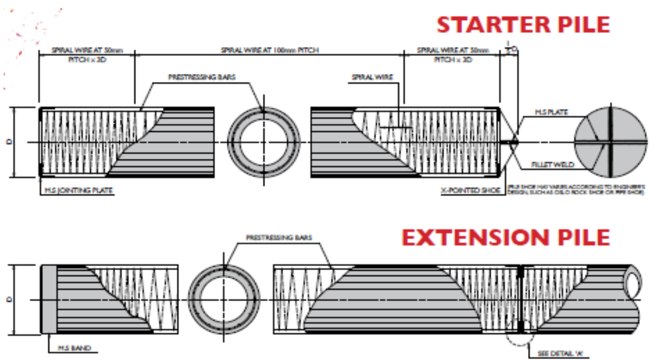


Figure 4.70. The Prestressed Concrete Spun Pile
(source: ICP PILES, Co., Ltd)

The soil data will use soil data in Kenjeran, Surabaya obtained from Soil and Rock Engineering Laboratory ITS Surabaya.

4.7.1.2. Bearing Capacity

<u>Bearing Capacity of Single Pile Foundation</u>

The bearing capacity of single pile foundation is determined by two factors, which are bearing capacity of pile (Q_p) and bearing capacity of soil (Q_s), so the total bearing capacity (Q_{total}) is stated as

$$Q_{total} = Q_p + Q_s$$

from Luciano Decourt (1982), the value of maximum bearing capacity of soil on single pile foundation (Q_L) is

$$Q_L = Q_P + Q_S$$

in which,

Q_L = max. bearing capacity of soil on pile foundation (ton)

Q_P = *resistance ultimate* at base of pile (ton)

Q_S = *resistance ultimate* due to soil cohesion/stickness (ton)

hence,

$Q_P = N_p \times K \times A_p$

$Q_S = (N_s/3 + 1) \times A_s$

in which,

N_p = the average value of N-SPT range at 4B above and 4B below pile foundation's base (unitless), B is diameter(meter)

K = coefficient of soil characteristic (ton/m^2)

= 12 ton/m^2 (clay)

= 20 ton/m^2 (silt clay)

= 25 ton/m^2 (sand clay)

= 40 ton/m^2 (sand)

A_p = the area of concrete of pile foundation's base (m^2)
N_s = the average value of N-SPT along its pile in soil (unitless), limited to $3 \leq N \leq 50$
A_s = the perimeter of pile multiplied by pile's depth (m^2)

note that the N-SPT value is corrected by equation proposed by Terzaghi & Peck, if the soil located below ground water level

$$N' = 15 + 0,5\,(N-15)$$

in which,
N' = N-SPT's corrected value (unitless)
N= N-SPT value, obtained from soil data (unitless)

Bearing Capacity of Group Pile Foundation

The bearing capacity of group pile foundation is obtained by multiplying bearing capacity of single pile foundation with efficiency factor, C_e, and the total number of pile, n,

$$Q_{L\,(group)} = Q_{L\,(single)} \times n \times C_e$$

hence, by equation proposed by Converse-Labarre,

$$C_e = 1 - \arctan\,(\phi/S)\left[\frac{(n-1)m+(m-1)n}{90\,mn}\right]$$

in which,
ϕ = diameter of pile (meter)
S = the distance between piles (meter)
m = the number of pile in one single row on pilecap
n = the number of pile in one single column on pilecap

<u>Repartition of Load on Group Pile Foundation</u>

If there's load which worked on the group pile of foundation (the group pile foundation is united by pilecap) whether vertical loads (V), horizontal loads (H), and moment force (M), then the value of equivalent vertical load (P_v) will be calculated by this equation proposed by Tomlison (1977)

$$P_{max} = \sum V/n \pm M_x\, y_{max}/\sum y_i^2 \pm M_y\, x_{max}/\sum x_i^2$$

in which,
P_{max} = the total of load on single pile in group pile foundation (ton)
$\sum V$ = total axial load (ton)
n = total number of pilecap
M_x = ultimate moment of X direction (tm)
y_{max} = maximum distance between furthest pile's axis and pilecap's neutral axis at Y direction (meter)
$\sum y_i^2$ = the sum of square of distance between pile's axis to pilecap's neutral axis at Y direction (m^2)
M_y = ultimate moment of Y direction (tm)
x_{max} = maximum distance between furthest pile's axis and pilecap's neutral axis at X direction (meter)
$\sum x_i^2$ = the sum of square of distance between pile's axis to pilecap's neutral axis at X direction (m^2)

note that value of x and y is positive if its direction same as e (eccentricity/neutral axis of pilecap), and vice versa.

4.7.1.3. Calculation of Pile Foundation's Bearing Capacity

<u>General Data</u>

The load which will be used for this calculation obtained from SAP 2000® programme, assumed that the designing of pillar's pile is typical for all pillar. Note that the combination of load will use 1,0 D + 1,0 L loads:

P	= 515,89 ton
M_{ux}	= 3,543 tm
M_{uy}	= 0,651 tm
H_x	= 1,29 ton
H_y	= 7,21 ton

Calculation of Single Pile Foundation's Bearing Capacity

The result of the calculation will use the data from Table 4.45. The calculation will use equation proposed by Luciano Decourt (1982)

$Q_L = Q_P + Q_S$

hence,

$Q_P = (N_p \text{ x } K) \text{ x } A_p$

$Q_P = (34 \text{ x } 25) \text{ x } 0{,}157080$ ton

$= 133{,}518$ ton

$Q_S = (N_s/3 + 1) \text{ x } A_s$

$= (6{,}887 + 1) \text{ x } (22/7 \text{ x } 0{,}6 \text{ x } 30)$ ton

$= 446{,}18$ ton

then,

$Q_L = 133{,}518 + 446{,}18 = 579{,}70$ ton (see Table 4.44)

Table 4.44. Recapitulation of Single Pile Foundation's Bearing Capacity Calculation

Depth (m)	N-SPT	N'-SPT	Soil Type*	K (t/m^2)	N_p	N_s	$N_s/3$	q_p (t/m^2)	Q_P (ton)	q_s (t/m^2)	Q_S (ton)	Q_L (ton)	Q_{ult} (SF=3) (ton)
0	0	0	LP	25	6	0	0	150	23,56	1	0	23,56	7,854
1	1	8	LP	25	8,4	4	1,33	210	32,98	2,33	4,4	37,38	12,46
2	1	8	LP	25	12	5,33	1,77	300	47,12	2,77	10,47	57,60	19,20
3	1	8	LL	20	6,67	6	2	133,3	20,94	3	16,97	37,91	12,63
4	1	8	LL	20	8	6,4	2,13	160	25,13	3,13	23,63	48,76	16,25
5	1	8	LP	25	8	6,66	2,22	200	31,41	3,22	30,38	61,79	20,59
6	1	8	LP	25	8	6,85	2,28	200	31,41	3,28	37,17	68,59	22,86
7	1	8	LP	25	8	7	2,33	200	31,41	3,33	44	75,41	25,13
8	1	8	LL	20	8	7,11	2,37	160	25,13	3,37	50,84	75,97	25,32
9	1	8	LL	20	8	7,2	2,4	160	25,13	3,4	57,70	82,83	27,61
10	1	8	LL	20	9	7,27	2,42	180	28,27	3,42	64,57	92,84	30,94
11	1	8	LL	20	10,78	7,33	2,44	215,71	33,88	3,44	71,44	105,33	35,11
12	1	8	LL	20	13	7,38	2,46	260	40,84	3,46	78,32	119,17	39,72
13	15	15	LL	20	15,28	7,92	2,64	305,71	48,02	3,64	89,30	137,32	45,77
14	26	20,5	LL	20	17,64	8,76	2,92	352,85	55,42	3,92	103,54	158,97	52,99
15	32	23,5	LL	20	20,07	9,68	3,22	401,42	63,05	4,23	119,62	182,68	60,89
16	33	24	LP	25	23,21	10,52	3,50	580,35	91,16	4,51	136,06	227,22	75,74
17	34	24,5	LP	25	25,71	11,30	3,76	642,85	100,98	4,77	152,86	253,84	84,61
18	35	25	LP	25	27,64	12,02	4,01	691,07	108,55	5,01	170,012	278,56	92,85

19	45	30	LP	25	29,28	12,92	4,30	732,14	115,01	5,30	190,19	305,19	101,73
20	50	32,5	LP	25	31	13,85	4,61	775	121,73	5,61	211,91	333,65	111,21
21	53	34	LP	25	32,85	14,77	4,92	821,42	129,03	5,92	234,6	363,63	121,21
22	55	35	LP	25	34,42	15,65	5,21	860,71	135,21	6,21	257,93	393,13	131,04
23	57	36	LP	25	35,14	16,5	5,5	878,57	138,06	6,5	281,91	419,92	139,97
24	60	37,5	LP	25	35,28	17,34	5,78	882,14	138,56	6,78	306,84	445,41	148,47
25	57	36	LP	25	35,28	18,05	6,01	882,14	138,56	7,01	330,90	469,47	156,49
26	55	35	LP	25	35,14	18,68	6,22	878,57	138,06	7,23	354,39	492,40	164,13
27	52	33,5	LP	25	34,9	19,21	6,40	873,21	137,16	7,40	377,01	514,17	171,39
28	53	34	LP	25	34,5	19,72	6,57	862,5	135,48	7,57	399,94	535,42	178,47
29	53	34	LP	25	34,2	20,2	6,73	855	134,30	7,73	422,90	557,20	185,73
30	54	34,5	LP	25	34	20,66	6,88	850	133,51	7,88	446,184	579,70	193,23

*LP = silt sand (lempung berpasir)
LL = sand clay (lempung berlanau)

Calculation of Group Pile Foundation's Bearing Capacity

The foundation will use pile foundation with 60 cm diameter. The distance (axis to axis) between piles is 180 cm. The distance between piles and pilecap's edge is 60 cm.

- distance between piles (S)

$2{,}5D \leq S \leq 3D$
$2{,}5 \times 60 \leq S \leq 3 \times 60$
$150 \text{ cm} \leq S \leq 180 \text{ cm}$
use S = 180 cm

- distance between piles and pilecap's edge (S_1)

$1D \leq S_1 \leq 2D$
$1 \times 60 \leq S_1 \leq 2 \times 60$
$60 \text{ cm} \leq S_1 \leq 120 \text{ cm}$
use S_1 = 60 cm

hence,

$$C_e = 1 - \arctan(\phi/S)\left[\frac{(n-1)m+(m-1)n}{90\,mn}\right]$$

in which,
ϕ = 0,6 m
S = 1,8 m
m = 2
n = 2
total number of piles = P/Q_L
= 579,70/193,23
= 2,67 ≈ 4 piles
(2x2, rectangular pilecap)

C_e = 0,79

then,

$$Q_{L\,(group)} = Q_{L\,(single)} \text{ x n x } C_e$$

in which,
$Q_{L\,(single)}$= 193,23 ton
n = 4
$Q_{L\,(group)}$ = 611,31 ton ≥ P = 515,89 ton (OK)

calculating P_{ult} value
P_{ult} = P + pilecap's weight
P = 515,89 ton
pilecap's weight = 3x3x1,5x2400 kg
= 32,4 ton
P_{ult} = 548,29 ton ≤ 611,31 ton (OK)

Checking P_{max} Value

The value of P_{max} will be calculated by this equation proposed by Tomlison (1977)

$$P_{max} = \sum V/n \pm M_x\, y_{max}/\sum y_i^2 \pm M_y\, x_{max}/\sum x_i^2$$

note that the moment on pilecap can cause compression or tension benhaviour. But since the compression loads are more considered than the tension load (because stucture's gravitational loads are greater than tension loads), so the equation of P_{max} is modified into

$$P_{max} = \sum V/n + M_x\, y_{max}/\sum y_i^2 + M_y\, x_{max}/\sum x_i^2$$

in which,
M_{ux} = 3,543 tm
M_{uy} = 0,651 tm
H_x = 1,29 ton
H_y = 7,21 ton

$\sum V = 515,89 + 32,4 = 548,29$ ton
$n = 4$
$M_x = M_{ux} + (H_y$ x pilecap's thickness)
$= 3,543 + (7,21 \times 1,5) = 14,358$ tm
$y_{max} = 0,9$ m
$\sum y_i^2 = 4 \times 0,9^2 = 3,24$ m^2
$M_y = M_{uy} + (H_x$ x pilecap's thickness)
$= 0,651 + (1,29 \times 1,5) = 2,586$ tm
$x_{max} = 0,9$ m
$\sum x_i^2 = 4 \times 0,9^2 = 3,24$ m^2
$P_{max} = 141,78$ ton $\leq Q_{L\,(single)} = 193,23$ ton (OK)

Checking Pile's Condition

The pile's conditions which will be checked are checking the axial load and checking the bending crackmoment

Checking Axial Load

$P_{max} \leq$ allowable compresion of pile
141,78 ton $\leq$ 292 ton (OK)

Checking Bending Crack Moment

by using equation proposed by Philiponat, the moment due to lateral load is obtained by multiplying L_e with H_y at Y direction and H_x at X direction. Note that L_e is 10D (for multilayer soil)

at X direction
$L_e = 10 \times D$ meter
$L_e = 10 \times 0,6 = 6$ meter
$M_x = L_e \times H_x$
$= 6$ m x $1,29$ ton $= 7,74$ tm
M_x of single pile $= M_x/n$ $(n = 4)$
$= 1,935$ tm $< M_{bending\ crack} = 19,6$ tm (OK)

at Y direction
L_e = 10 x D meter
L_e = 10 x 0,6 = 6 meter
$M_y = L_e \times H_y$
= 6 m x 7,21 ton = 43,26 tm
M_y of single pile = M_y/n (n = 4)
= 10,815 tm < $M_{bending\ crack}$ = 19,6 tm (OK).

4.7.1.4. Reinforcement of Pilecap

General Data

The general data for calculation of reinforcement of pillar's pilecap will be shown below

pilecap's dimension	= 3000 mm ×3000 mm
pilecap's thickness	= 1500 mm
clear cover	= 50 mm
bar's diameter (D)	= 25 mm
f'_c	= 30 MPa
f_y	= 390 MPa
pillar's dimension	= 800 mm x 800 mm
dx = 1500 - 50 - (25/2)	= 1437,5 mm
dy = 1500 - 50 - 25 - (25/2)	= 1412,5 mm

For $\beta_1$value is calculated by using condition in SNI (2013) paragraph 10.2.7.3. (f'c in 28days = 30 MPa, see Table 4.7)

$\beta_1 = 0,85 - 0,05\, \frac{(f'c-28)}{7} \geq 0,65$

$\beta_1 = 0,85 - 0,05\, \frac{(30-28)}{7} = 0,8357143 \geq 0,65$

As = $\pi/4xd^2$ = 132,78 mm^2
ϕ = 0,9 (SNI, 2013 Figure S9.3.2)
m = $f_y/(0,85xf'_c)$ = 15,29
ρ_b= 0,85 x β_1 xf'_c/f_y x (600/(600 + f_y))
ρ_b = 0,0331
ρ_{max} = 0,75 ρ_b
ρ_{max} = 0,0,025
ρ_{min} = 0,25 x $(f'_c{}^{0,5})/f_y$
ρ_{min}= 0,0035

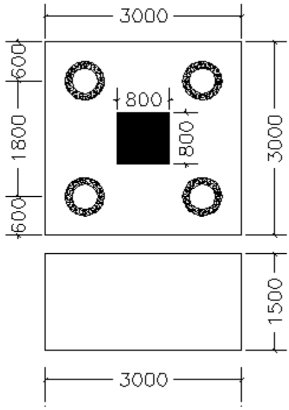


Figure 4.71. Dimension of pilecap
(source: Private Documentation)

Calculation of Reinforcement Bar

The general data for reinforcement bar of slab will be shown below (note that the calculation will use static mechanic principal)

slab's thickness	= 1500 mm
clear cover	= 50 mm
bar's diameter	= 25 mm, As = $\pi/4 x d^2$ = 491,07 mm^2
b	= 3000 mm
d_x	= 1437,5 mm
d_y	= 1412,5 mm
ϕ	= 0,9
q_u (pilecap's weight per m)	= 3 x 1,5 x 2400 kg/m
	= 10800 kg/m
P_{max}	= 141,78 ton
$P_t = 2 \times P_{max}$	= 283558,3 kg

a_x (distance of furthest pile to pillar, X direction) = 0,9 m
b_x (distance of furthest pile to pilecap, X direction) = 0,6 m
a_y (distance of furthest pile to pillar, Y direction) = 0,9 m
b_y (distance of furthest pile to pilecap, Y direction) = 0,6 m
$M_{ux} = (P_t \text{ x a}) – (0,5 \text{ x } q_u \text{ x } (a_x + b_x)^2)) = 243052,5$ kgm
$M_{uy} = (P_t \text{ x a}) – (0,5 \text{ x } q_u \text{ x } (a_y + b_y)^2)) = 243052,5$ kgm

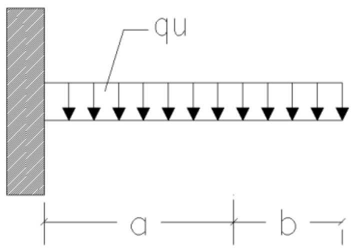


Figure 4.72. Analyzing of pilecap's calculation
(source: Private Documentation)

Reinforcement Bar (X direction)
$M_{nx} = M_{ux}/\phi = 270058,33$ kgm
b = 3000 mm
$d_x = 1437,5$ mm

$$\text{Rn} = \frac{\text{Mn}}{\text{b} \times d^2} = 0,435 N/mm^2$$

$$\rho_{\text{need}} = \frac{1}{\text{m}}\left(1 - \sqrt{1 - \frac{2 \times \text{m} \times \text{Rn}}{\text{fy}}}\right) = 0,0011$$

$\rho_{need} < \rho_{min} = 0,0011 < 0,0035$
use $\rho = \rho_{min} = 0,0035$
$As_{need} = \rho x b x d = 15141,37 \text{ mm}^2$
$n = As_{need}/As_{bar} = 15141,37 \text{ mm}^2/491,07 \text{ mm}^2 = 30,833$
use n = 32 → $As_{use} = nxAs_{bar} = 15714,29 \text{ mm}^2$
$As_{use} > As_{need} = 15714,29 \text{ mm}^2 > 15141,37 \text{ mm}^2$ (OK)
s = b/n = 3000 mm/32 = 93,75 mm
So, the transversal bar will use D25-90.

Reinforcement Bar (Y direction)
$M_{ny} = M_{uy}/\phi = 270058,33$ kgm
b = 3000 mm
$d_y = 1412,5$ mm

$$\mathrm{Rn} = \frac{\mathrm{Mn}}{\mathrm{b} \times d^2} = 0,4511 N/mm^2$$

$$\rho_{need} = \frac{1}{\mathrm{m}}\left(1 - \sqrt{1 - \frac{2 \times \mathrm{m} \times \mathrm{Rn}}{\mathrm{fy}}}\right) = 0,001167$$

$\rho_{need} < \rho_{min} = 0,001167 < 0,0035$
use $\rho = \rho_{min} = 0,0035$
$As_{need} = \rho x b x d = 14878,04\ mm^2$
$n = As_{need}/As_{bar} = 14878,04\ mm^2/491,07\ mm^2 = 30,29$
use n = 32 → $As_{use} = nxAs_{bar} = 15714,29\ mm^2$
$As_{use} > As_{need} = 15714,29\ mm^2 > 14878,04\ mm^2$ (OK)
s = b/n = 3000 mm/32 = 93,75 mm
So, the transversal bar will use D25-90.

Checking The Pons Shear Force of Pilecap

The designing process of pilecap's thickness, it is important to checking the pons shear force in which its condition $\phi V_c \geq P_{max}$. According to SNI 2847:2013 paragraph 11.11.2.1, the value of V_c is taken as the least value of these below

$$Vc = 0,17 \times \left(1 + \frac{2}{\beta}\right)\lambda\sqrt{f'c} \times b_o \times d$$

$$Vc = 0,083 \times \left(\frac{\alpha_s \times d}{b_o}\right)\lambda\sqrt{f'c} \times b_o \times d$$

$$Vc = 0,33\lambda \times \sqrt{f'_c} \times b_o \times d$$

in which,
β = ratio between length and width
= 3000/3000 = 1
b_o = perimeter of critical section on pilecap
$b_o = 2 \times (b_k + d) + 2 \times (h_k + d)$
b_k = 800 mm (pillar's length)
h_k = 800 mm (pillar's width)
d = 1437,5 mm (effective thickness of pilecap)
b_o = 8950 mm
α_s = 40 (interior pillar)
λ = 1 (normal concrete)

thus,
V_{c1} = 35938644,45 N
V_{c2} = 37576334,89 N
V_{c3} = 23254417 N

use $V_c = V_{c3}$ = 23254417 N
$\phi V_c \geq P_{max}$
0,75 x 23254417 N = 17440812,75 N ≥ 515,89 ton
17440812,75 N ≥ 5158900 N (OK)

<u>Checking Deep Beam Condition</u>

The deep beam condition should follow condition in SNI 2847:2013 paragraph 10.7, 11.7, and 12.10.6.

- L_n pilecap = 3000 mm ≤ 4h = 6000 mm (OK)

<u>Checking Deep Beam (X direction)</u>
$V_u \leq \phi \times 0{,}83 \times b_w \times d \times (f'_c{}^{0,5})$
(SNI 2847:2013 paragraph 11.7)

5482900 N $\leq \phi \times 0{,}83 \times b_w \times d_x \times (f'_c{}^{0,5})$
5482900 N $\leq 0{,}75 \times 0{,}83 \times 3000 \times 1437{,}5 \times (30^{0,5})$
5482900 N ≤ 14703783 N (OK)

$A_v > 0{,}0025 \times b_w \times S$
(SNI 2847:2013 paragraph 11.7.4.1)

in which,
$S \leq d/5$ or $S \leq 300$ mm
$S \leq 287{,}5$ mm
$S \leq 300$ mm
use s = 287,5 mm

$A_v = 15141{,}37 \text{ mm}^2 > 2156{,}25 \text{ mm}^2$ (OK)

<u>Checking Deep Beam (Y direction)</u>
$V_u \leq \phi \times 0{,}83 \times b_w \times d \times (f'_c{}^{0,5})$
(SNI 2847:2013 paragraph 11.7)
$5482900 \text{ N} \leq \phi \times 0{,}83 \times b_w \times d_x \times (f'_c{}^{0,5})$
$5482900 \text{ N} \leq 0{,}75 \times 0{,}83 \times 3000 \times 1412{,}5 \times (30^{0,5})$
$5482900 \text{ N} \leq 14448065 \text{ N}$ (OK)

$A_v > 0{,}0025 \times b_w \times S$
(SNI 2847:2013 paragraph 11.7.4.1)

in which,
$S \leq d/5$ or $S \leq 300$ mm
$S \leq 287{,}5$ mm
$S \leq 300$ mm
use s = 287,5 mm

$A_v = 14878{,}04 \text{ mm}^2 > 2118{,}75 \text{ mm}^2$ (OK)

4.7.2. Designing The Tie Beam

4.7.2.1. Preface

The tie beam funcionate as the connector between pilecaps and as stiffener. The loads which will be retained by the tie beam are its own weight, walls, and tension or compression axial force which obtained by 10% value of pillar's axial load.

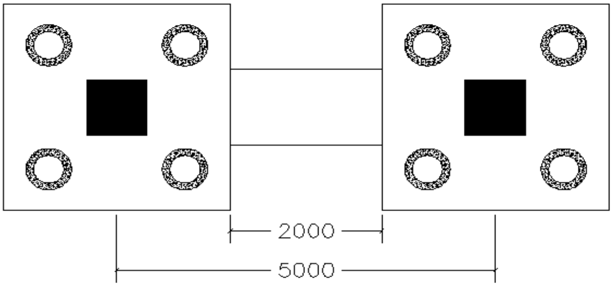


Figure 4.73. The tie beam's dimension
(source: Private Documentation)

4.7.2.2. Reinforcement of Tie Beam

General Data

The general data of tie beam will be shown below

length of tie beam, L = 2000 mm
clear cover = 50 mm
bar's diameter = 25 mm, As = $\pi/4 x d^2$ = 491,07 mm^2
stirrup (D/deform) = 22 mm, As = 380,28 mm^2
b = 800 mm
h = 1100 mm
d = 1100 - 50 - 22 - (25/2) = 1015,5 mm
f'_c = 30 MPa
f_y = 390 MPa
P_u = 515,89 ton
N_u = 10% x P_u = 51,589 ton

Calculation of Reinforcement Bar

The reinforcement bar (flexural bar) of tie beam will retain the loads which are axial load and flexural load. The reinforcement of tie beam (flexural bar) will be assumed as reinforcement of pillar. The loads which will be retained by tie beam are

axial load N_u = 10% x P_u = 51,589 ton
tie beam = 0,8 x 1,1 x 2,4 t/m = 2,112 t/m
brick's wall = 4 x 0,25 t/m = 1 t/m
Q_u = 1,2 x (2,112 + 1) t/m = 3,734 t/m
M_u = 1/12 x Q_u x L^2 (continuous beam)
M_u = 1,2448 tm = 12,448 kNm

by using PCACOL programme, the needed ammount of flexural bar is obtained

P_u = 515,89 kNm
M_u = 12,448 kNm

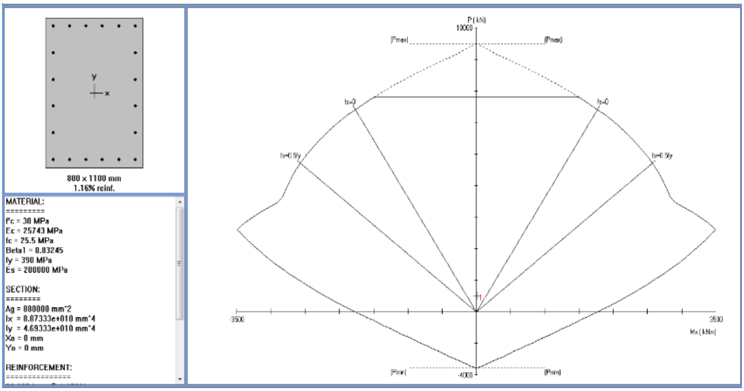


Figure 4.74. The interaction graphic between axial force and moment in tie beam
(source: Private Documentation)

from interaction diagram, it is obtained
ρ (ratio) = 1,159% with 20D25 reinforcement bar
use 20D25 (A_s = 10200 mm^2).

Calculation of Reinforcement Bar (Shear)

From interaction graphic, the M_{pr} value is 2221,36 kNm

$V_u = (M_{pr} + M_{pr})/L_n$

$= (2221,36 + 2221,36)$ kNm/2 meter

$= 2221,36$ kN $= 2221360$ N

$V_c = 1/6 \times (f'_c{}^{0,5}) \times b_w \times d \times (1 + N_u/(14\ A_g))$

$= 1/6 \times (30^{0,5}) \times 800 \times 1015,5 \times (1 + 515890/(14 \times 800 \times 1100)$ N $= 772670,93$ N

$= 772670,93$ N

$\phi\ V_c = 0,75 \times 772670,93$ N $= 579503,19$ N

$0,5\ \phi\ V_c = 0,5 \times 579503,19$ N $= 289751,6$ N

V_s min $= V_u/\phi$

$= 2221360$ N/0,75

$= 2961813,3$ N

$A_v = 2 \times A_s = 760,57\ mm^2$

$s = A_v \times f_y \times d/V_s$

$= 760,57 \times 390 \times 1015,5/2961813,3$ mm

$= 101,7$ mm ≈ 100 mm

$s \leq d/2$

100 mm $\leq$ 1015,5/2 mm

100 mm $\leq$ 507,75 mm (OK)

use stirrup D22-100.

Checking Deep Beam Condition

The deep beam condition should follow condition in SNI 2847:2013 paragraph 10.7, 11.7, and 12.10.6.

- L_n tie beam = 2000 mm $\leq$ 4h = 4400 mm (OK)

Checking Deep Beam

$V_u \leq \phi \times 0,83 \times b_w \times d \times (f'_c{}^{0,5})$

(SNI 2847:2013 paragraph 11.7)

$2221360\ N \leq \phi \times 0,83 \times b_w \times d_x \times (f'_c{}^{0,5})$

$2221360 \text{ N} \leq 0{,}75 \times 0{,}83 \times 800 \times 1015{,}5 \times (30^{0,5})$
$2221360 \text{ N} \leq 2769937 \text{ N}$ (OK)

$A_v > 0{,}0025 \times b_w \times S$
(SNI 2847:2013 paragraph 11.7.4.1)

in which,
$S \leq d/5$ or $S \leq 300$ mm
$S \leq 203{,}1$ mm
$S \leq 300$ mm
use $s = 203{,}1$ mm

$A_v = 760{,}57 \text{ mm}^2 > 406{,}2 \text{ mm}^2$ (OK).

4.8. CONSTRUCTION METHOD

4.8.1. Construcion Method of Precast Concrete Element

4.8.1.1. Preface

The construction method ofthis project will be explained in general (not spesified). It includes prefabricated of precast concrete element, transportation and storage of precast concrete element, location and capacity of tower crane, and erection of precast concrete element.

4.8.1.2. Prefabricated of Precast Concrete Element

The prefabricated of precast concrete element is important especially about the compressive strength of precast concrete element, f'_c, and the reinforcement bar, f_y. The precast concrete is casted in concrete mixing plant factory with strict quality control.

These conditions below have to be noted:

1. precast concrete material such as sand, water, gravel, and cement must be checked according to mix design plan
2. the formwork shall be checked before casting process
3. the curing process of concrete shall be done correctly

4. the precast concrete's age shall be checked
5. good management and good factory condition will help much more in order to produce good quality of precast concrete.

4.8.1.3. Transportation of Precast Concrete Element

The tansportation of precast concrete element will use trailer truck. The dimension of trailer truck will be shown below (trailer's length is approx. 40 meter, width 2,6 meter,maximum capacity 80 ton)

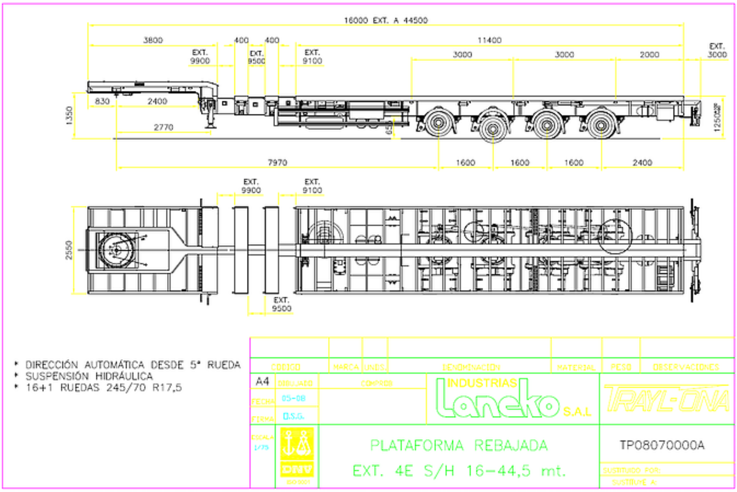


Figure 4.75. The dimension of trailer
(source: Laneko S.A.L, Co., Ltd)

note that the transportation process of precast concrete element from concrete plant factory will use mobile crane or crane. Then it is transported by truck trailer to project's location. On project's location the precast concrete element will be lifted by crane. Note that the storage place for precast concrete elements should in dry-chemical alkali free place, and stored in proper position.

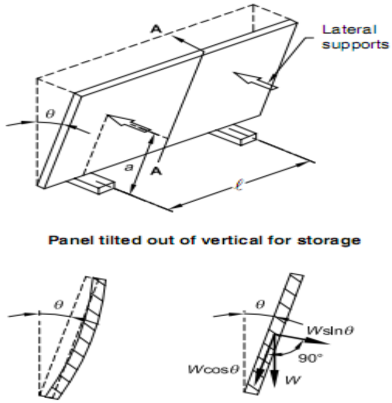


Figure 4.76. The proper position of storage of precast concrete element (source: PCI, 2004)

4.8.1.4. Capacity of Tower Crane

The tower crane which will be used is TowerCrane XMCG XC400. The spesification of tower crane will be shown in Figure 4.77.

XCP400

起重特性 Load diagrams

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~15.96	20	25	30	35	40	45	50	55	60	65	70	75	80
80	IV	起重量(t) Load	20	15.28	11.66	9.3	7.64	6.41	5.46	4.71	4.1	3.59	3.16	2.79	2.48	2.2
	II		10			9.96	8.33	7.13	6.2	5.46	4.86	4.36	3.94	3.58	3.27	3.0

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.5~20.38	25	30	35	40	45	50	55	60	65	70
70	IV	起重量(t) Load	20	15.74	12.67	10.51	8.91	7.68	6.69	5.9	5.23	4.68	4.2
	II		10				9.65	8.43	7.46	6.67	6.02	5.47	5.0

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~20.83	25	30	35	40	45	50	55	60
60	IV	起重量(t) Load	20	16.15	13	10.8	9.16	7.89	6.89	6.08	5.4
	II		10				9.92	8.67	7.68	6.87	6.2

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~21.51	25	30	35	40	45	50
50	IV	起重量(t) Load	20	16.77	13.52	11.24	9.54	8.24	7.2
	II		10					9.03	8.0

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~22.84	25	30	35	40
40	IV	起重量(t) Load	20	18.01	14.54	12.11	10.3
	II		10				

Figure 4.77. The capacity of tower crane
(source: XCMG, Co., Ltd)

The spesification of tower crane

- type: Tower Crane Xcmg XCP400
- maximum range: 80 m
- maximum load: 20 ton
- furthest range on project’s site: 50 m
- maximum load (at 50 meter): 4,7 ton

checking the capacity of tower crane

1. precast concrete element: slab

maximumload: 5 m x 2,5 m x 0,1 m x 2400 kg/m^3
= 3 ton ≤ 4,7 ton (OK)

2. precast concrete element: secondary beam
maximum load: 5 m x 0,3 m x 0,45 m x 2400 kg/m^3
= 1,62 ton ≤ 4,7 ton (OK)

3. precast concrete element: primary beam
maximum load: 5 m x 0,4 m x 0,65 m x 2400 kg/m^3
= 3,12 ton ≤ 4,7 ton (OK).

4.8.1.5. Erection of Precast Concrete Element

Casting of Pilar

After the installation of spun pile foundation and casting of pilecap-sloof, then the installation of pillar's reinforcement bar along its console should be done. After installing the formwork for pillar, the casting process of pillar could be done.

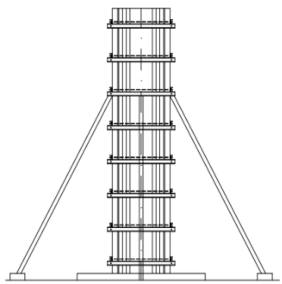


Figure 4.78. The formwork of pillar
(source: Private Documentation)

Erection of Precast Concrete Primary Beam

After the casting process of pillar, then the primary beam is installed by putting it to pillar's console. After that, the *overtopping* concrete is casted over the connection. Note that the *scaffolding* is needed as temporary support for precast concrete primary beam.

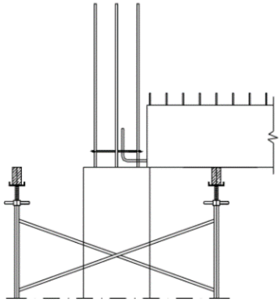


Figure 4.79. The erection of precast concrete primary beam (source: Private Documentation)

Erection of Precast Concrete Secondary Beam

The precast concrete secondary beam is connected to precast concrete primary beam by console. After that, the *overtopping* concrete is casted over the connection. Note that the *scaffolding* is needed as temporary support for precast concrete secondary beam.

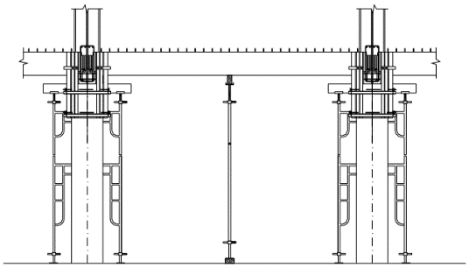


Figure 4.80. The erection of precast concrete secondary beam (source: Private Documentation)

Erection of Precast Concrete Slab

The precast concrete slab is lifted by tower crane, then put on precast beam. After that, the *overtopping* concrete is casted over the connection (5 cm *overtopping* concrete).

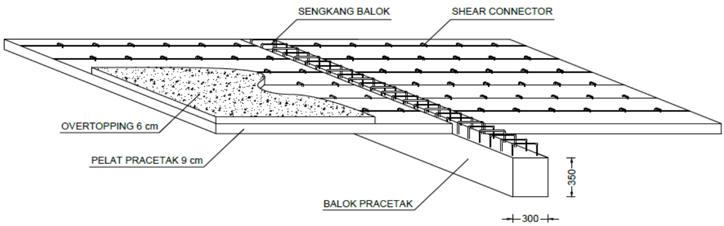


Figure 4.81. The erection of precast concrete slab
(source: Private Documentation)

4.8.2. Construcion Method of Basement Structure

The construction method of basement will use *bottom up* method. Mote that the basement's structure will use cast-in place concrete. The construction method of basement will be explained below

- installation of pile foundation
- installation of sheet pile
- excavation
- dewatering
- casting of pilecap
- casting of tie beam& raft foundation
- casting of basement's wall, slab, etc.

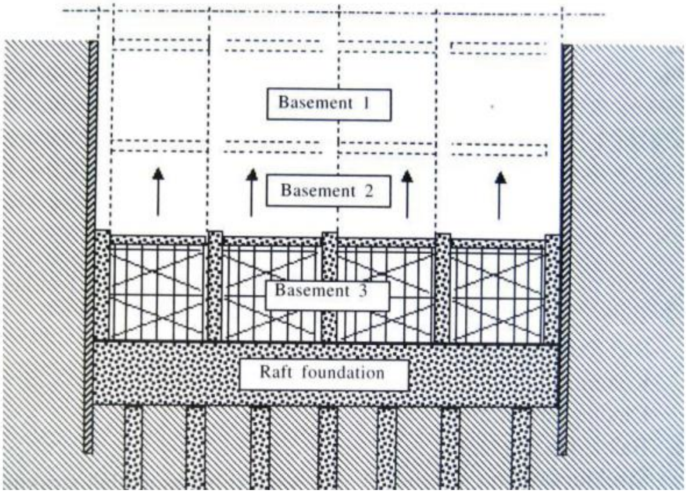


Figure 4.82. The construction method of basement
(source: *http://dodybrahmantyo.dosen.narotama.ac.id/*)

the dewatering process will use *cut off* method in which the diaphragm will use *sheet pile*, *concrete diaphragm wall*, or *concrete secant pile.*

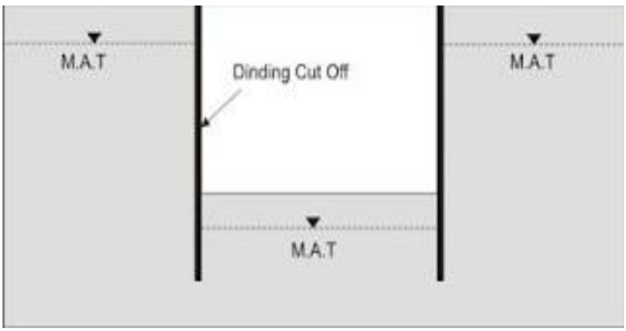


Figure 4.83. The section cut of *cutoff* method
(source: Private Documentation)

CHAPTER V
CONCLUSION

5.1. Conclusion

Based on this final project titled "Redesigning The Building's Structure of Hotel Novotel THE SAMATOR Surabaya by Using Precast Concrete", the conclusions can be described through these points

1. According to SNI 2847:2013, it is obtained these structure's dimension
   a. secondary structure
      - dimension of precast secondary beam:
        b = 30 cm, h = 45 cm
      - dimension of bordes beam (stair):
        b = 30 cm, h = 60 cm
      - dimension of elevator beam:
        b = 20 cm, h= 65 cm
      - precast slab's thickness:
        h = 10 cm, overtopping 5 cm
      - stair structure:
        slab, h = 20 cm, bordes, h = 20 cm
      - ramp:
        h = 30 cm
      - dimension of bordes beam (ramp):
        b = 40 cm, h = 80 cm

   b. primary structure
      - dimension of precast primary beam:
        b = 40 cm, h = 65 cm
      - dimension of pillar:
        b x b = 80 x 80 cm^2
      - shear wall's thickness:
        h = 40 cm

- dimension of pile foundation:
  D = 60 cm, L = 30 m (three segmetals)
- dimension of pilecap (pillar):
  p x l x h = 3 x 3 x 1,5 m^3

c. basement structure
- dimension of basement's secondary beam:
  b = 50 cm, h = 70 cm
- dimension of basement's primary beam:
  b = 65 cm, h = 80 cm
- slab's thickness:
  h = 20 cm
- basement wall's thickness:
  h = 30 cm

2. The connection of precast concrete elements will use wet connection. Note that the dimension of short console on pillar is 500 x 400 mm^2 while the dimension of short console on primary beam 300 x 200 mm^2.
3. The connection detailing, pile foundation, pilecap, and other structure will be shown as technical drawing on enclosure.

5.2. Advices

Advice(s) for this final project might be described as follow

1. the method of casting process of precast concrete element is essential and should be supervised with strict quality control
2. the connection between precast concrete elements should be done as simple as possible so that the erection process can be done correctly and accordingly
3. the research of precast concrete method is needed in order to gain more efficiency in construction process

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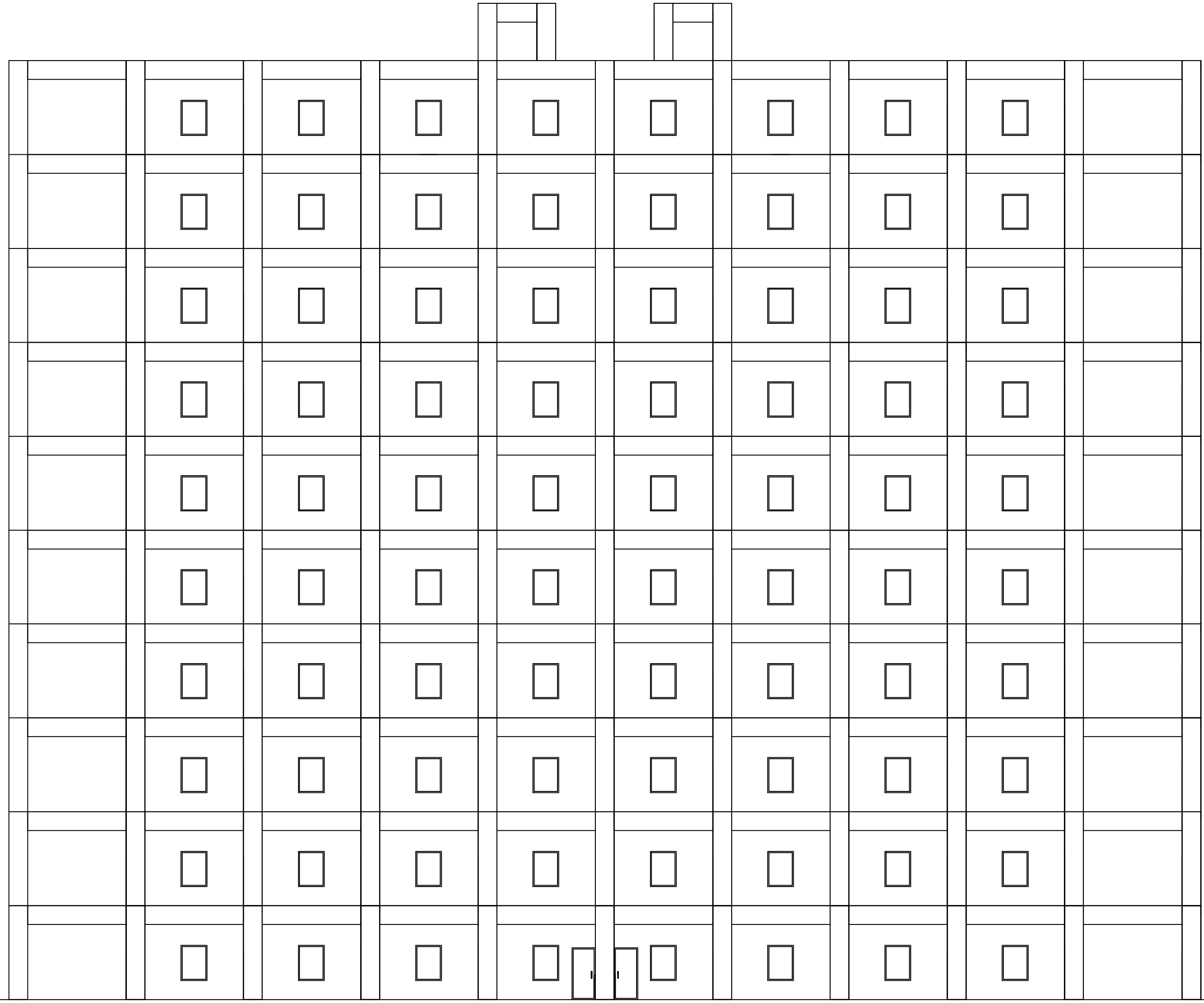
ENCLOSURES



FRONT SIDE OF BUILDING
SCALE 1:100

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2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:100

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

3112100025

NUMBER	TOTAL
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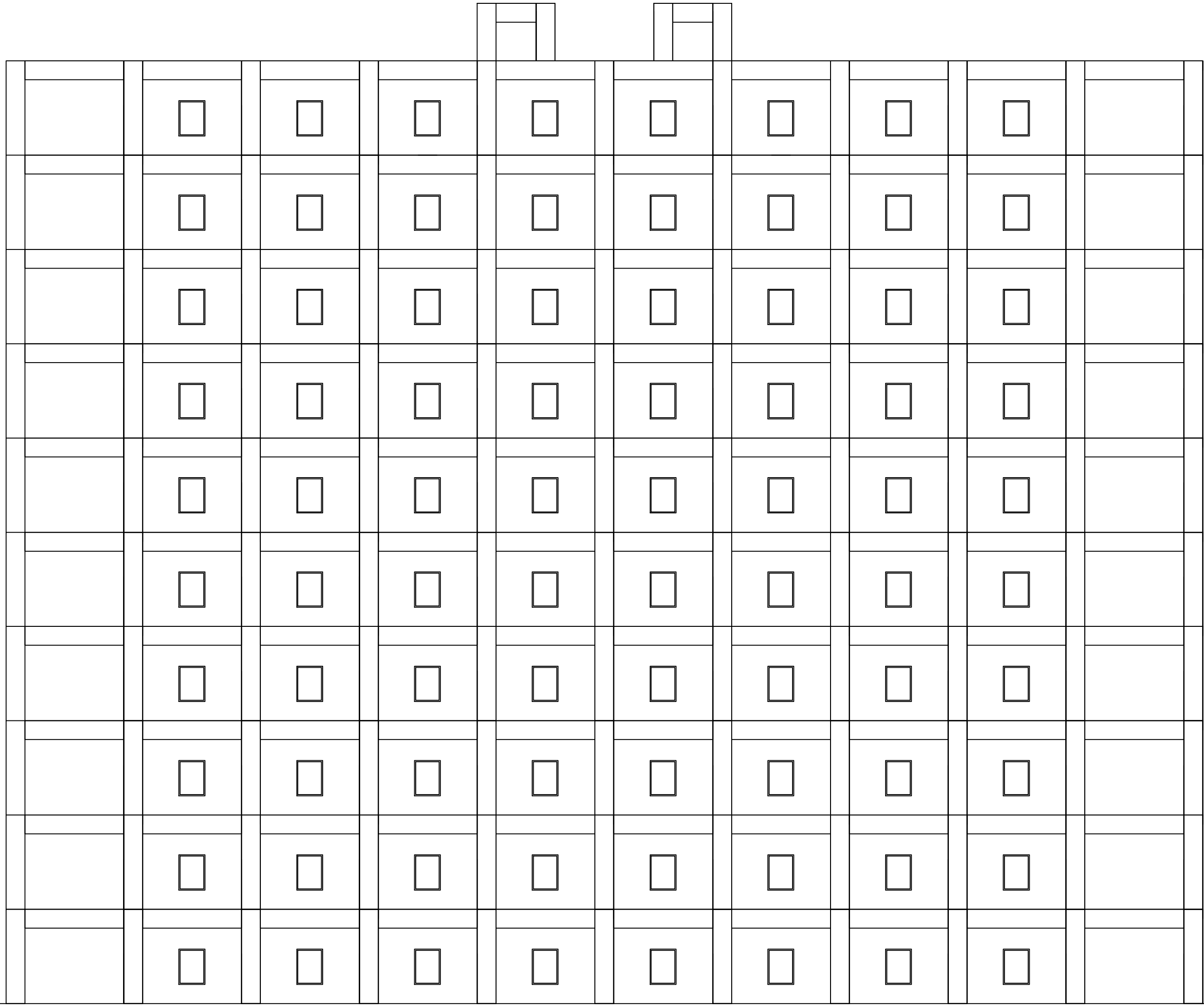

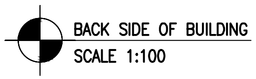

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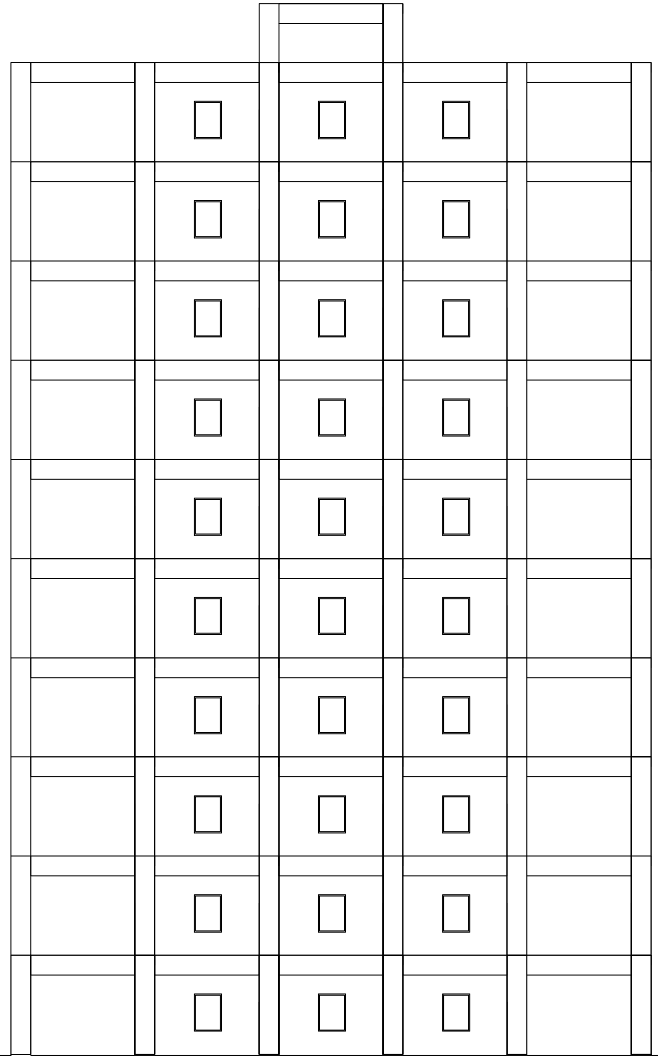

LEFT SIDE OF BUILDING
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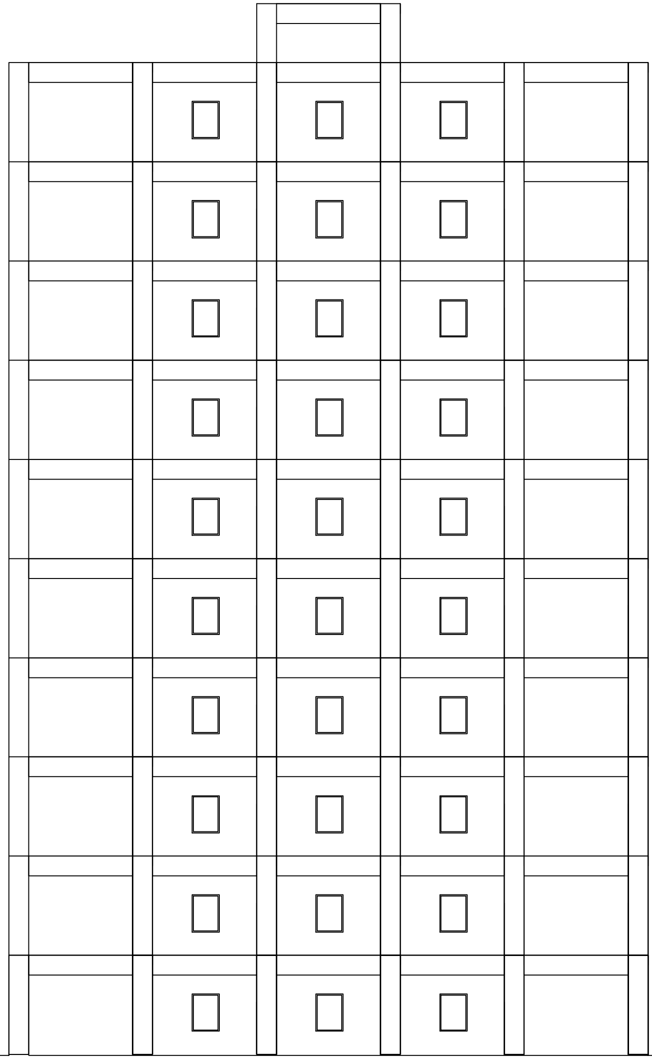

RIGHT SIDE OF BUILDING
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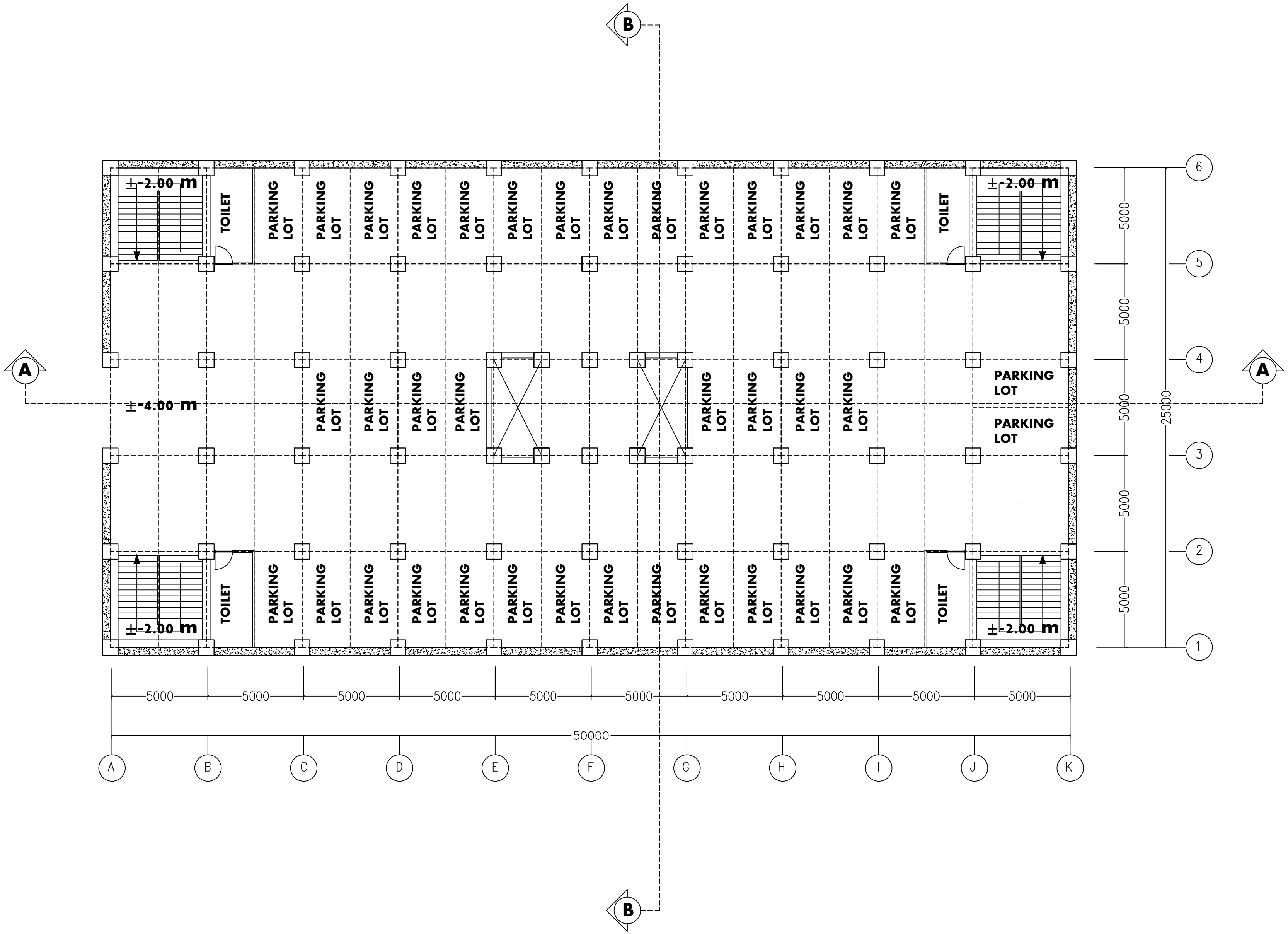

ARCHITECTURAL BLUEPRINT FOR LOWER GROUND (BASEMENT) FLOOR
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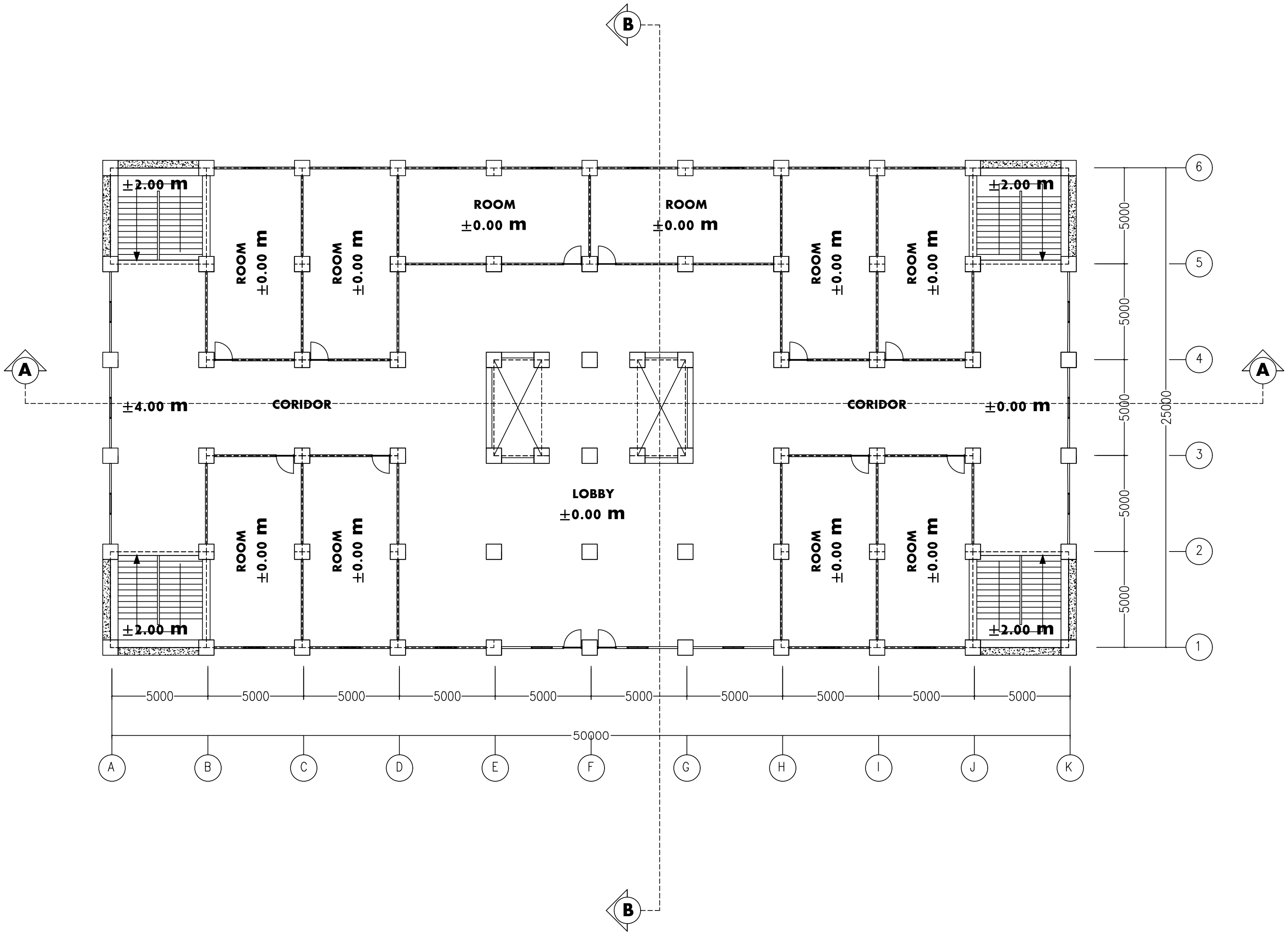

ARCHITECTURAL BLUEPRINT FOR GROUND FLOOR
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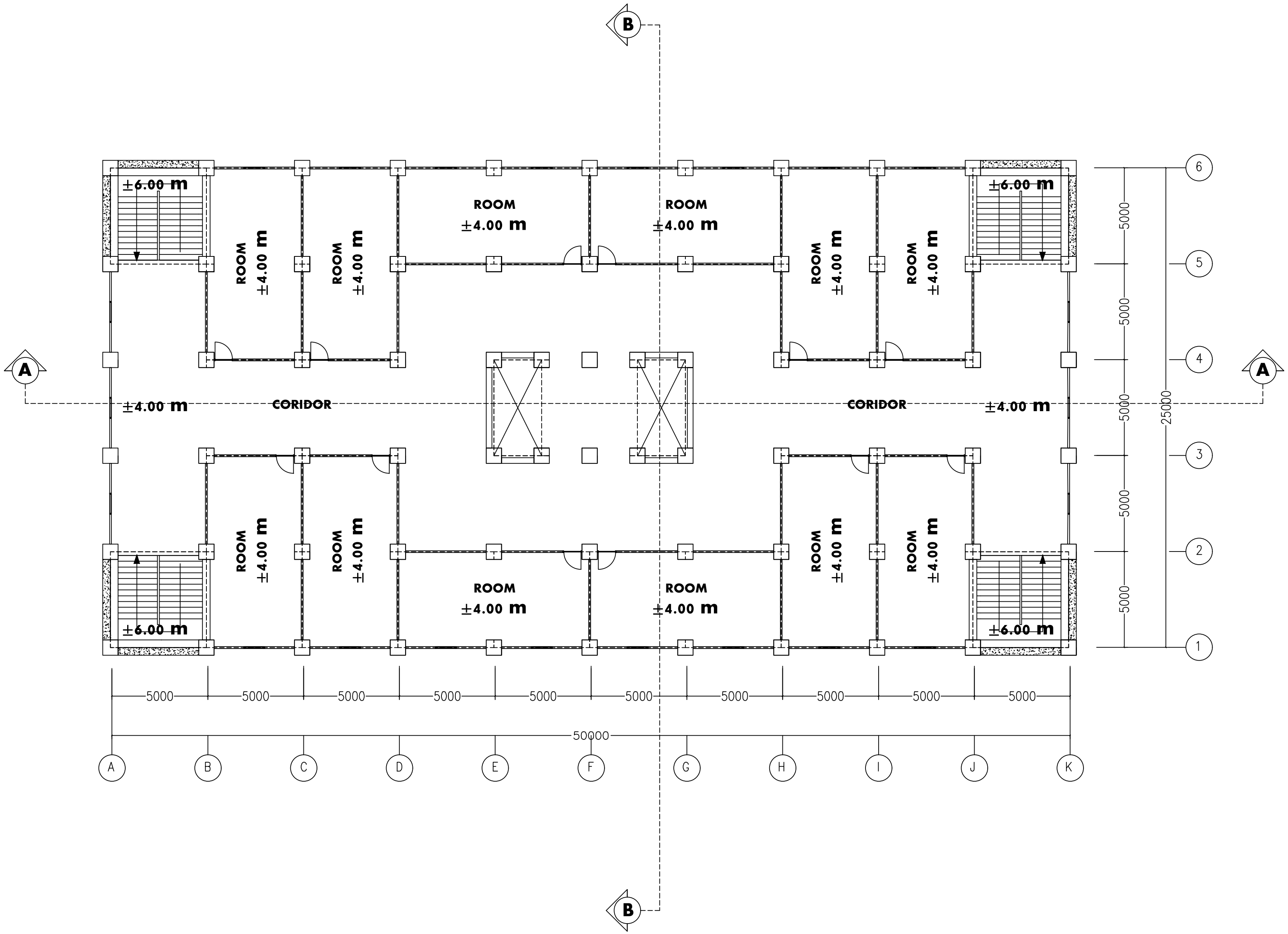

ARCHITECTURAL BLUEPRINT FOR 1ST, 2ND, 3RD, 4TH, 5TH, 6TH, 7TH, 8TH, 9TH FLOOR
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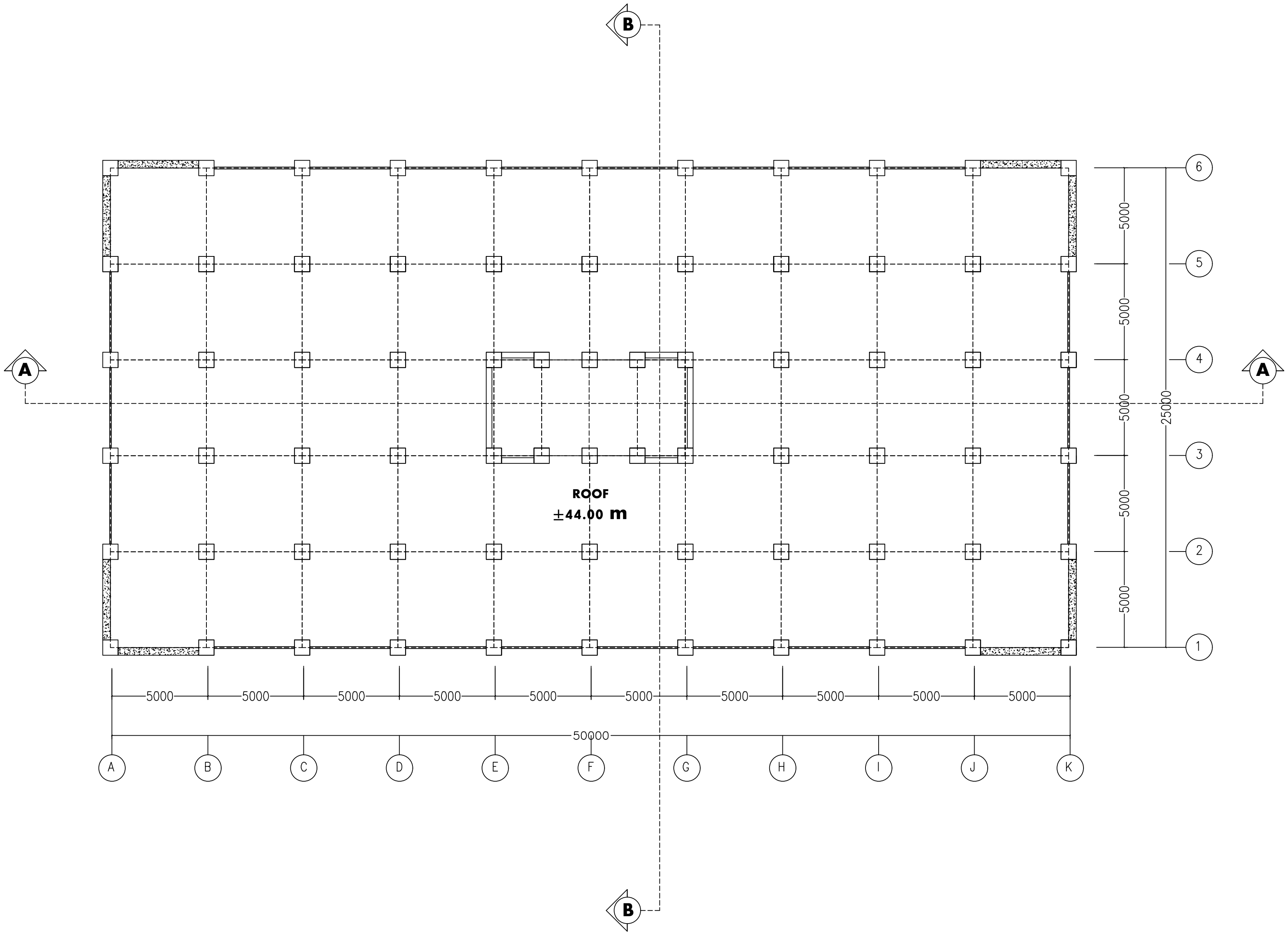

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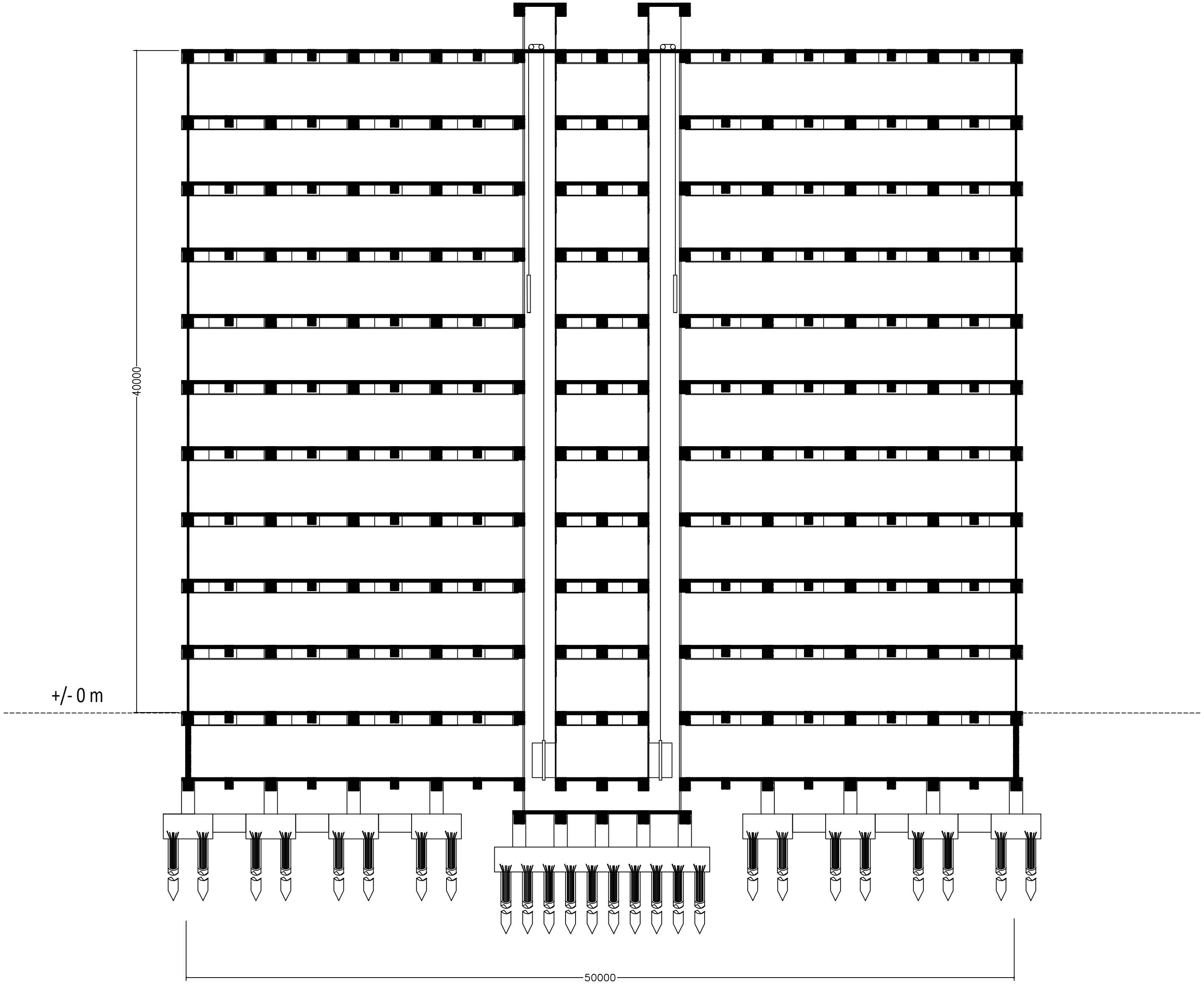

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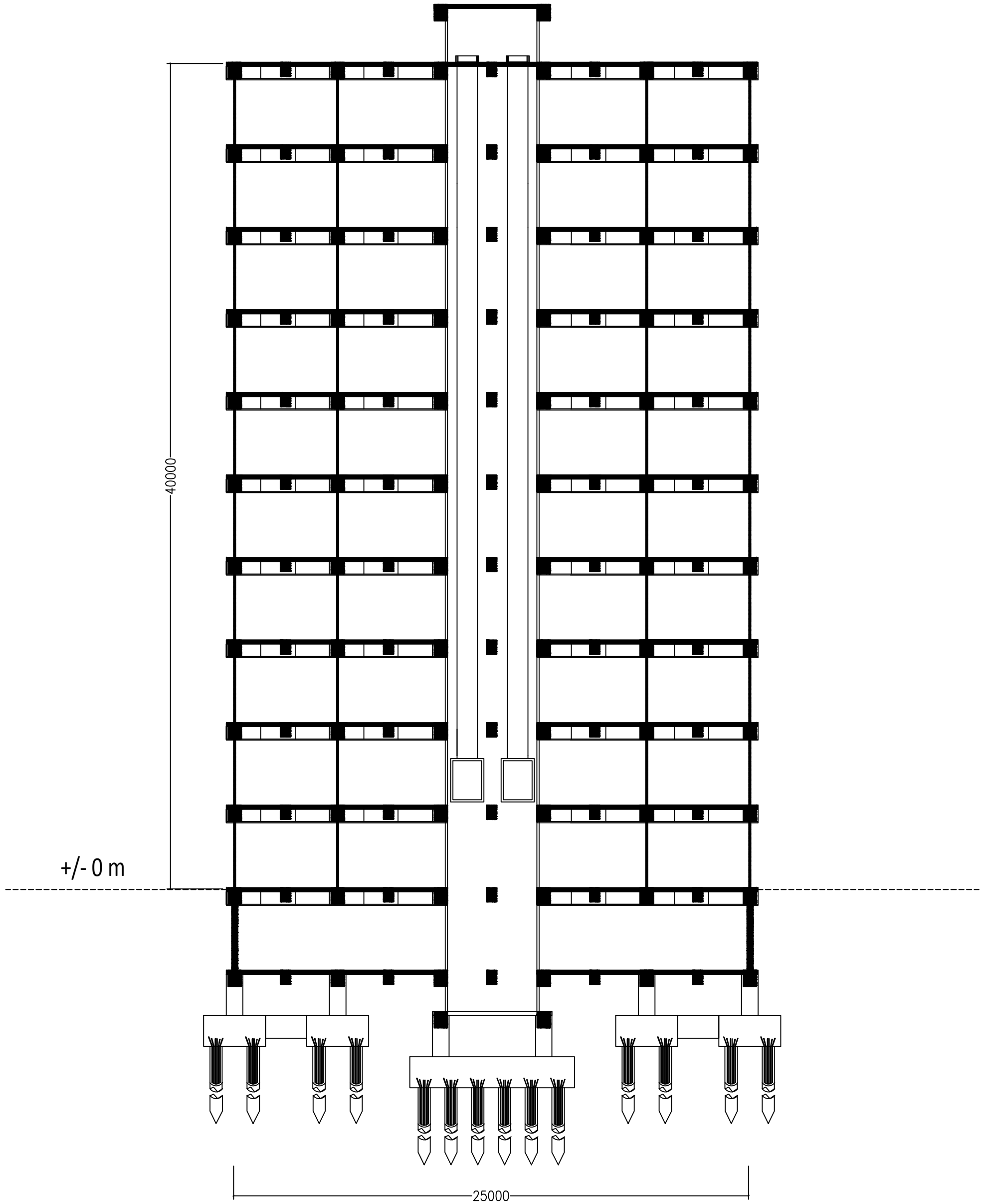

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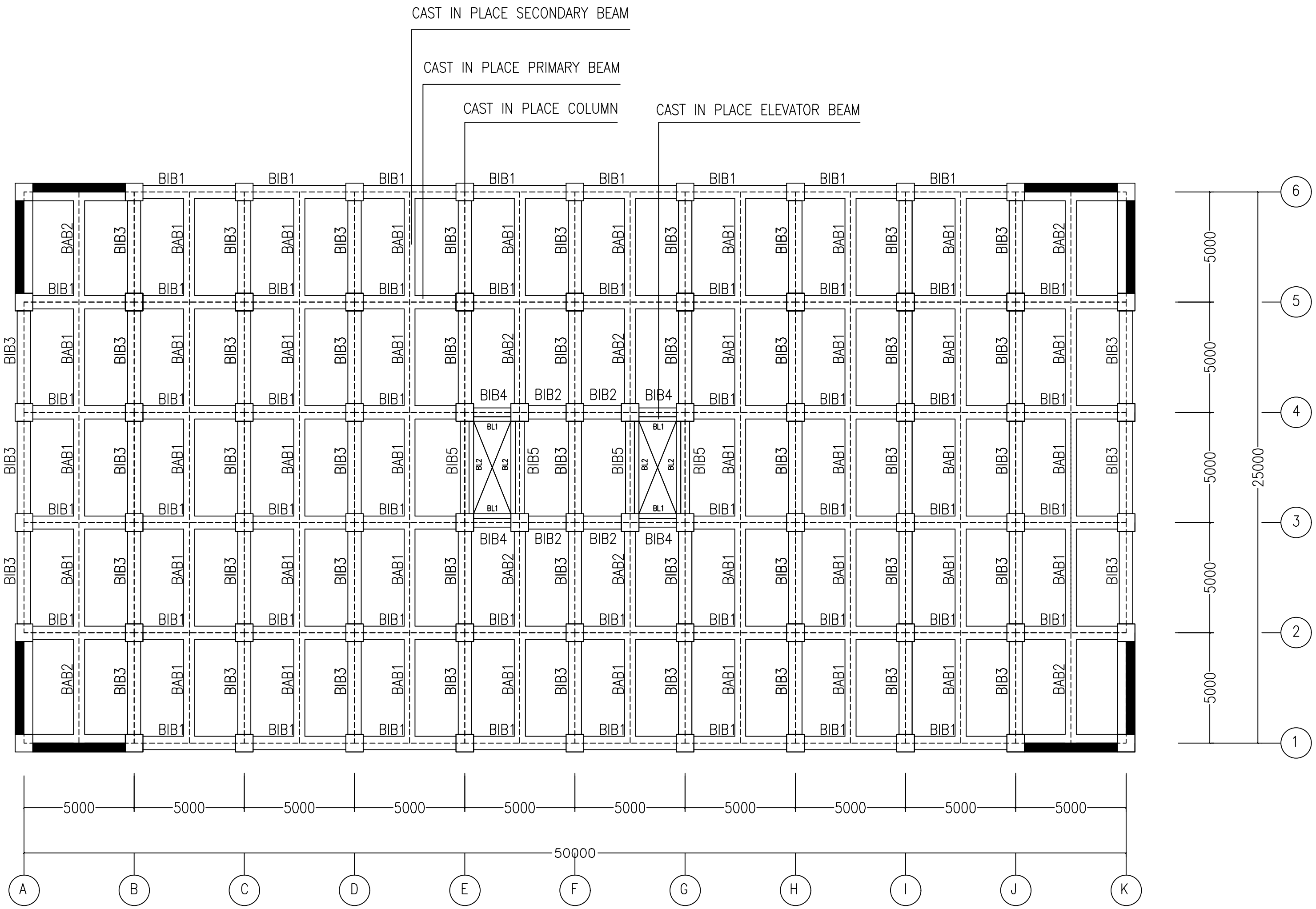

BEAM DESIGNING BLUEPRINT FOR LOWER GROUND (BASEMENT) FLOOR

SCALE 1:100

LEGEND

BEAM STRUCTURE	DIMENSION		
	b	h	L
BIB1	650	800	5000
BIB2	650	800	2500
BIB3	650	800	5000
BIB4	400	800	2500
BIB5	400	800	5000
BAB1	500	700	5000
BAB2	500	700	5000

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TECHNICAL DRAWING

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STUDENT's ID NUMBER

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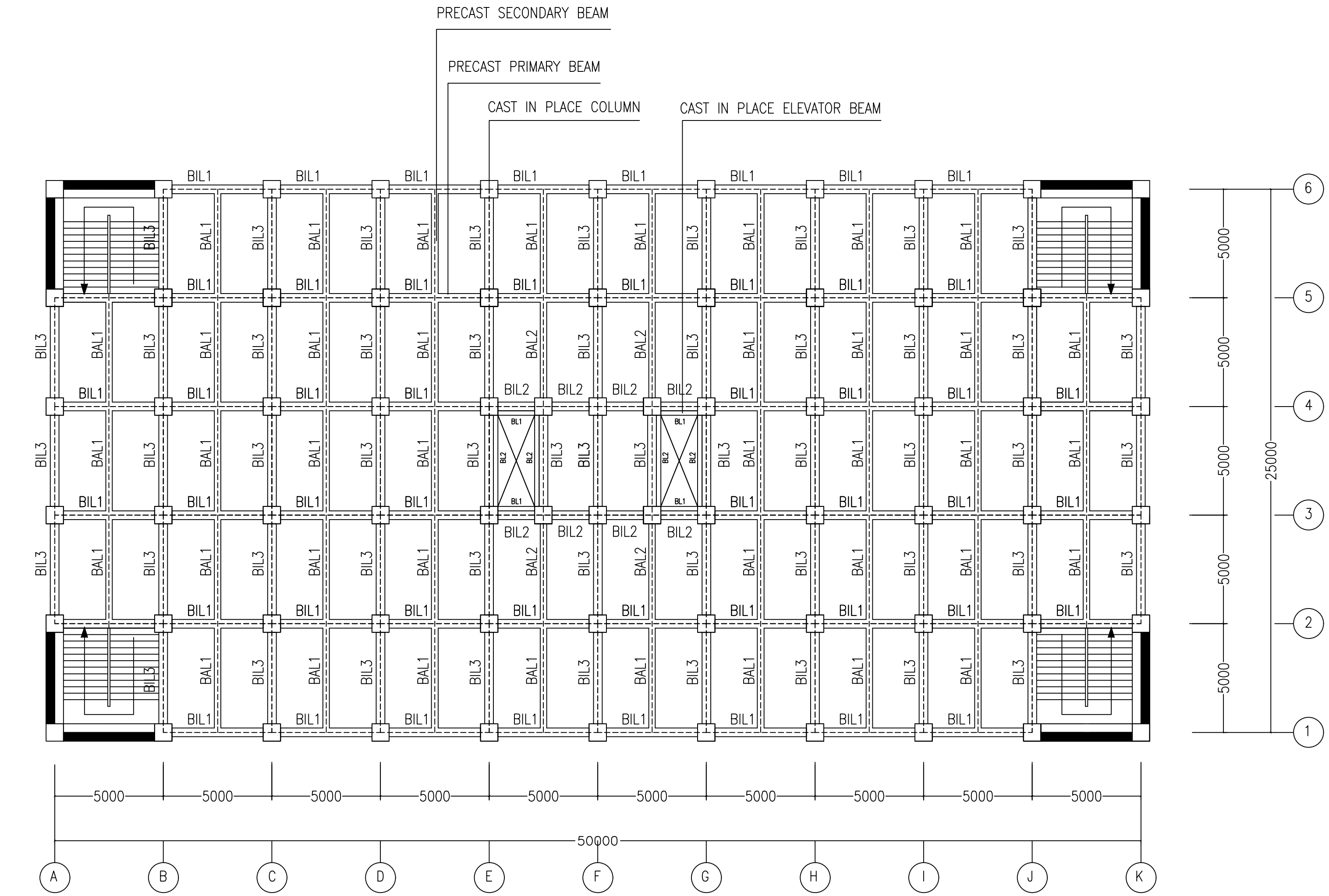

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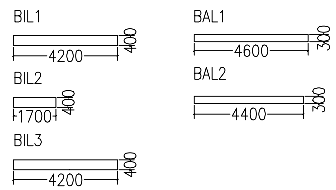

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BIL2	400	650	1700
BIL3	400	650	4200
BAL1	300	450	4600
BAL2	300	450	4400
BL1	200	650	1700
BL2	200	650	4200

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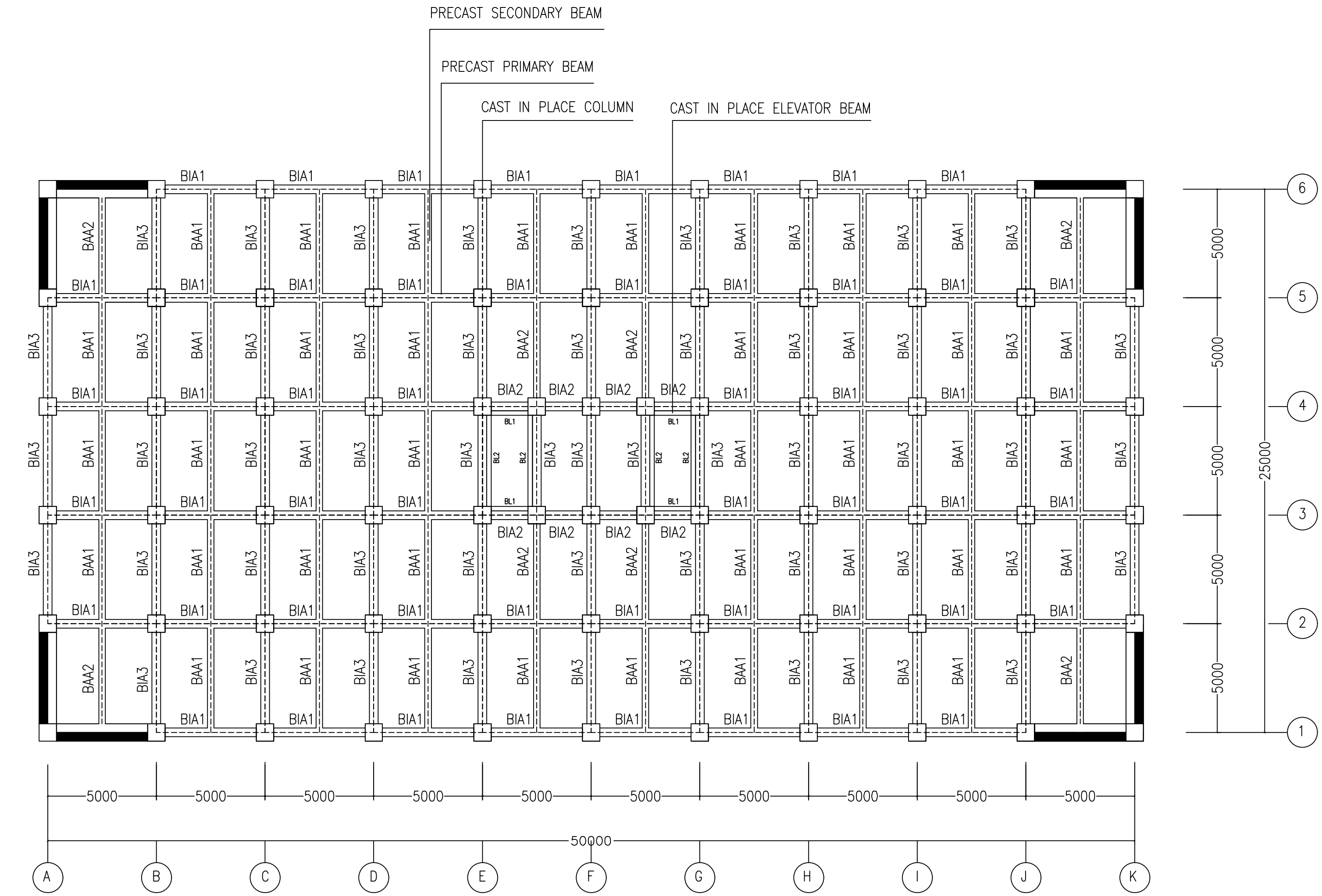

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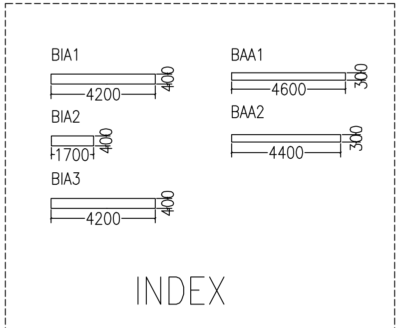

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BIA2	400	650	1700
BIA3	400	650	4200
BAA1	300	450	4600
BAA2	300	450	4400
BL1	200	650	1700
BL2	200	650	4200

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TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
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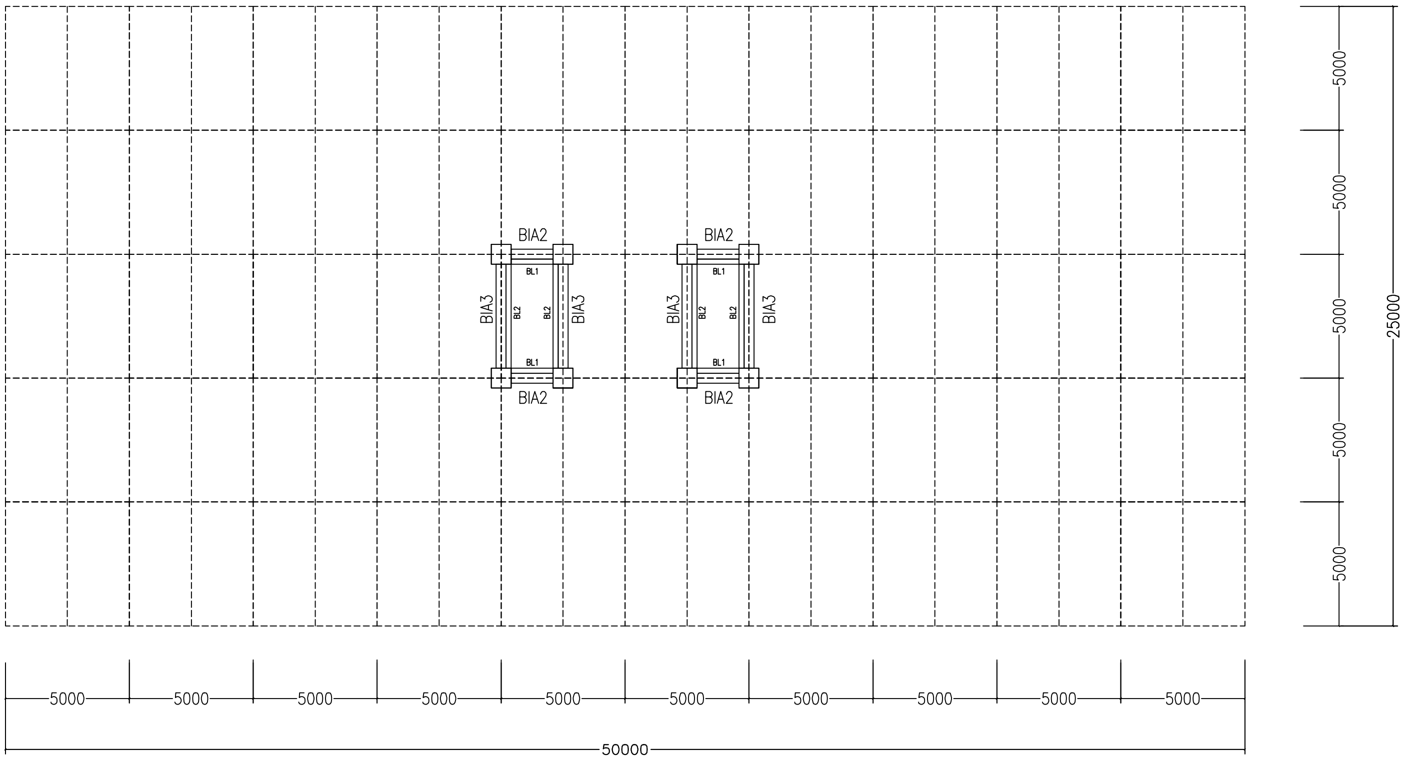

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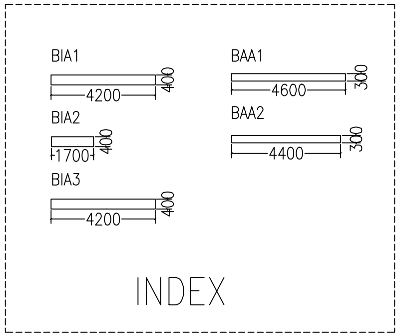

LEGEND

BEAM STRUCTURE	DIMENSION		
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BIA2	400	650	1700
BIA3	400	650	4200
BL1	200	650	1700
BL2	200	650	4200

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2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

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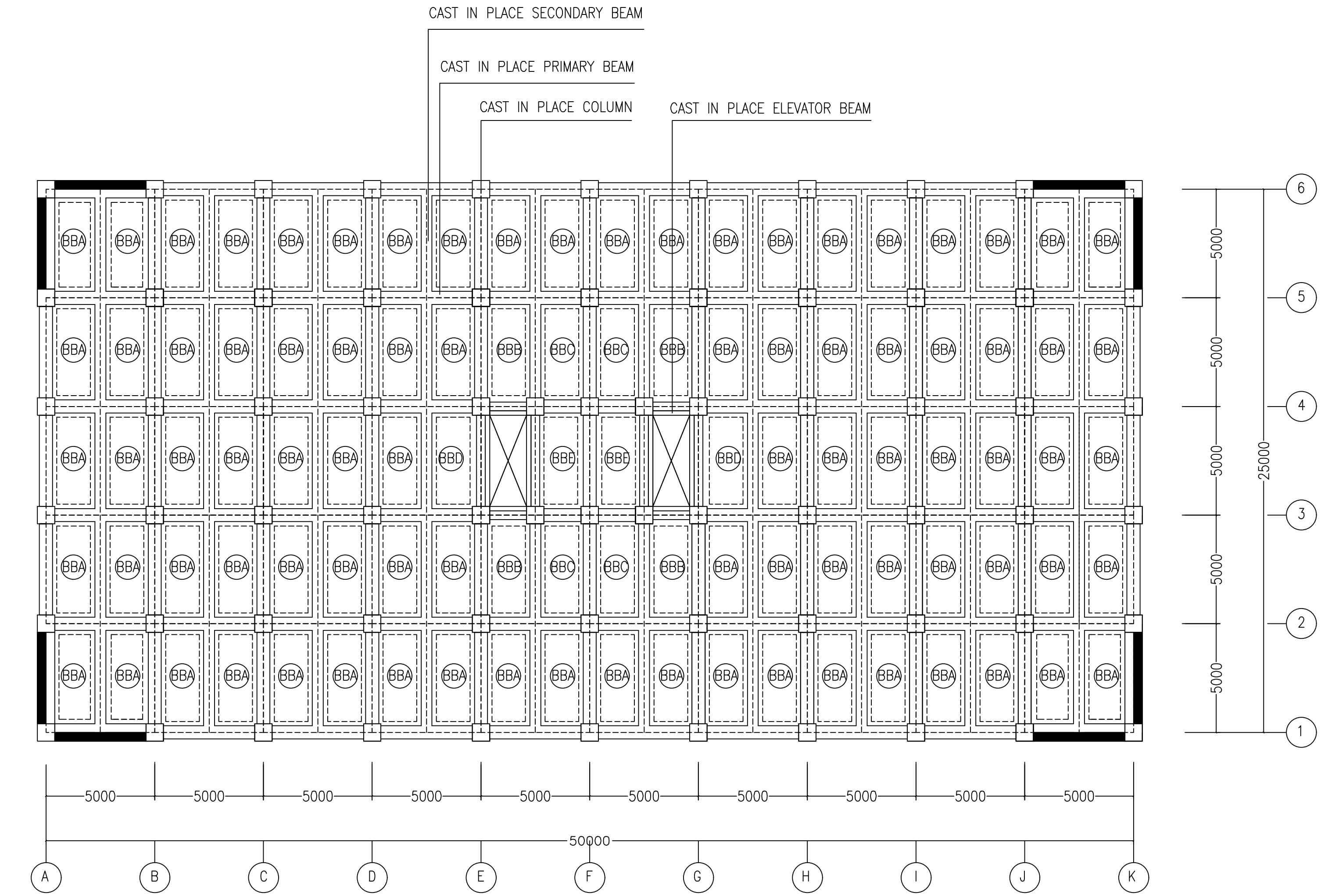

SLAB DESIGNING BLUEPRINT FOR LOWER GROUND (BASEMENT) FLOOR
SCALE 1:100

LEGEND

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BBB	5000	2500	200
BBC	5000	2500	200
BBD	5000	2500	200
BBE	5000	2500	200

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FACULTY OF CIVIL ENGINEERING AND PLANNING
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2016

PROJECT's NAME

FINAL PROJECT

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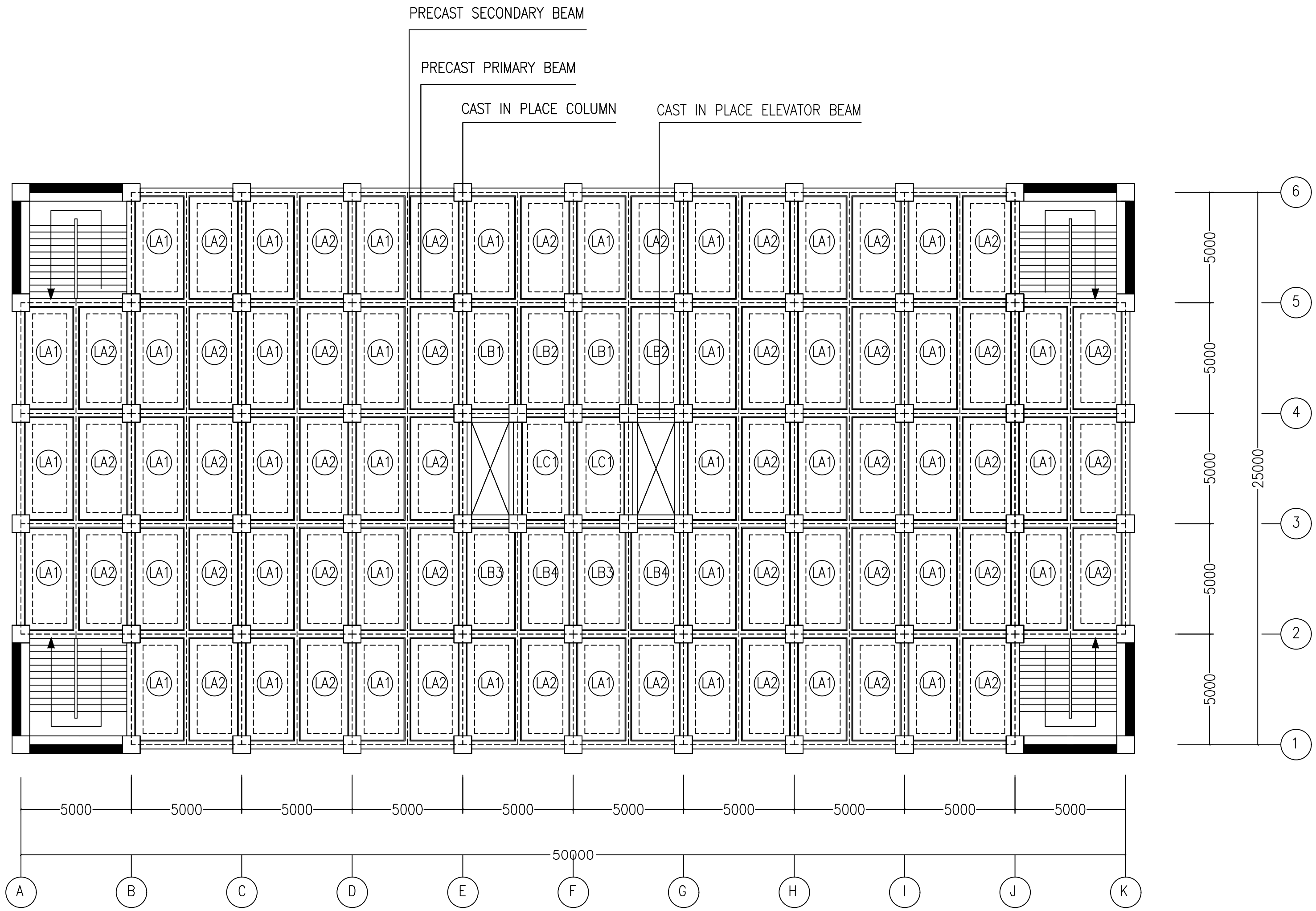

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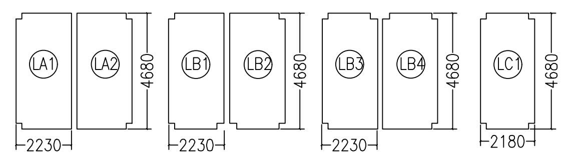

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FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

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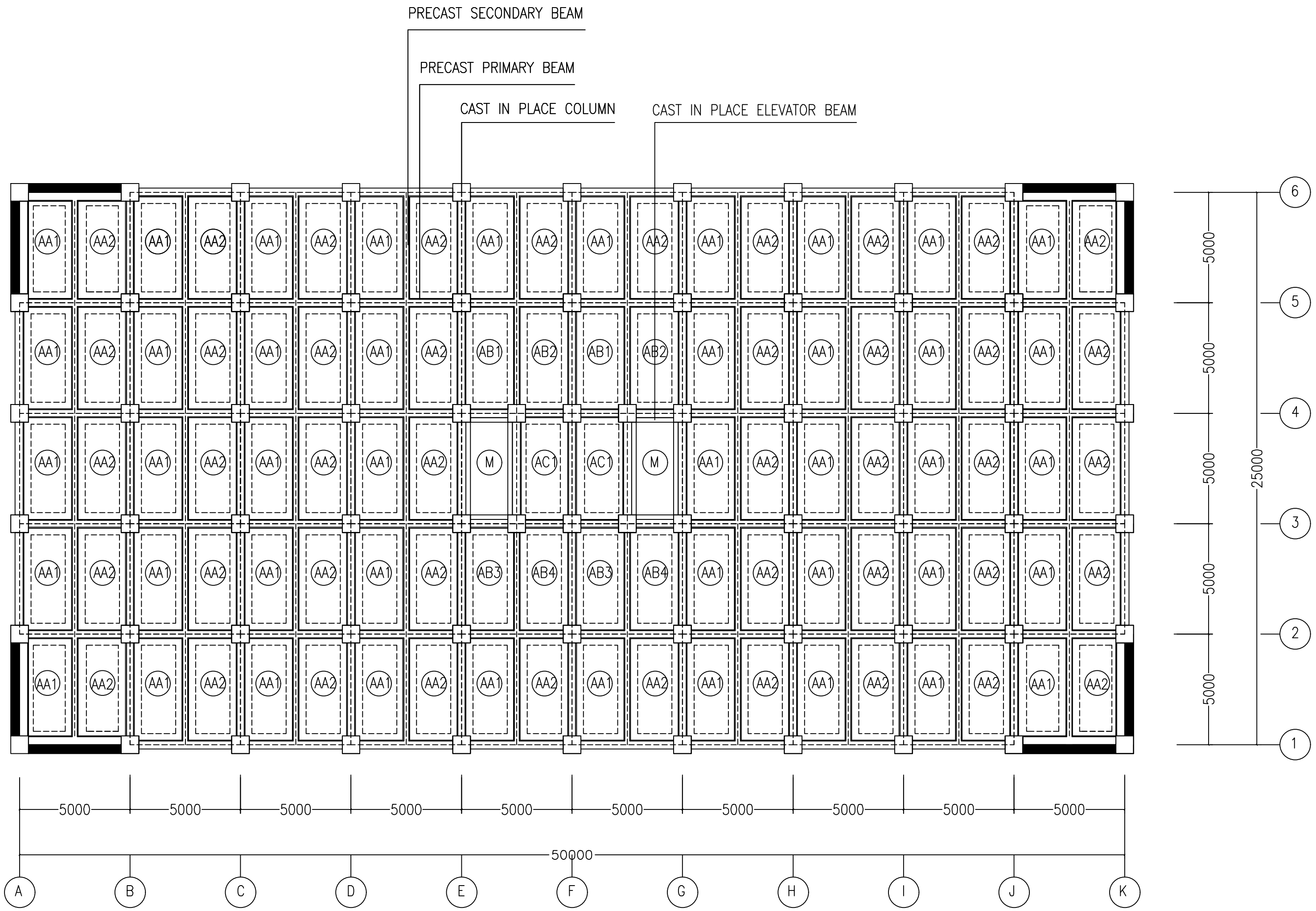

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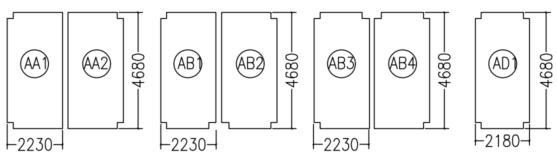


INDEX

LEGEND

SLAB STRUCTURE	DIMENSION		
	P	L	t
AA1	4680	2230	90
AA2	4680	2230	90
AB1	4680	2230	90
AB2	4680	2230	90
AB3	4680	2230	90
AB4	4680	2230	90
AC1	4680	2230	90

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:100

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

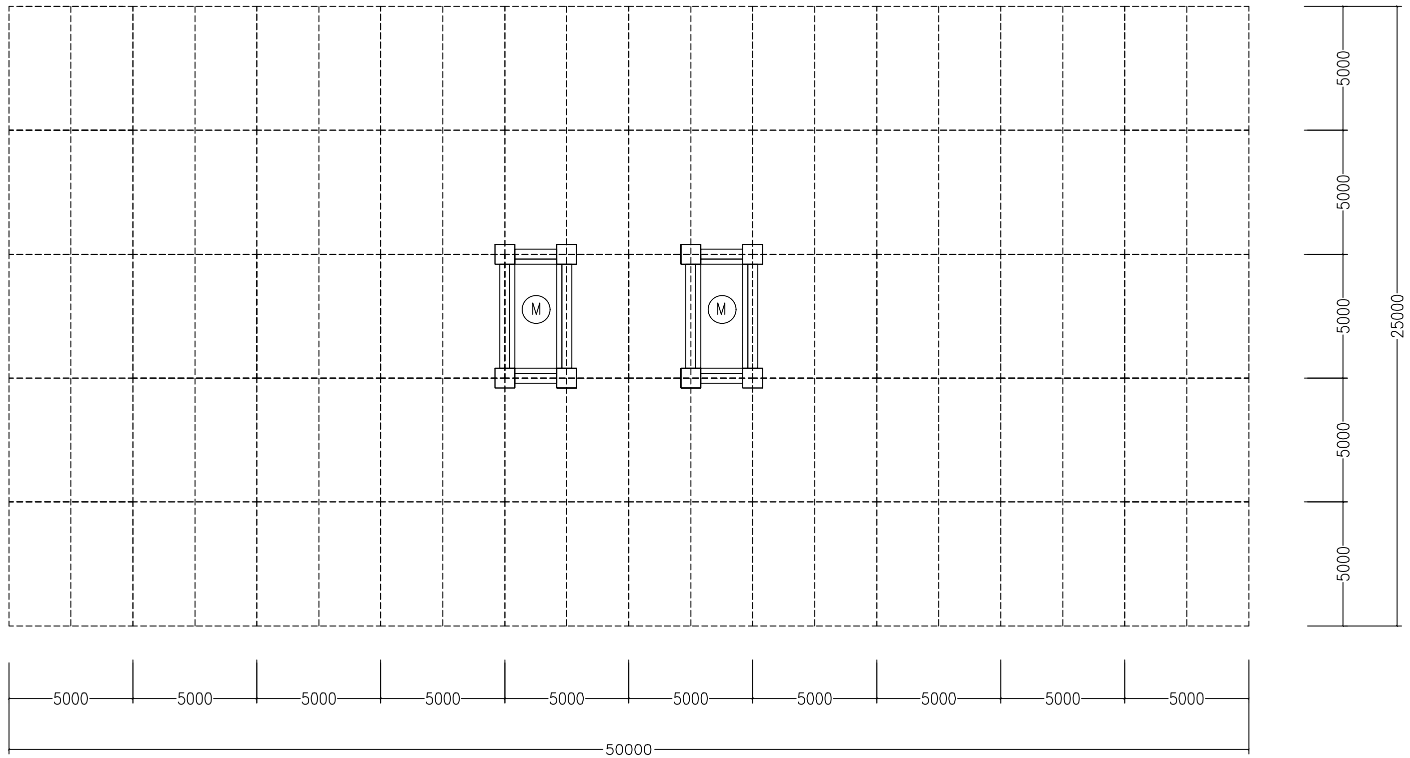
STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

3112100025

NUMBER	TOTAL
17	61



SLAB DESIGNING BLUEPRINT FOR ROOF (ELEVATOR)
SCALE 1:100

LEGEND

SLAB STRUCTURE	DIMENSION		
	P	L	t
M	5000	2500	150

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:100

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

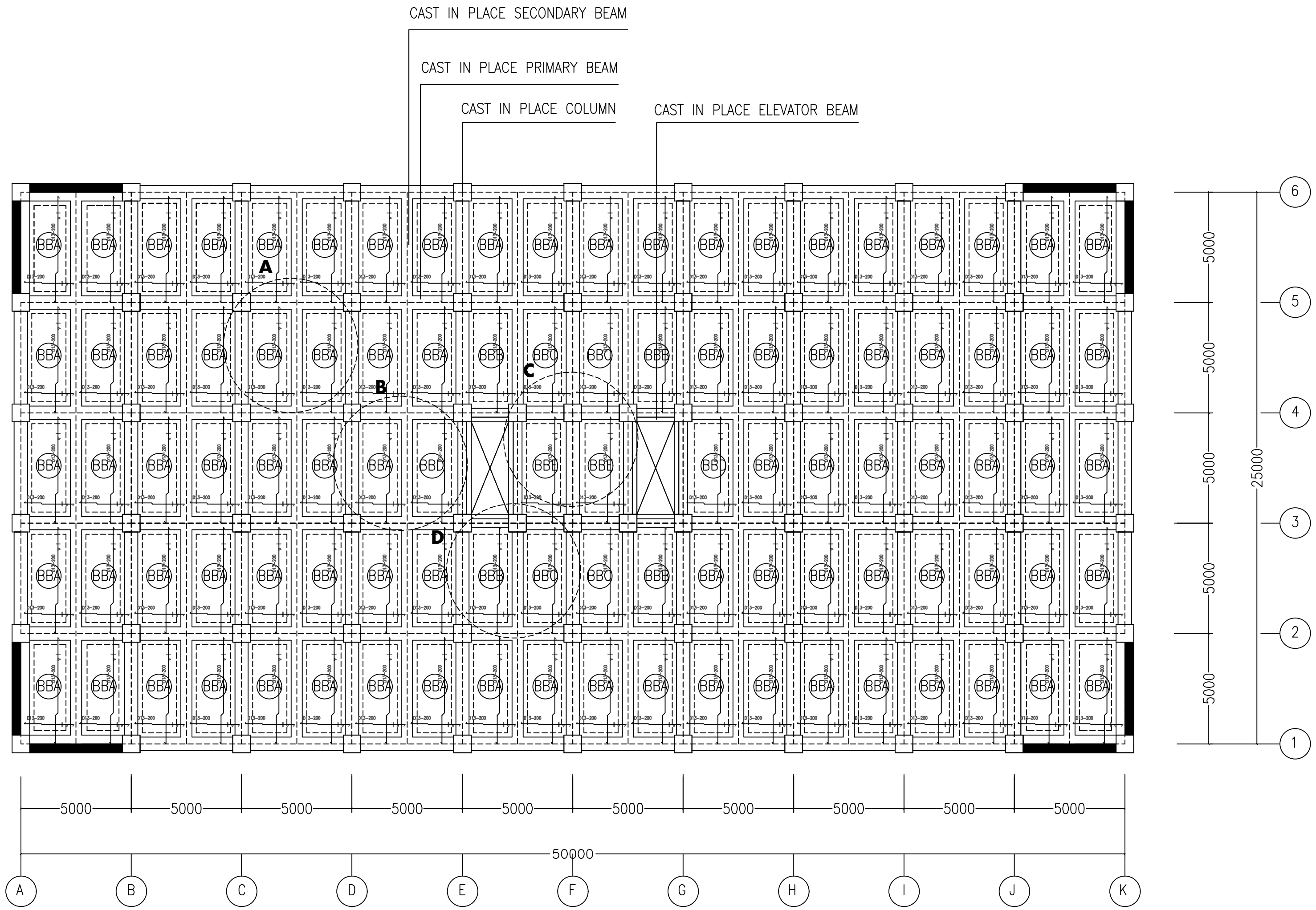
STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

3112100025

NUMBER	TOTAL
18	61



SLAB REINFORCEMENT BLUEPRINT FOR LOWER GROUND (BASEMENT) FLOOR
SCALE 1:100

LEGEND

SLAB STRUCTURE	DIMENSION		
	P	L	t
BBA	5000	2500	200
BBB	5000	2500	200
BBC	5000	2500	200
BBD	5000	2500	200
BBE	5000	2500	200

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
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PROJECT's NAME

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TECHNICAL DRAWING

STRUCTURAL

SCALE

1:100

ADVISOR LECTURER

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Dr. Ir. DJOKO IRAWAN, MS.

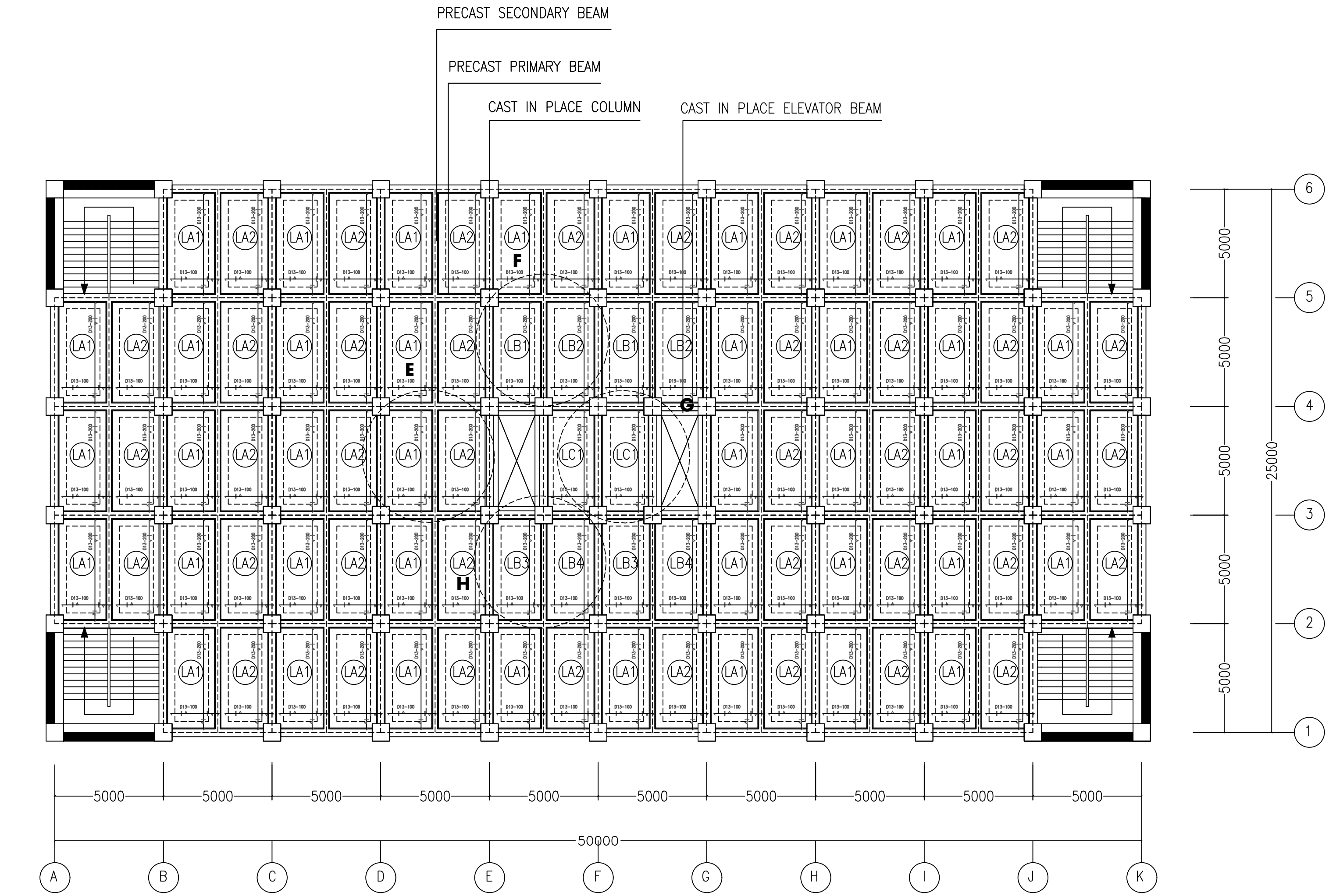
STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

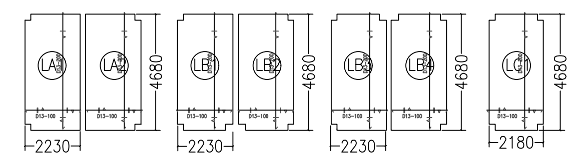
3112100025

NUMBER	TOTAL
19	61



SLAB REINFORCEMENT BLUEPRINT FOR GROUND, 1ST, 2ND, 3RD, 4TH, 5TH, 6TH, 7TH, 8TH, 9TH FLOOR

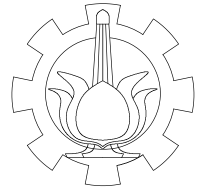
SCALE 1:100



INDEX

LEGEND

SLAB STRUCTURE	DIMENSION		
	P	L	t
LA1	4680	2230	100
LA2	4680	2230	100
LB1	4680	2230	100
LB2	4680	2230	100
LB3	4680	2230	100
LB4	4680	2230	100
LC1	4680	2230	100



DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:100

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

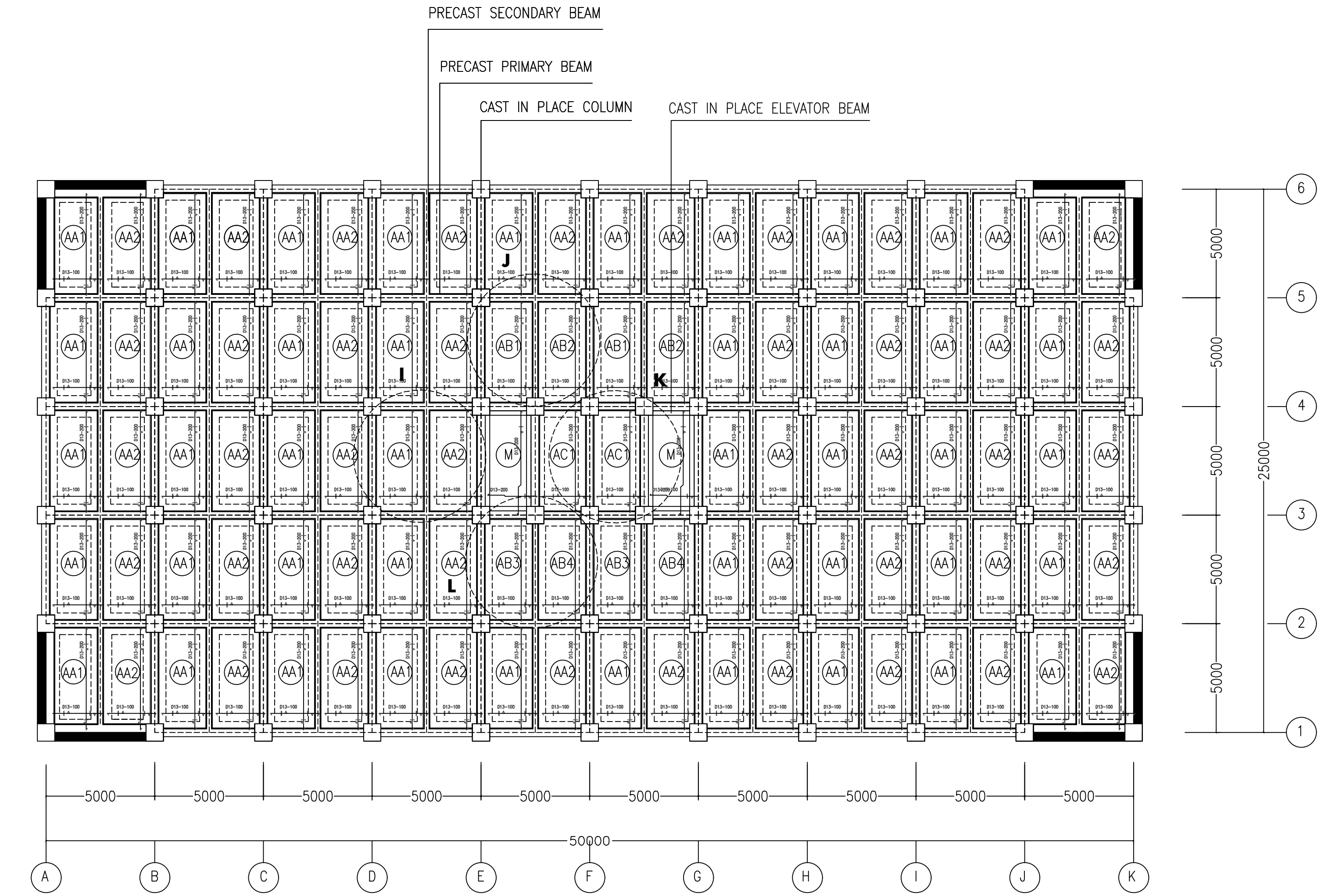
STUDENT's NAME

SOCA FAHREZA I.

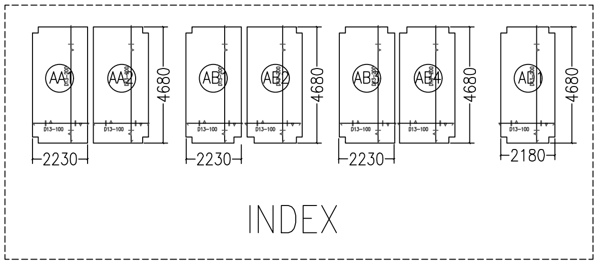
STUDENT's ID NUMBER

3112100025

NUMBER	TOTAL
20	61



SLAB REINFORCEMENT BLUEPRINT FOR ROOF
SCALE 1:100



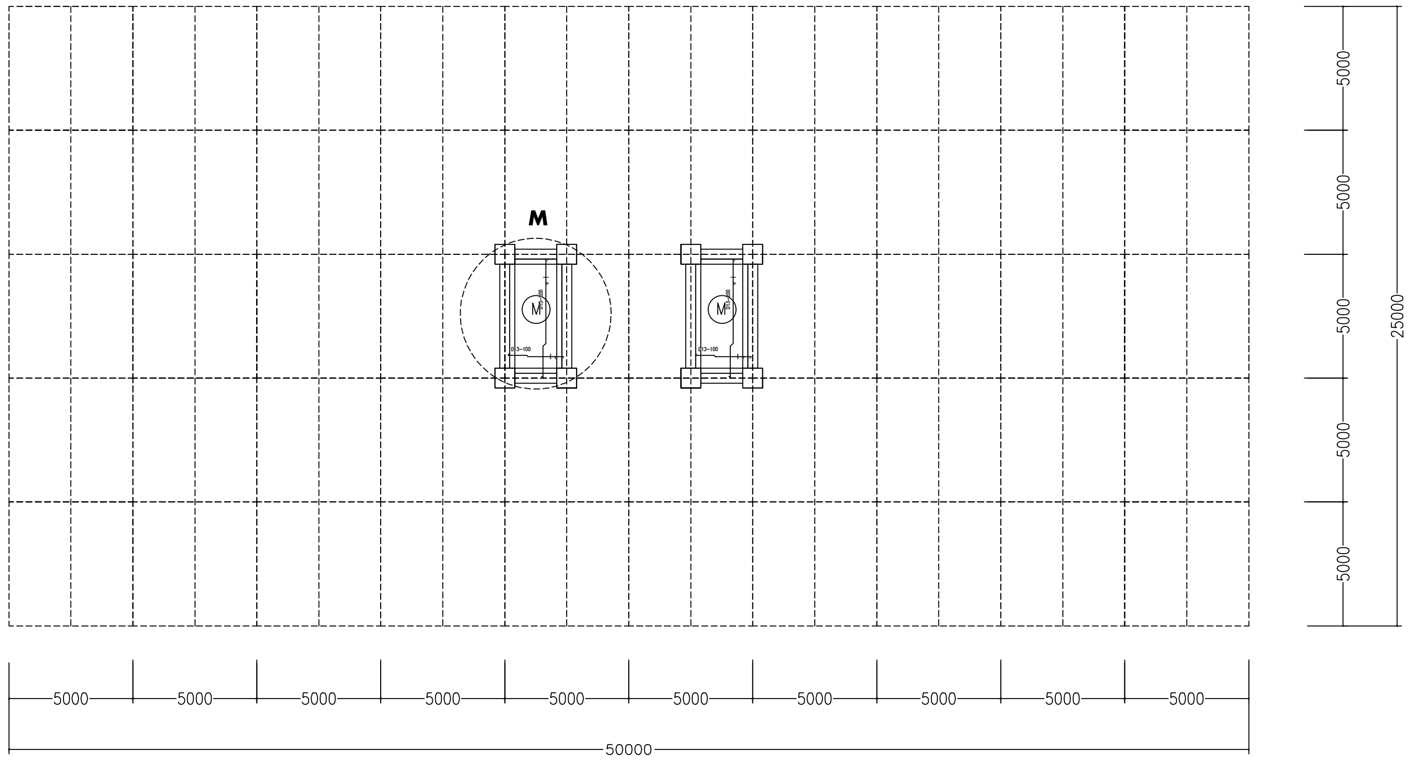
LEGEND

SLAB STRUCTURE	DIMENSION		
	P	L	t
AA1	4680	2230	90
AA2	4680	2230	90
AB1	4680	2230	90
AB2	4680	2230	90
AB3	4680	2230	90
AB4	4680	2230	90
AC1	4680	2230	90

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	FINAL PROJECT
TECHNICAL DRAWING	STRUCTURAL
SCALE	1:100
ADVISOR LECTURER	DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.
STUDENT's NAME	SOCA FAHREZA I.
STUDENT's ID NUMBER	3112100025

NUMBER	TOTAL
21	61



SLAB REINFORCEMENT BLUEPRINT FOR ROOF (ELEVATOR)
SCALE 1:100

LEGEND			
SLAB STRUCTURE	DIMENSION		
	P	L	t
M	5000	2500	150

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:100

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

3112100025

NUMBER	TOTAL
22	61

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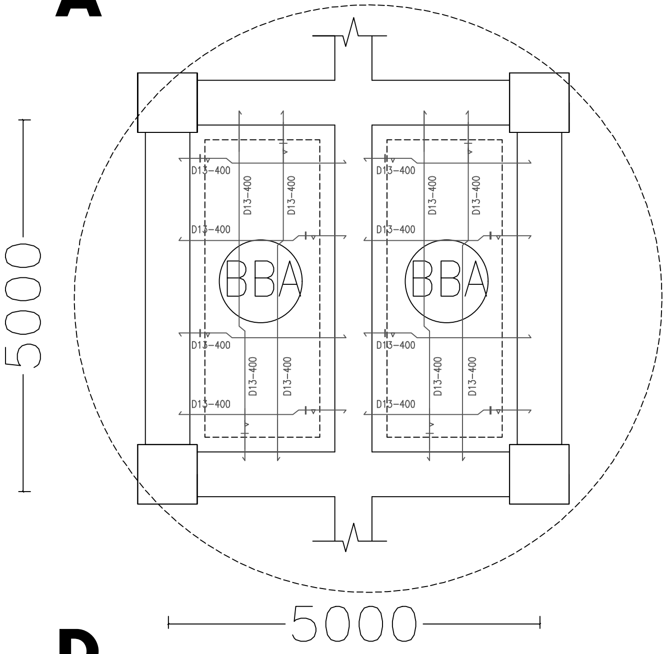

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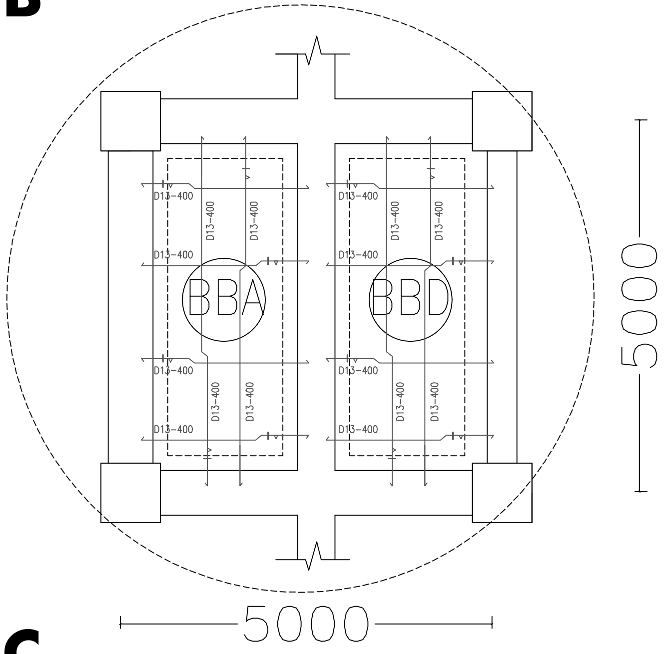

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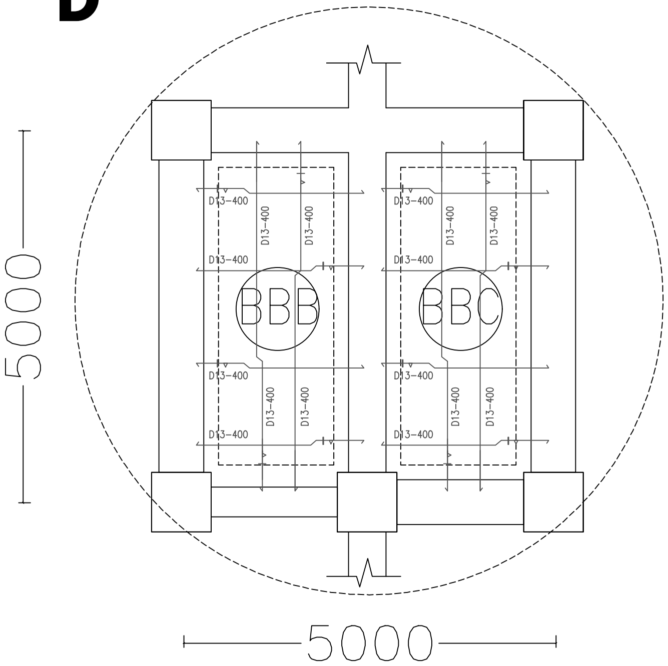

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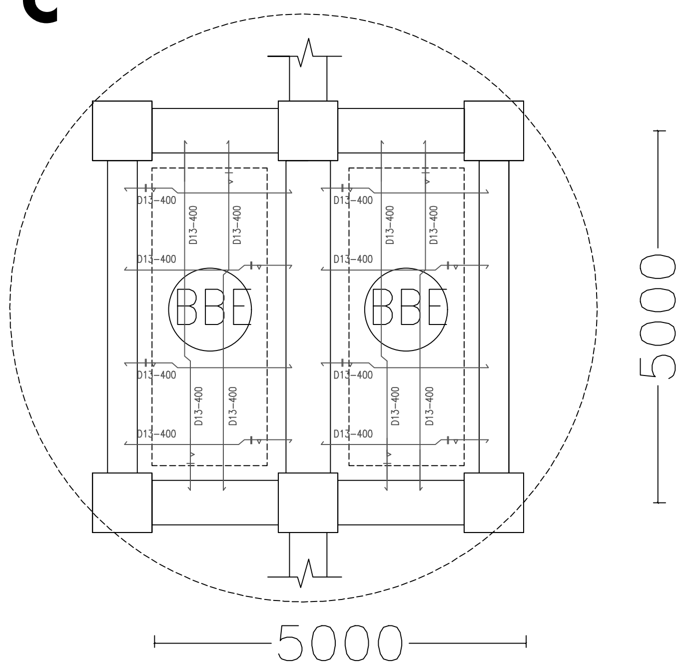
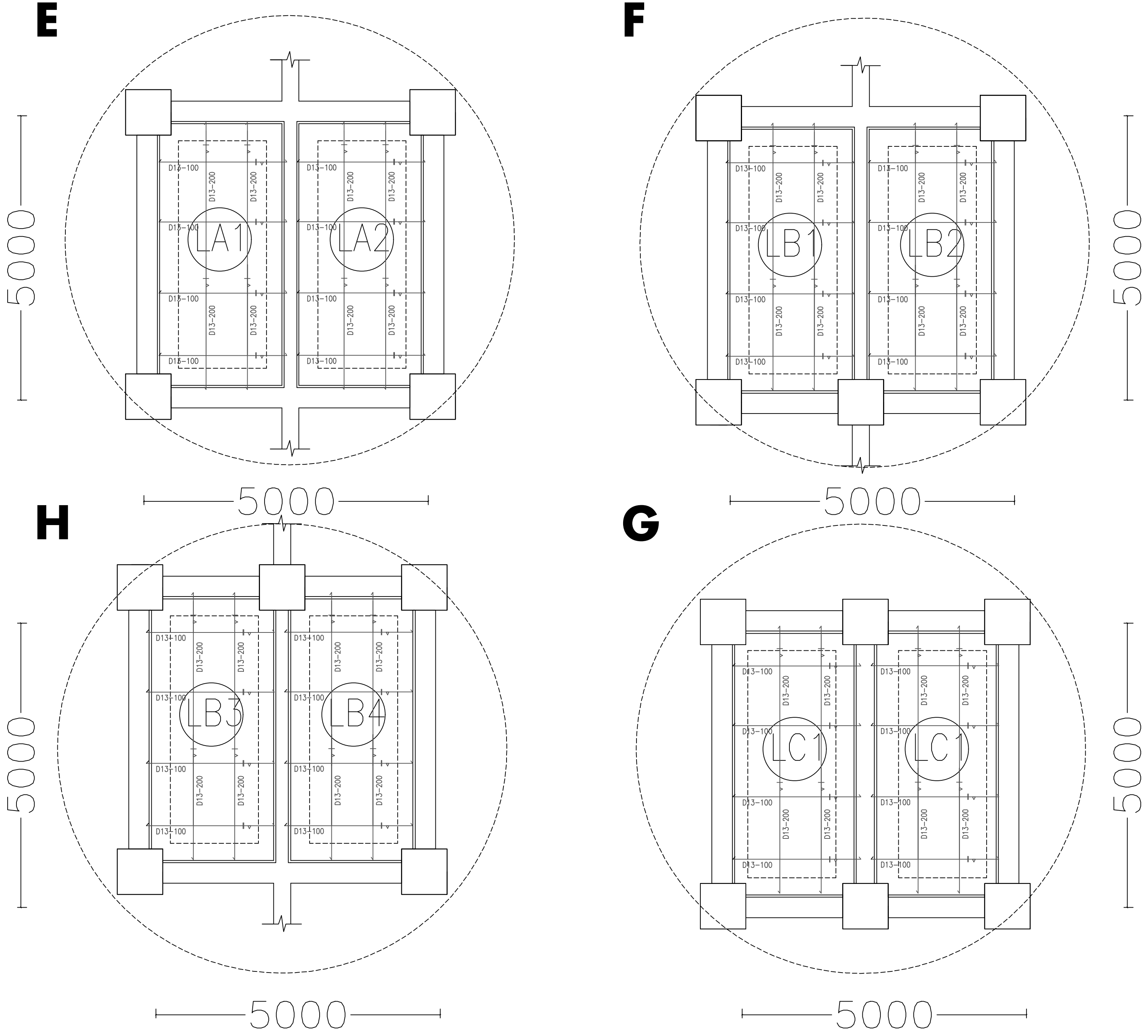


SECTION DETAILING OF SLAB FOR LOWER GROUND (BASEMENT) FLOOR
SCALE 1:30

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:30	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
23	61



SECTION DETAILING OF SLAB FOR GROUND, 1ST, 2ND, 3RD, 4TH, 5TH, 6TH, 7TH, 8TH, 9TH FLOOR
SCALE 1:30

DEPARTEMENT OF CIVIL ENGINEERING
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TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:30

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

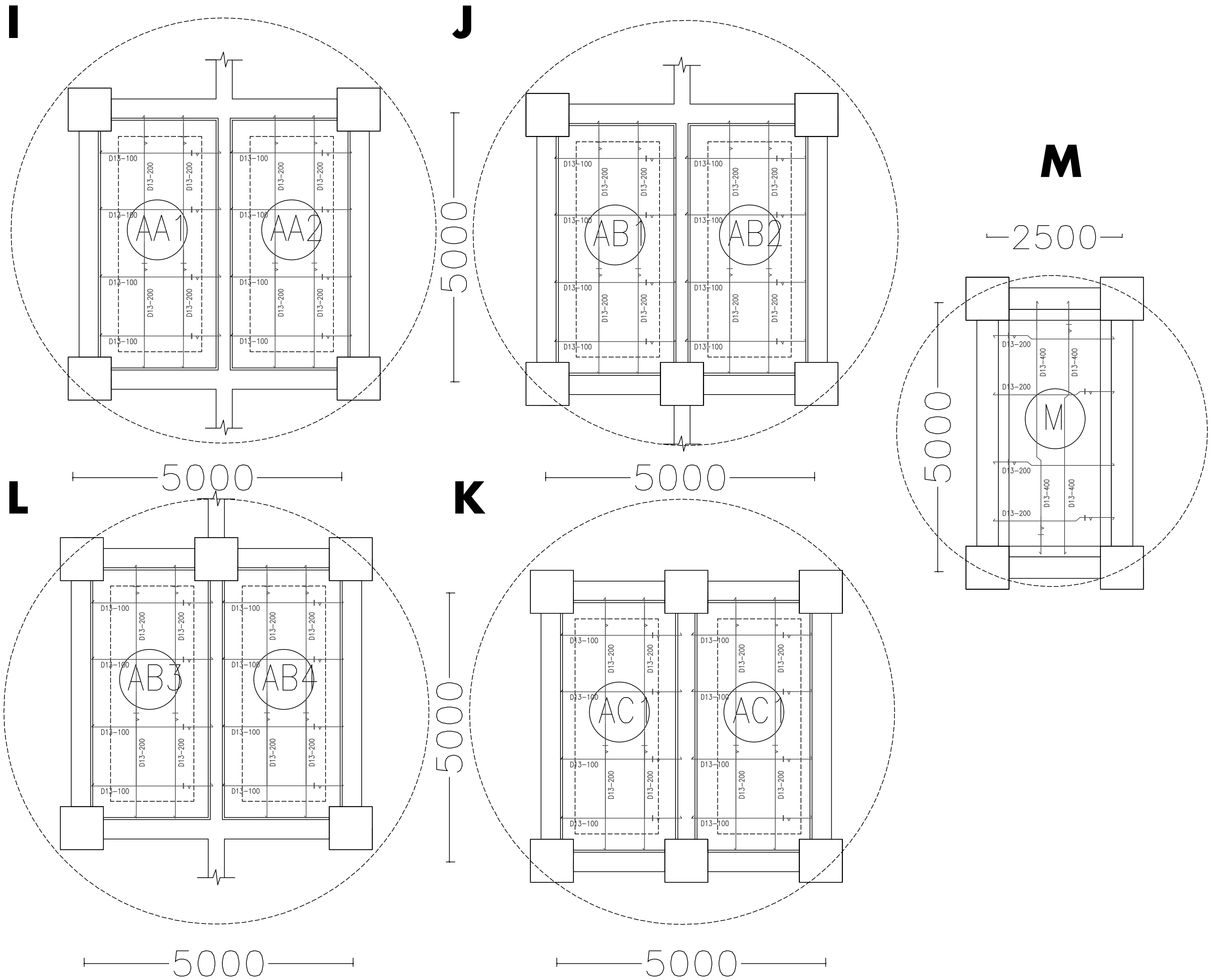
STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

3112100025

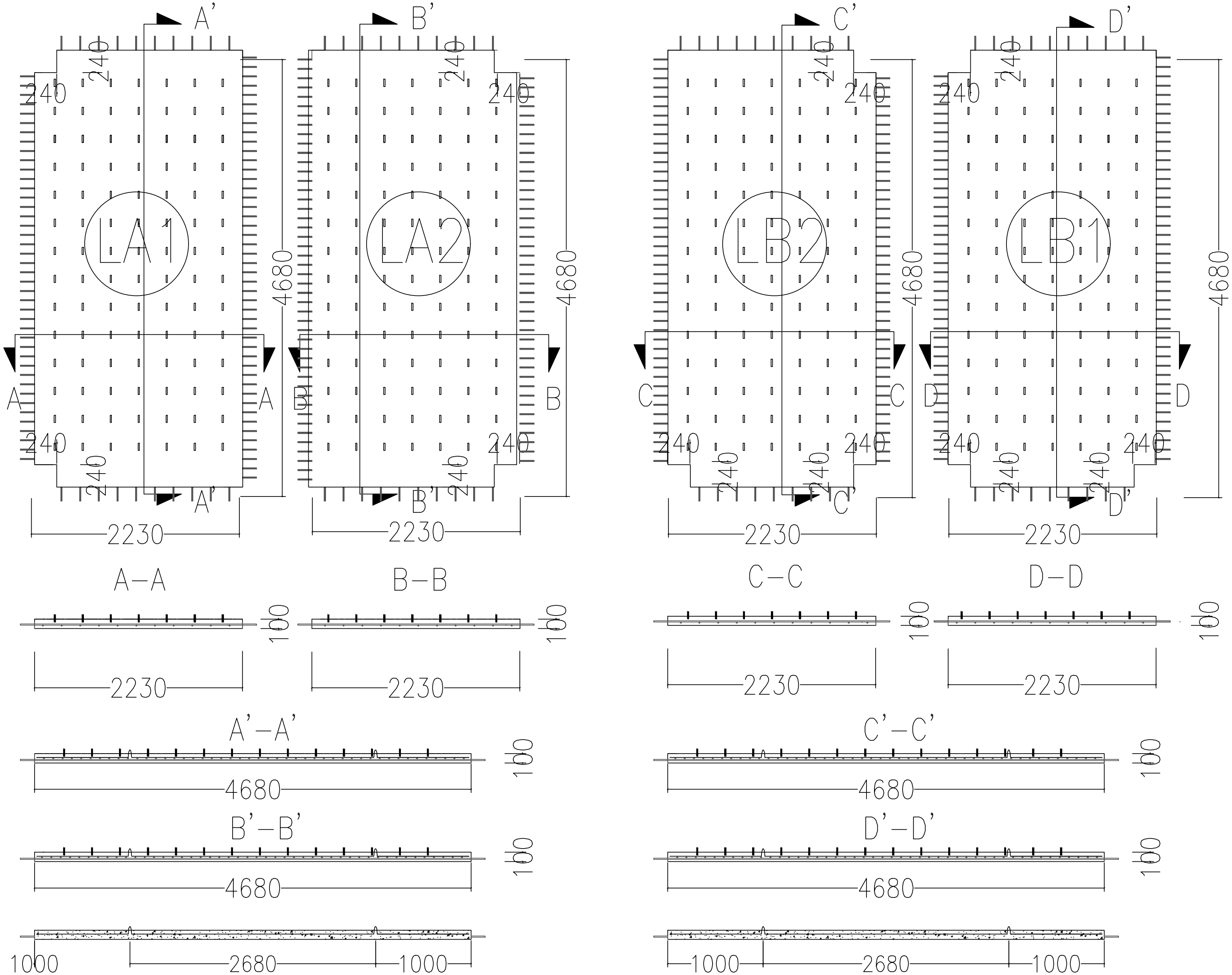
NUMBER	TOTAL
24	61



SECTION DETAILING OF SLAB FOR ROOF
SCALE 1:30

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:30	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
25	61

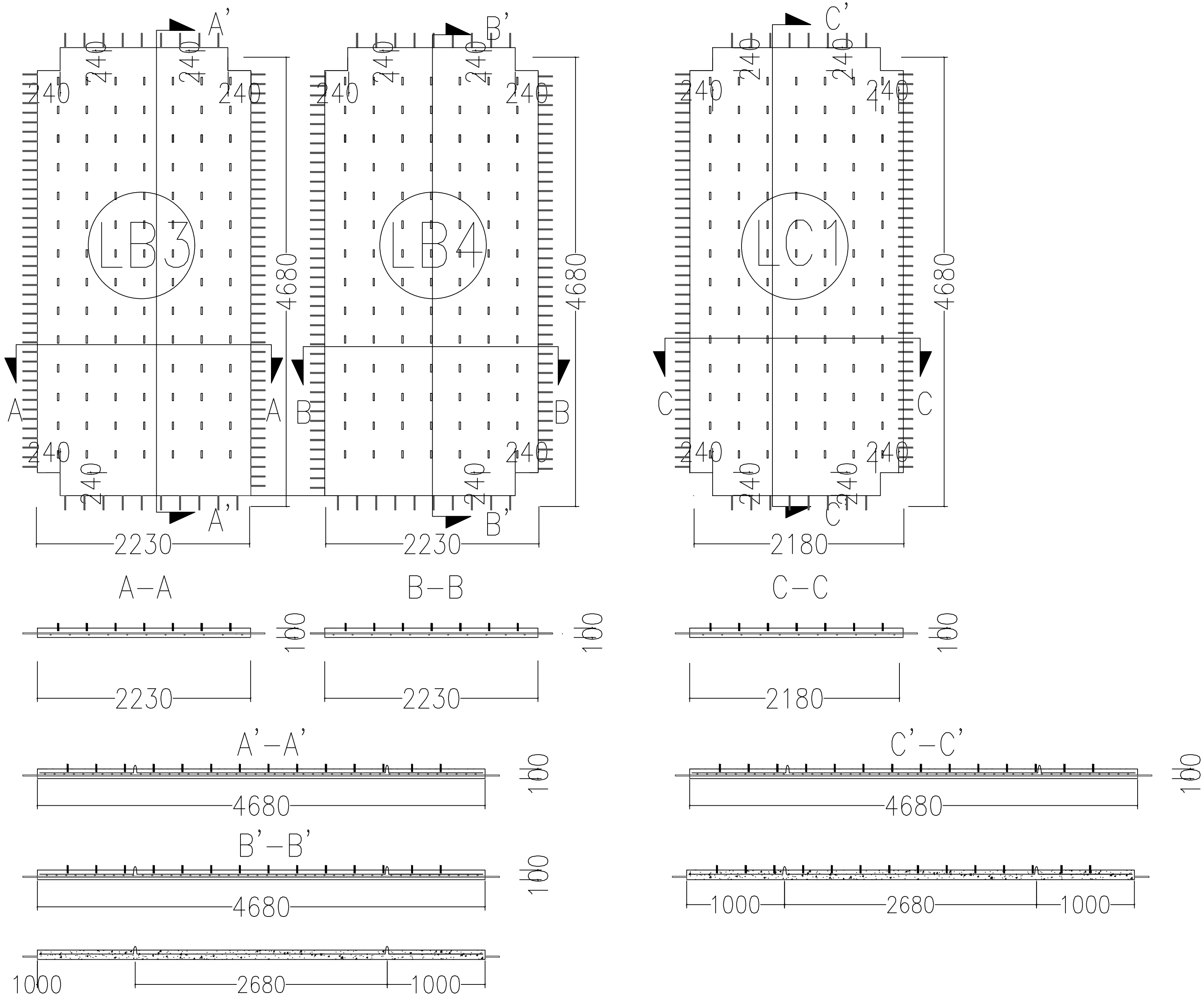


PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR LA1, LA2, LB1, LB2 (BEFORE MONOLITH CONDITION)
SCALE 1:20

SHEAR CONECTOR DETAILING
SCALE 1:5

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:20	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
26	61

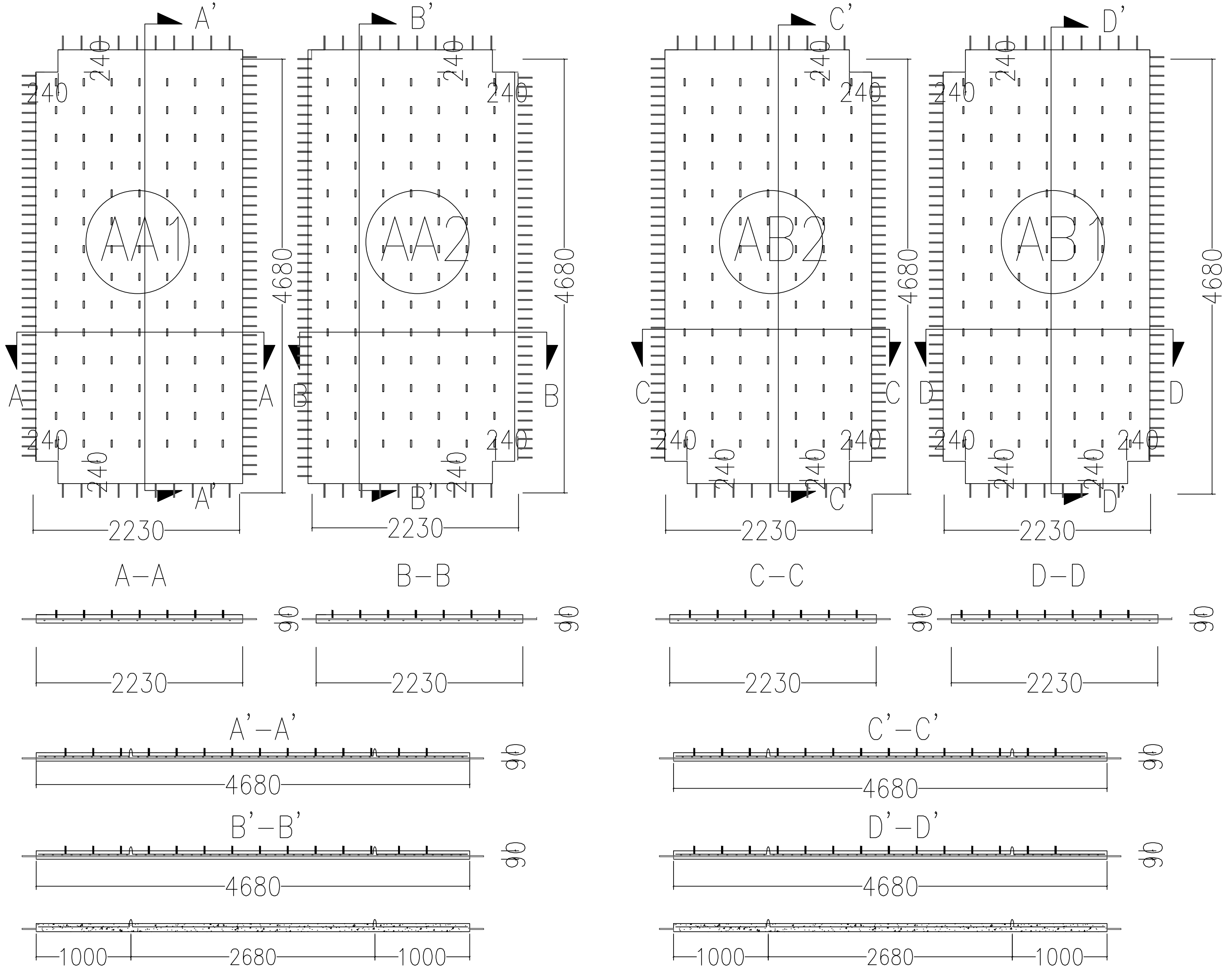


PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR LB3, LB4, LC1 (BEFORE MONOLITH CONDITION)
SCALE 1:20

SHEAR CONECTOR DETAILING
SCALE 1:5

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:20	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
27	61

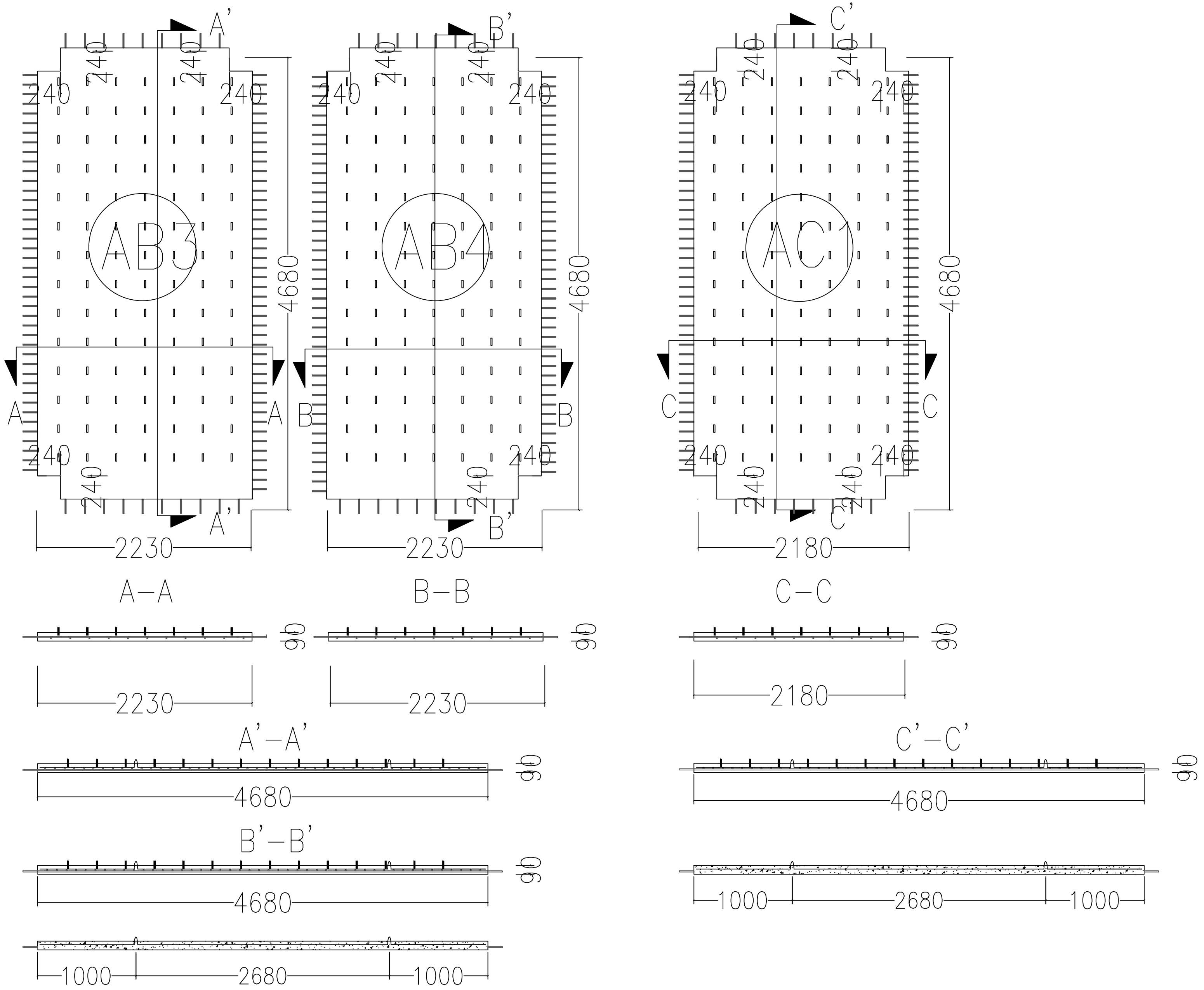


PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR AA1, AA2, AB1, AB2 (BEFORE MONOLITH CONDITION)
SCALE 1:20

SHEAR CONECTOR DETAILING
SCALE 1:5

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:20	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
28	61

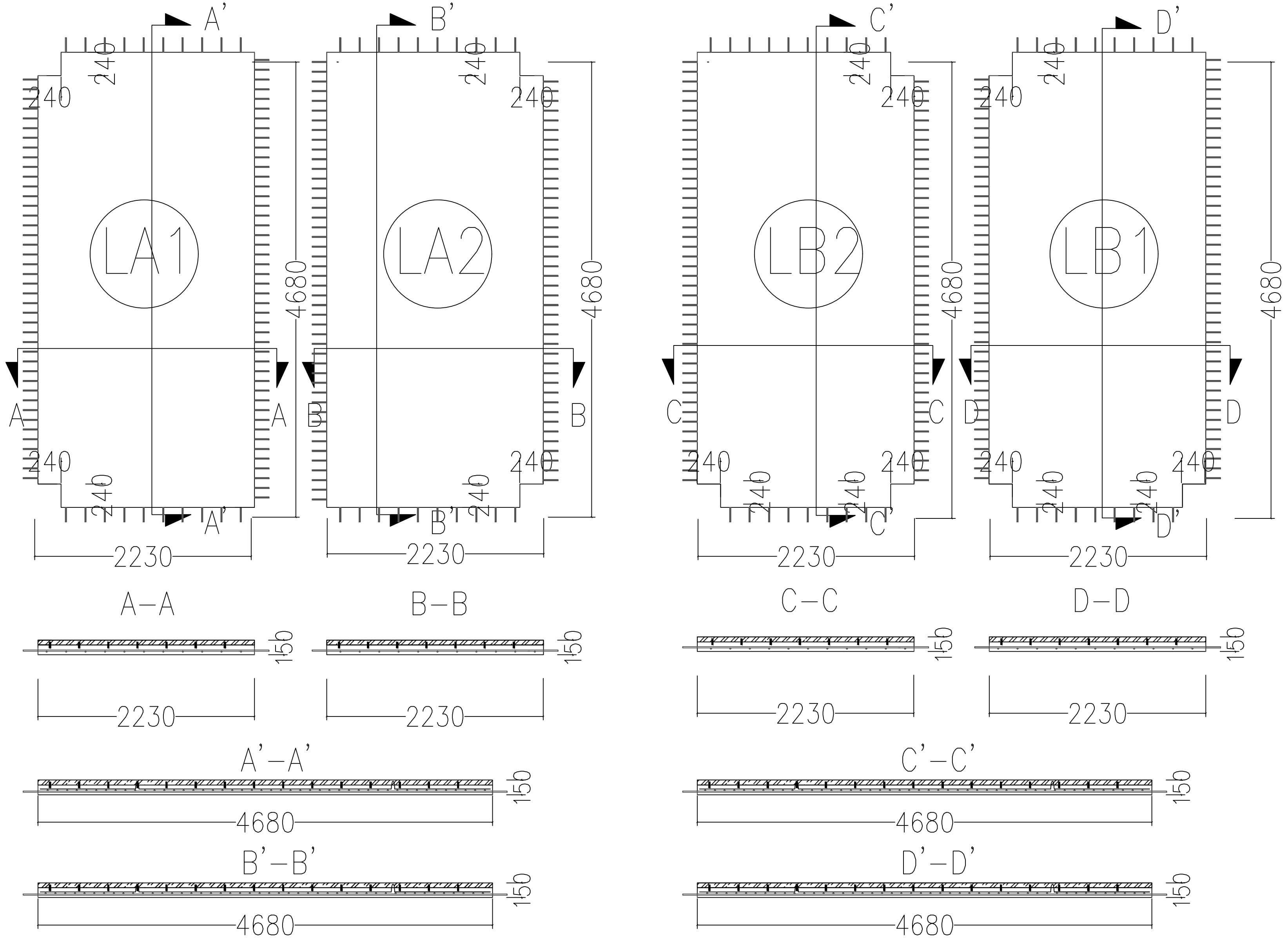


PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR AB3, AB4, AC1 (BEFORE MONOLITH CONDITION)
SCALE 1:20

SHEAR CONECTOR DETAILING
SCALE 1:5

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

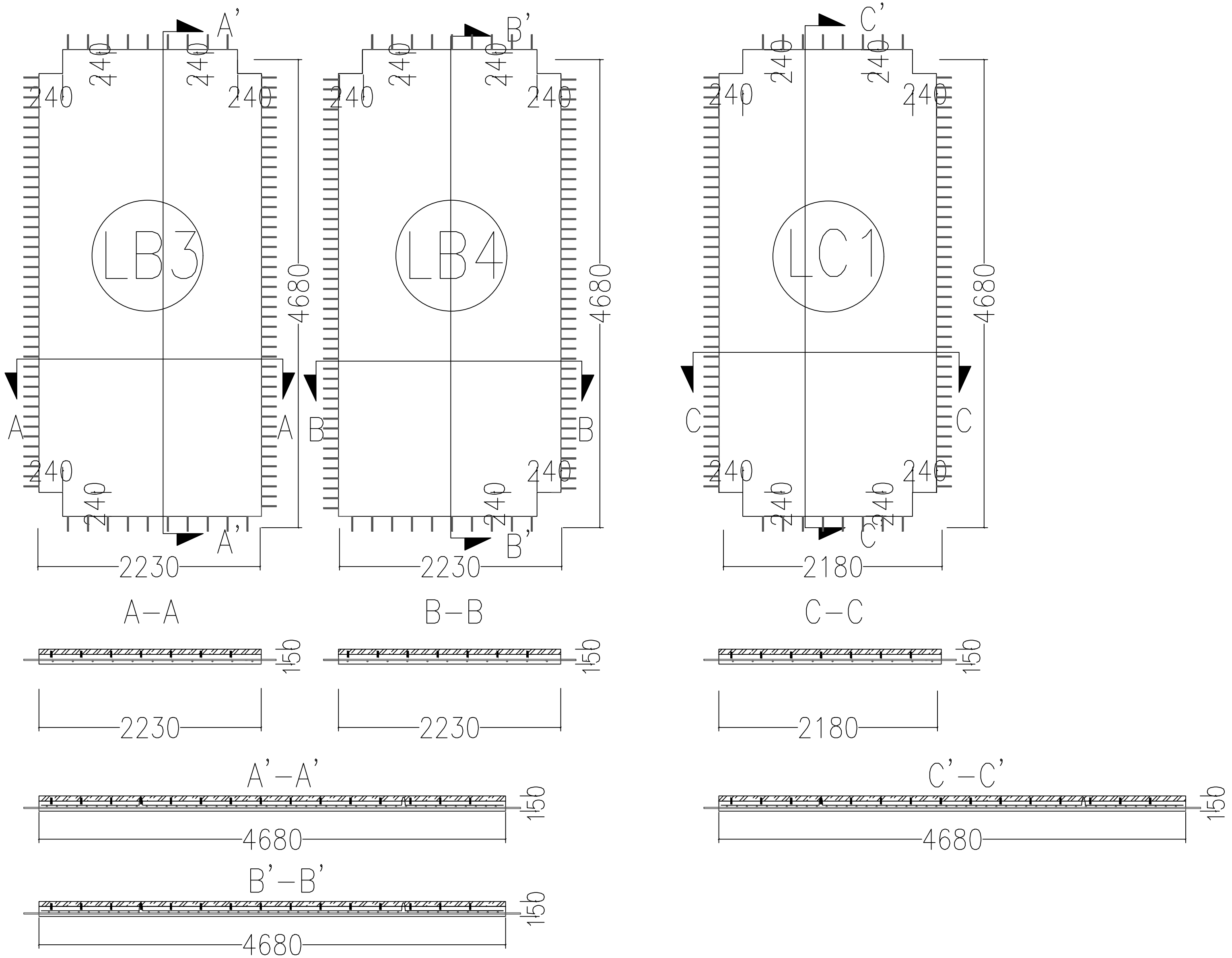
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:20	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
29	61



PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR LA1, LA2, LB1, LB2 (AFTER MONOLITH CONDITION)
SCALE 1:20

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:20	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
30	61



PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR LB3, LB4, LC1 (AFTER MONOLITH CONDITION)
SCALE 1:20

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:20

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

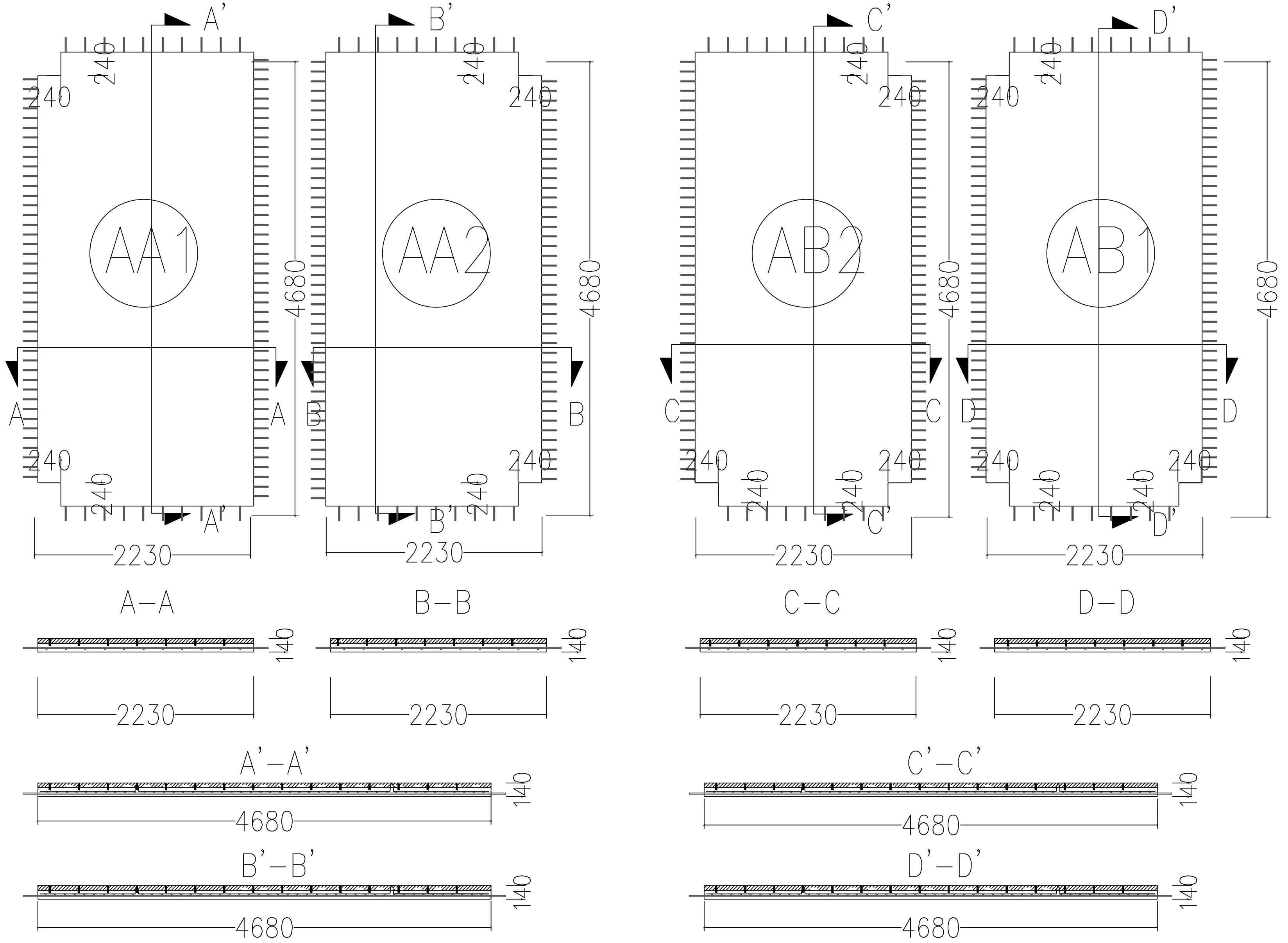
STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

3112100025

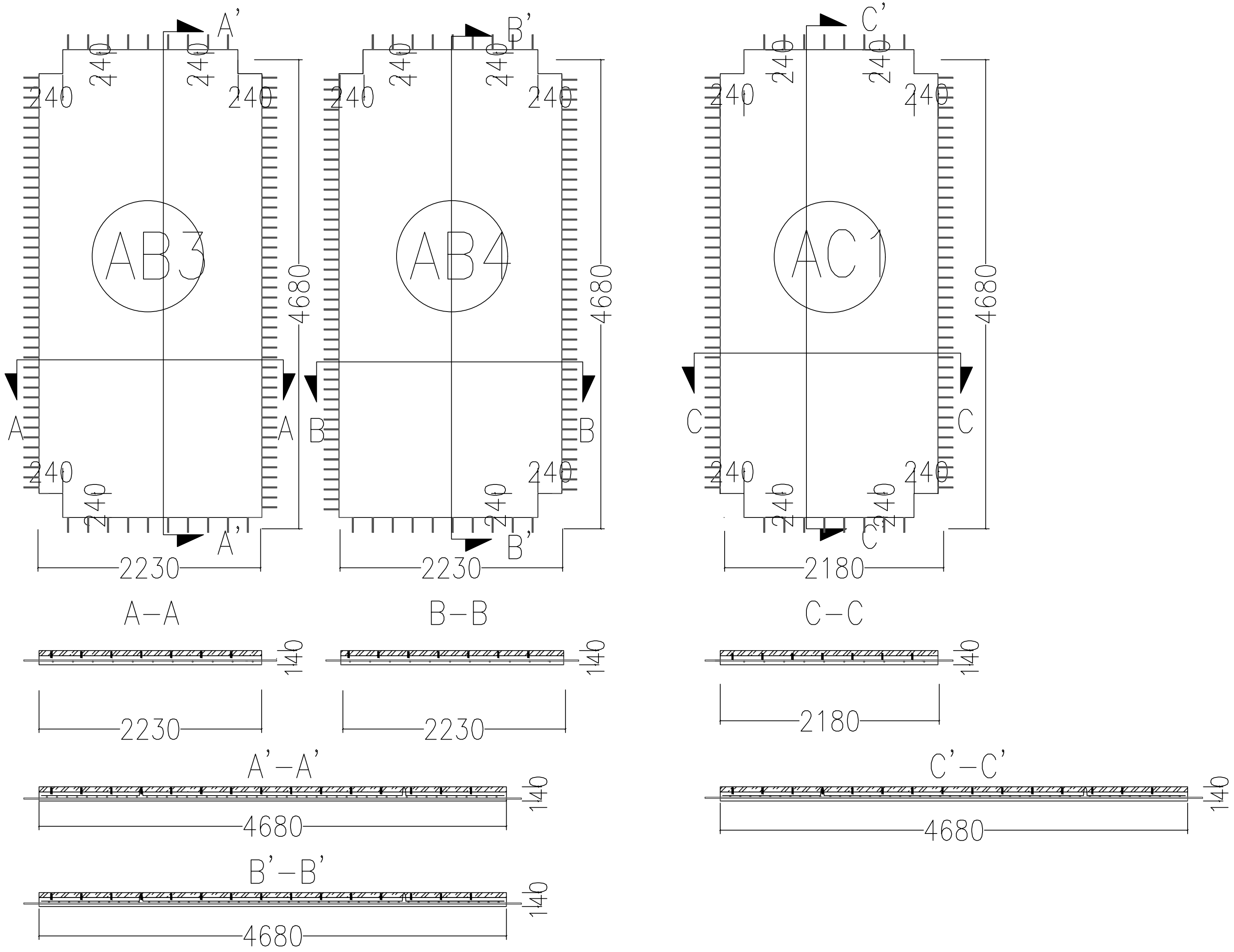
NUMBER	TOTAL
31	61



PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR AA1, AA2, AB1, AB2 (AFTER MONOLITH CONDITION)
SCALE 1:20

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

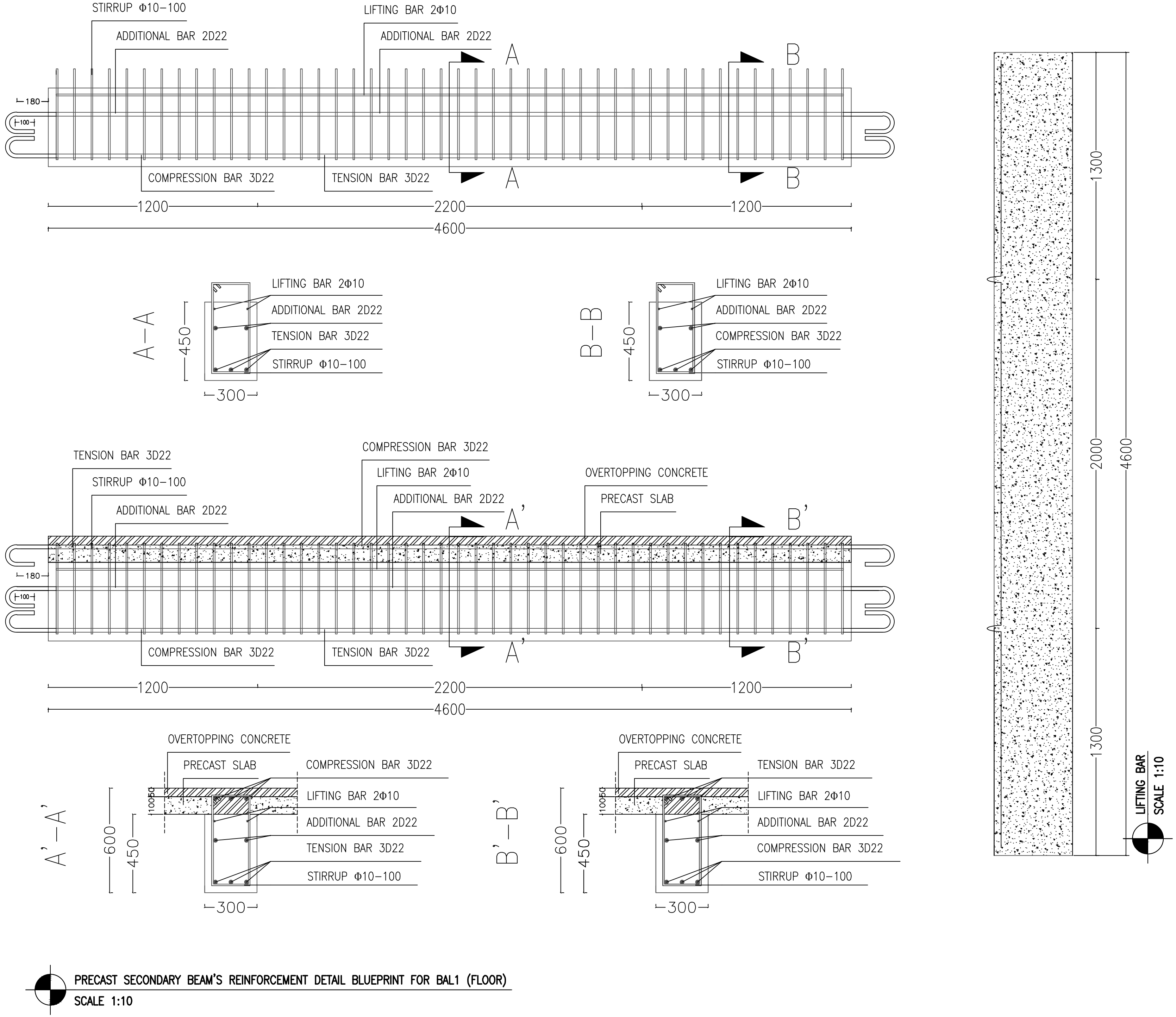
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:20	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
32	61



PRECAST SLAB DESIGNING DETAIL BLUEPRINT FOR AB3, AB4, AC1 (AFTER MONOLITH CONDITION)
SCALE 1:20

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:20	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
33	61



DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:10

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

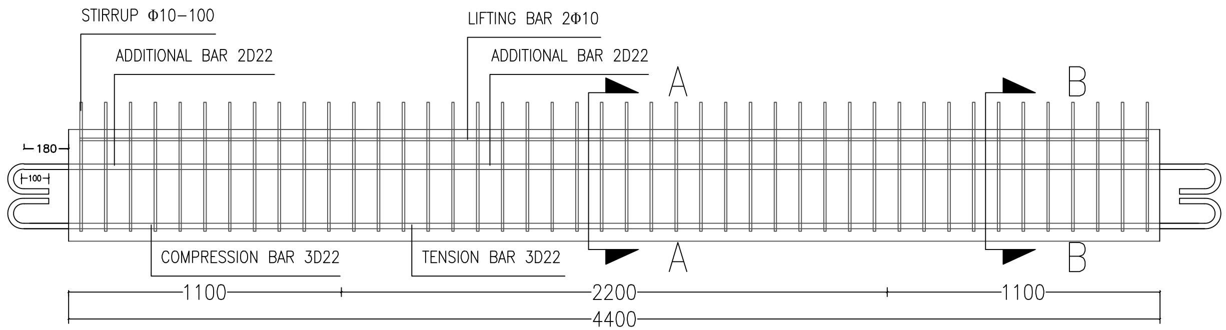
STUDENT's NAME

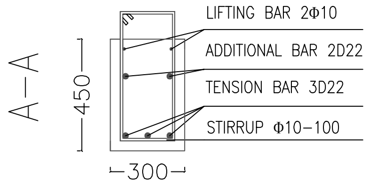
SOCA FAHREZA I.

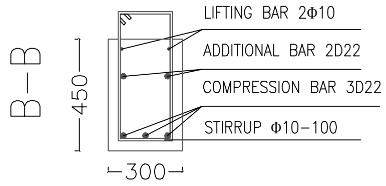
STUDENT's ID NUMBER

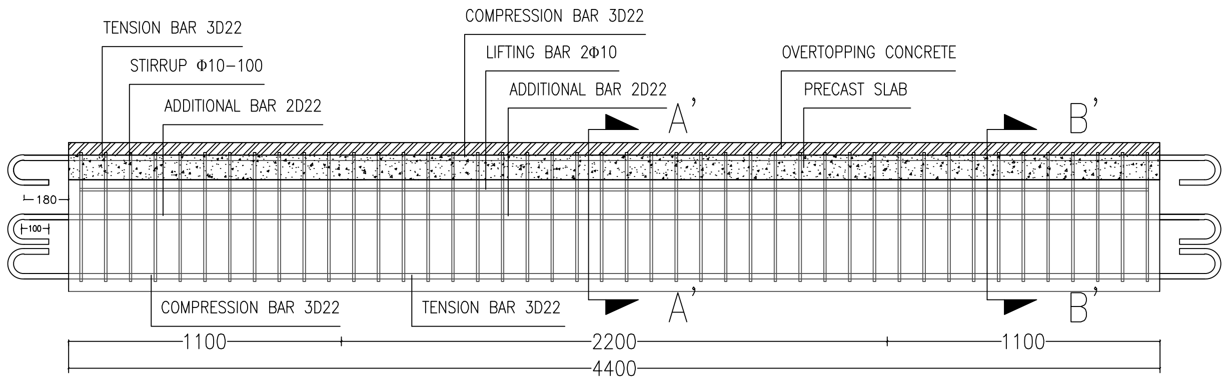
3112100025

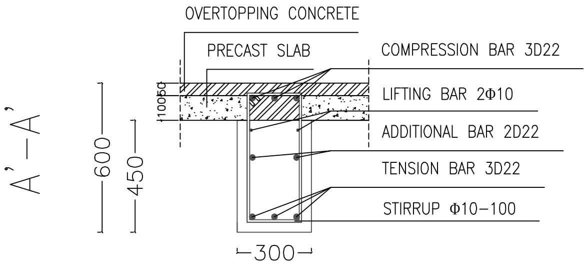
NUMBER	TOTAL
34	61

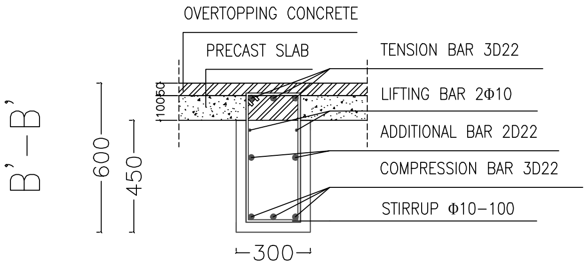


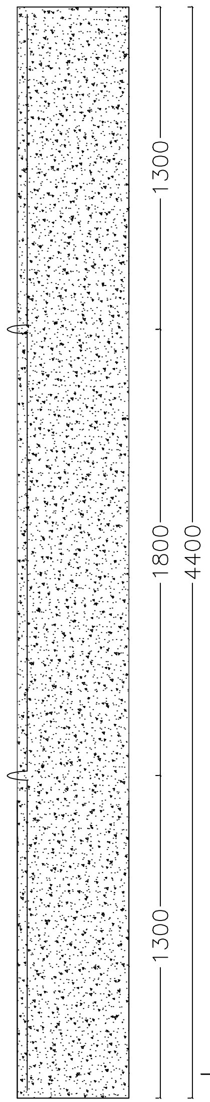












LIFTING BAR
SCALE 1:10

PRECAST SECONDARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BAL2 (FLOOR)
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:10

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

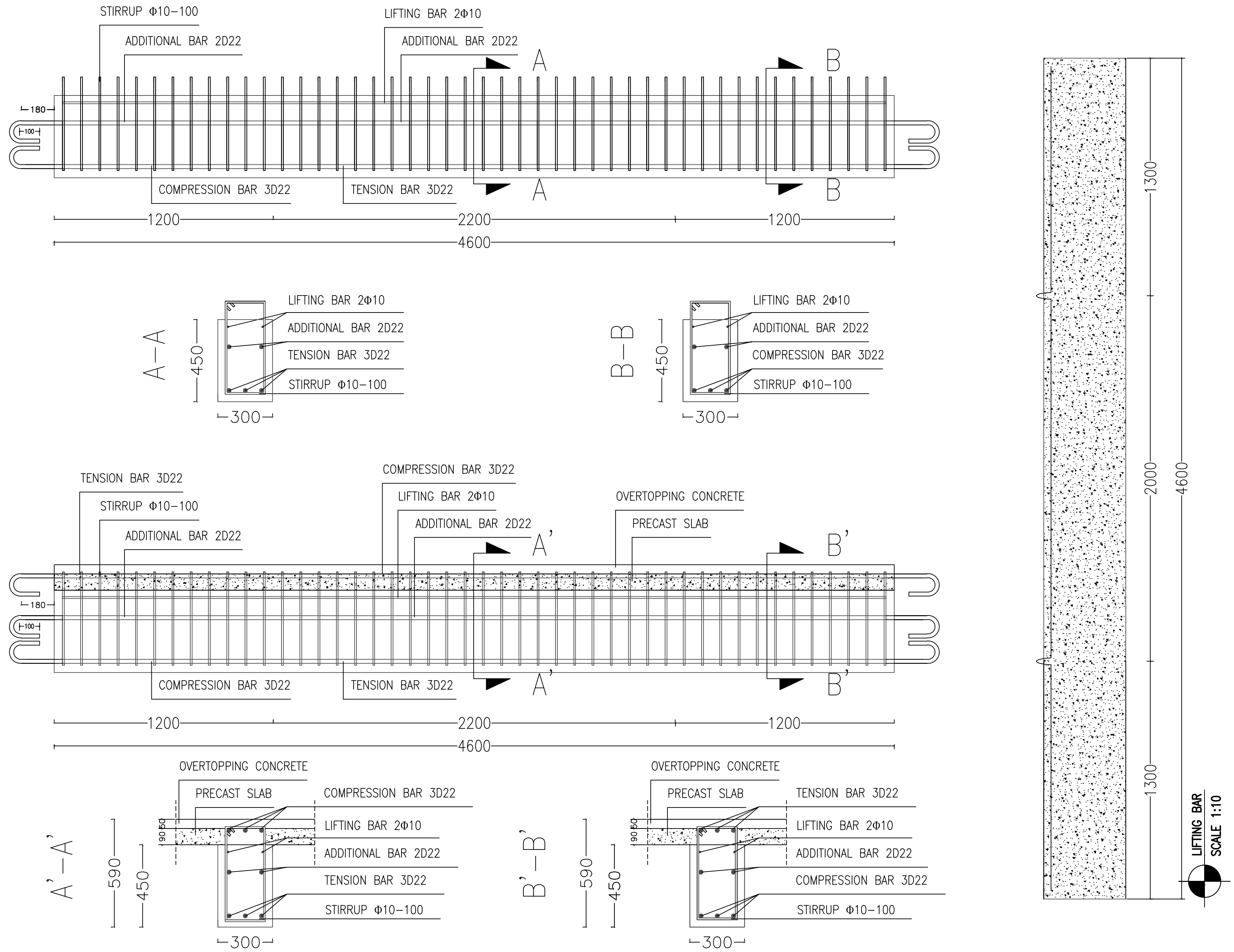
STUDENT's NAME

SOCA FAHREZA I.

STUDENT's ID NUMBER

3112100025

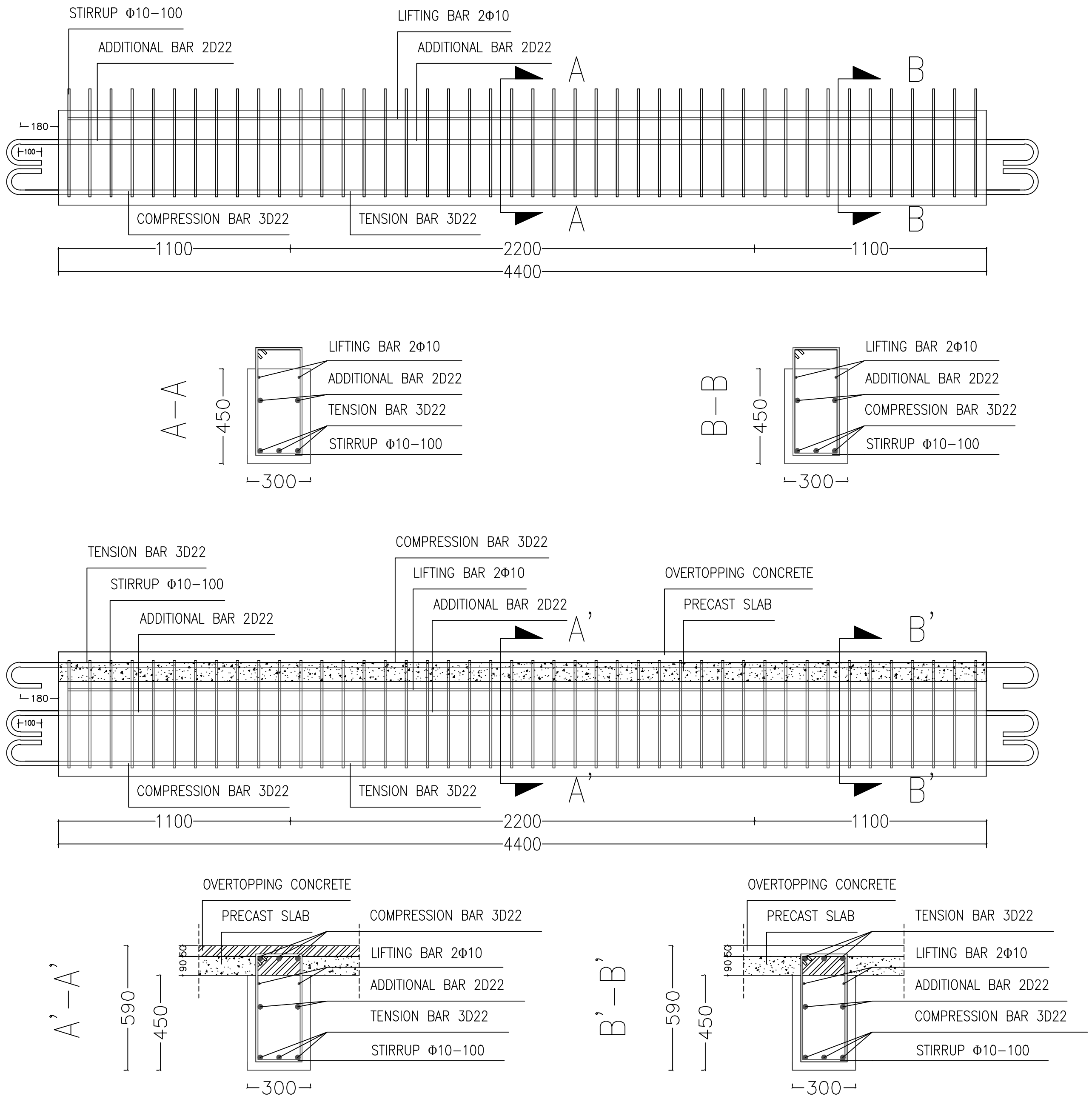
NUMBER	TOTAL
35	61

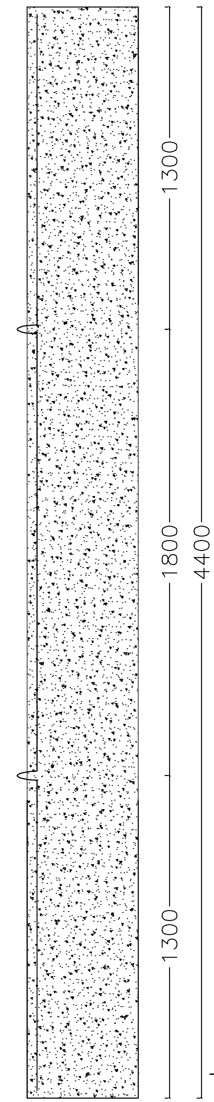


PRECAST SECONDARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BAA1 (ROOF)
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
36	61





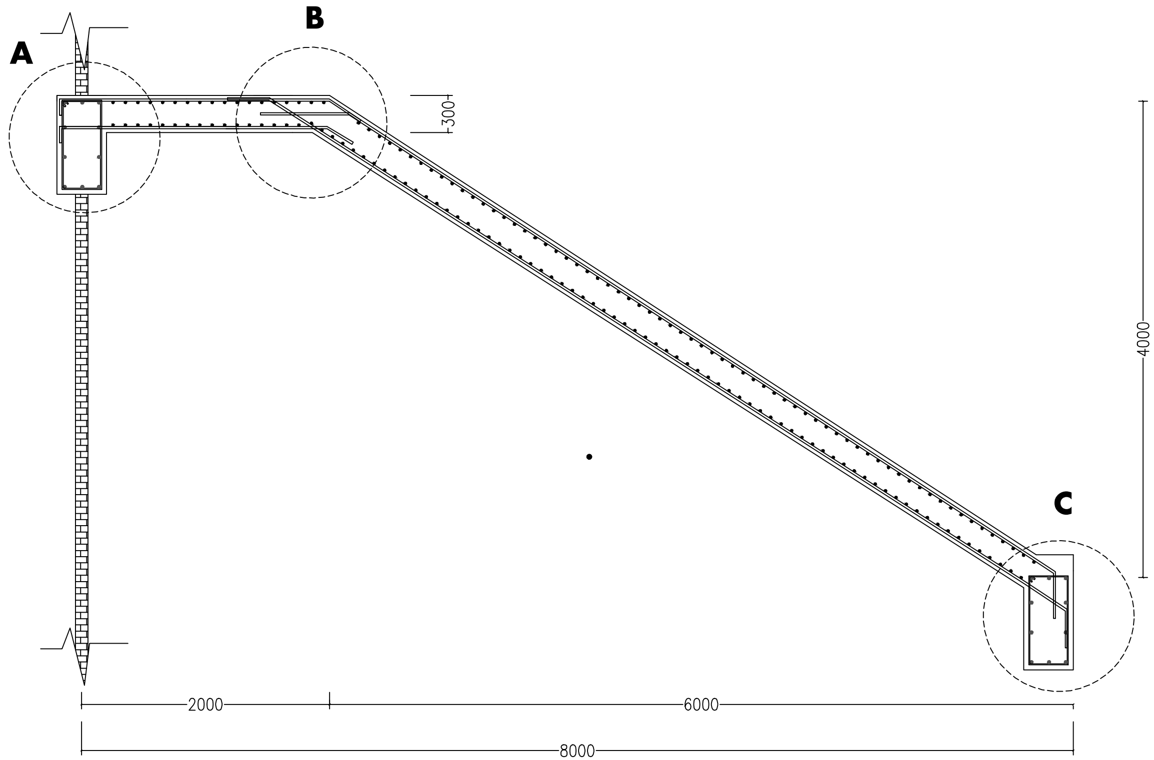
LIFTING BAR
SCALE 1:10

PRECAST SECONDARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BAA2 (ROOF)
SCALE 1:10

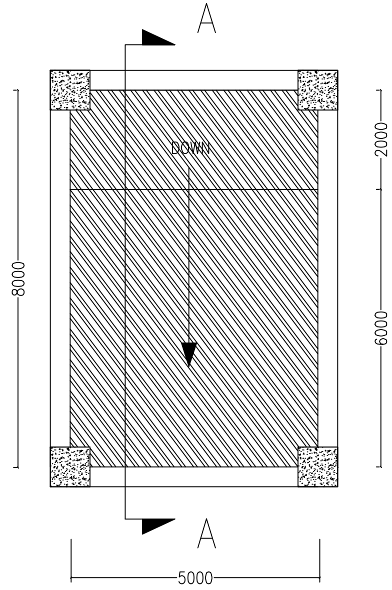
DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME
FINAL PROJECT
TECHNICAL DRAWING
STRUCTURAL
SCALE
1:10
ADVISOR LECTURER
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.
STUDENT's NAME
SOCA FAHREZA I.
STUDENT's ID NUMBER
3112100025

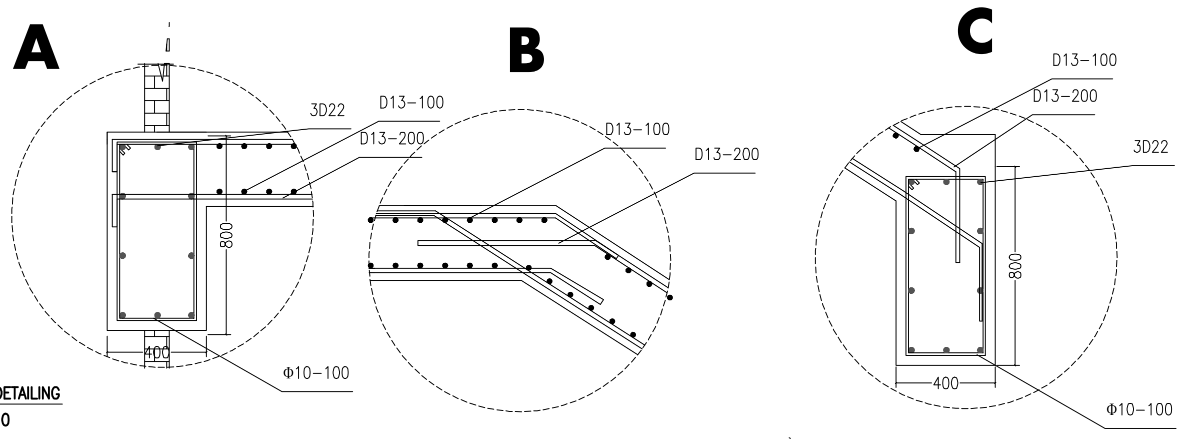
NUMBER	**TOTAL**
37	61



SECTION CUT A-A
SCALE 1:20



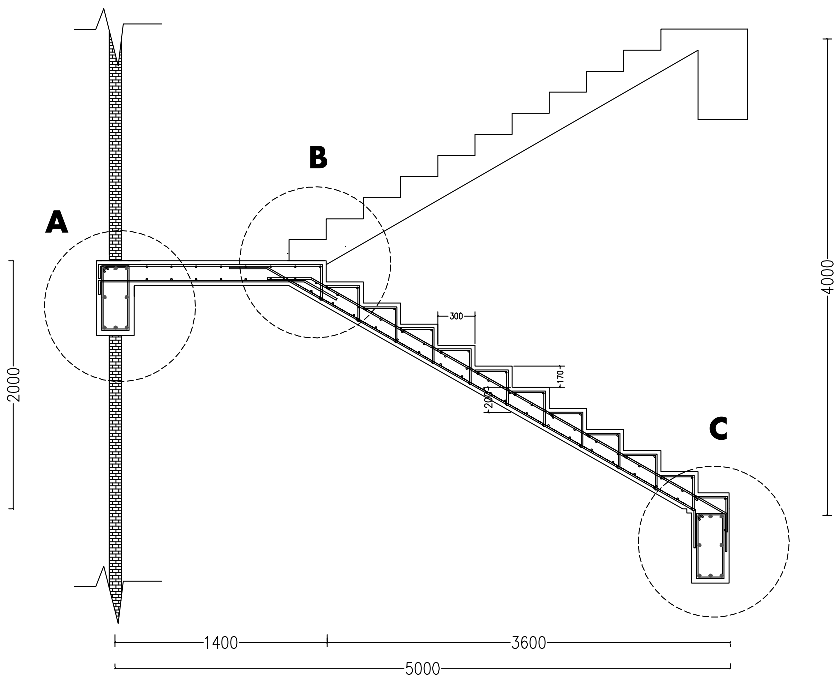
STRUCTURE OF RAMP
SCALE 1:50



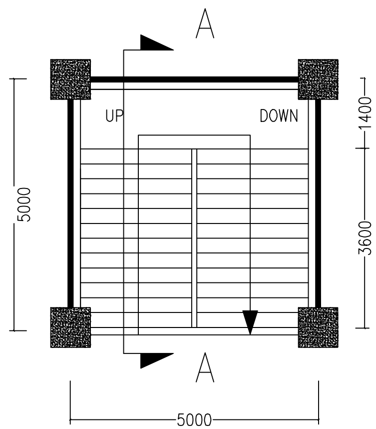
SECTION DETAILING
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

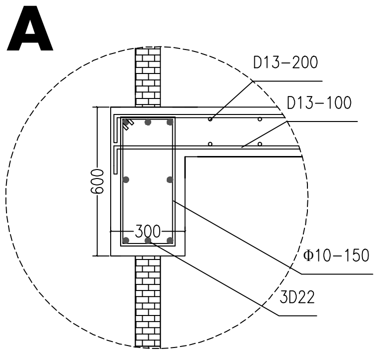
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
38	61

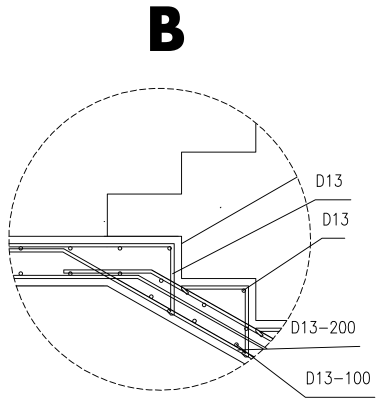


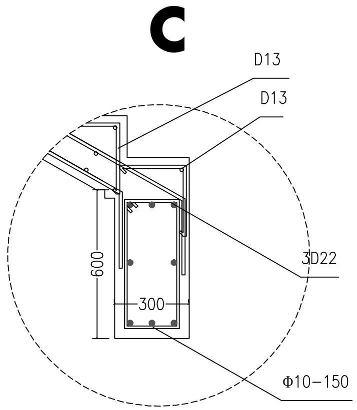
SECTION CUT A–A
SCALE 1:20



STRUCTURE OF STAIR
SCALE 1:50







SECTION DETAILING
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:100

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

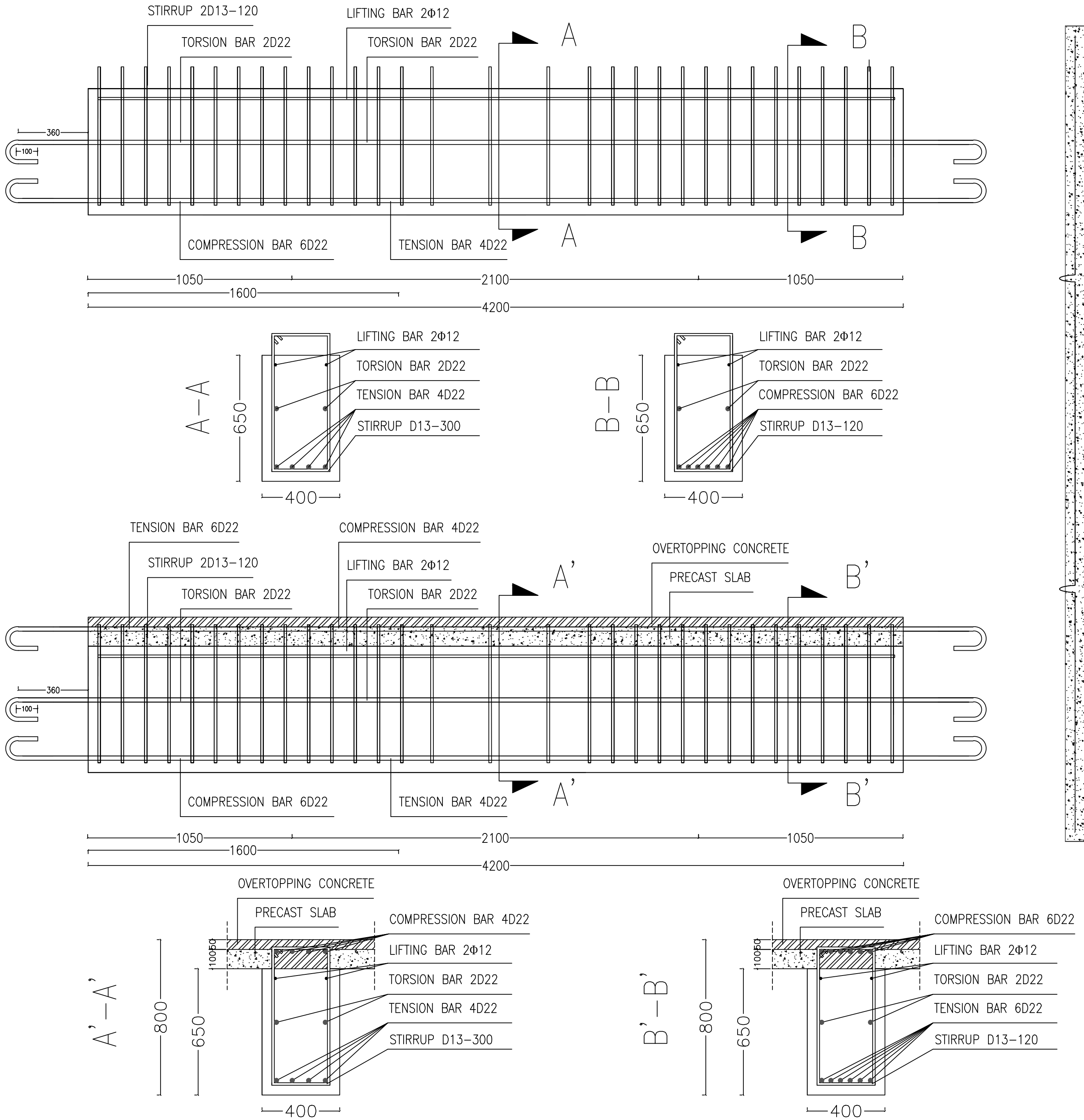
STUDENT's NAME

SOCA FAHREZA I.

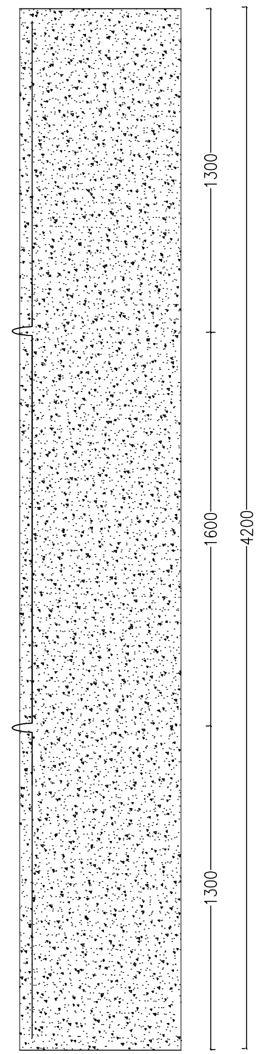
STUDENT's ID NUMBER

3112100025

NUMBER	TOTAL
39	61



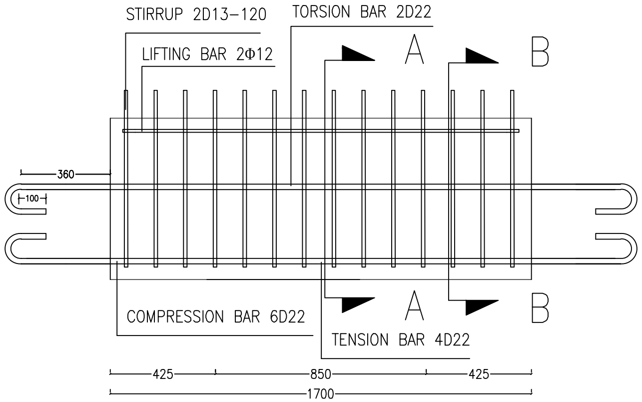
PRECAST TRANSVERSAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIL1 (FLOOR)
SCALE 1:10

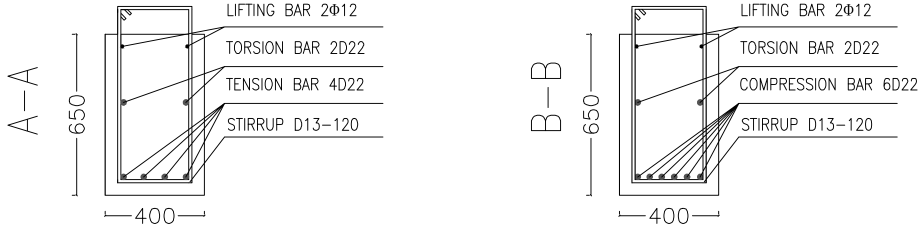


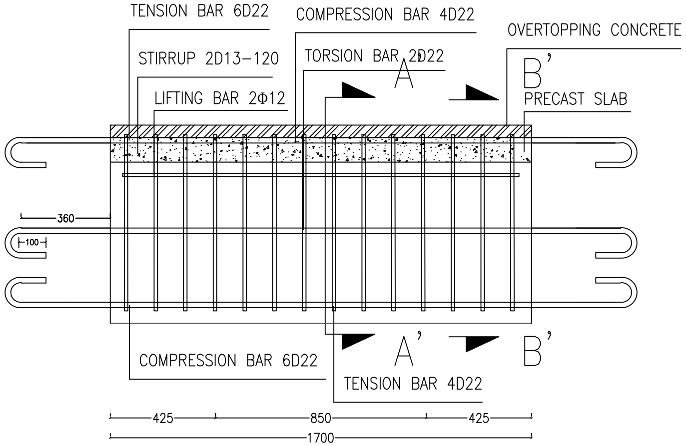
LIFTING BAR
SCALE 1:10

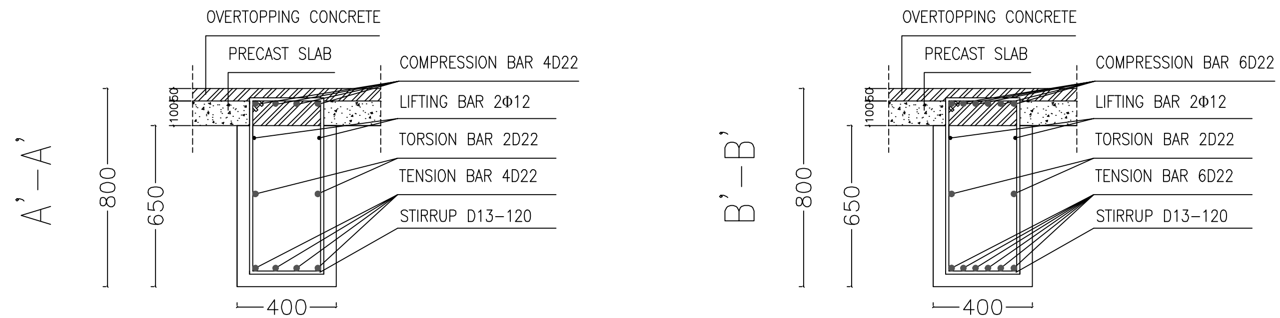
DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

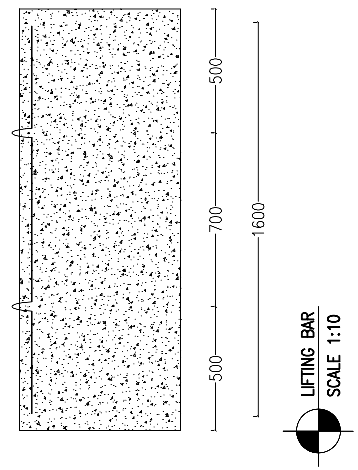
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
40	61







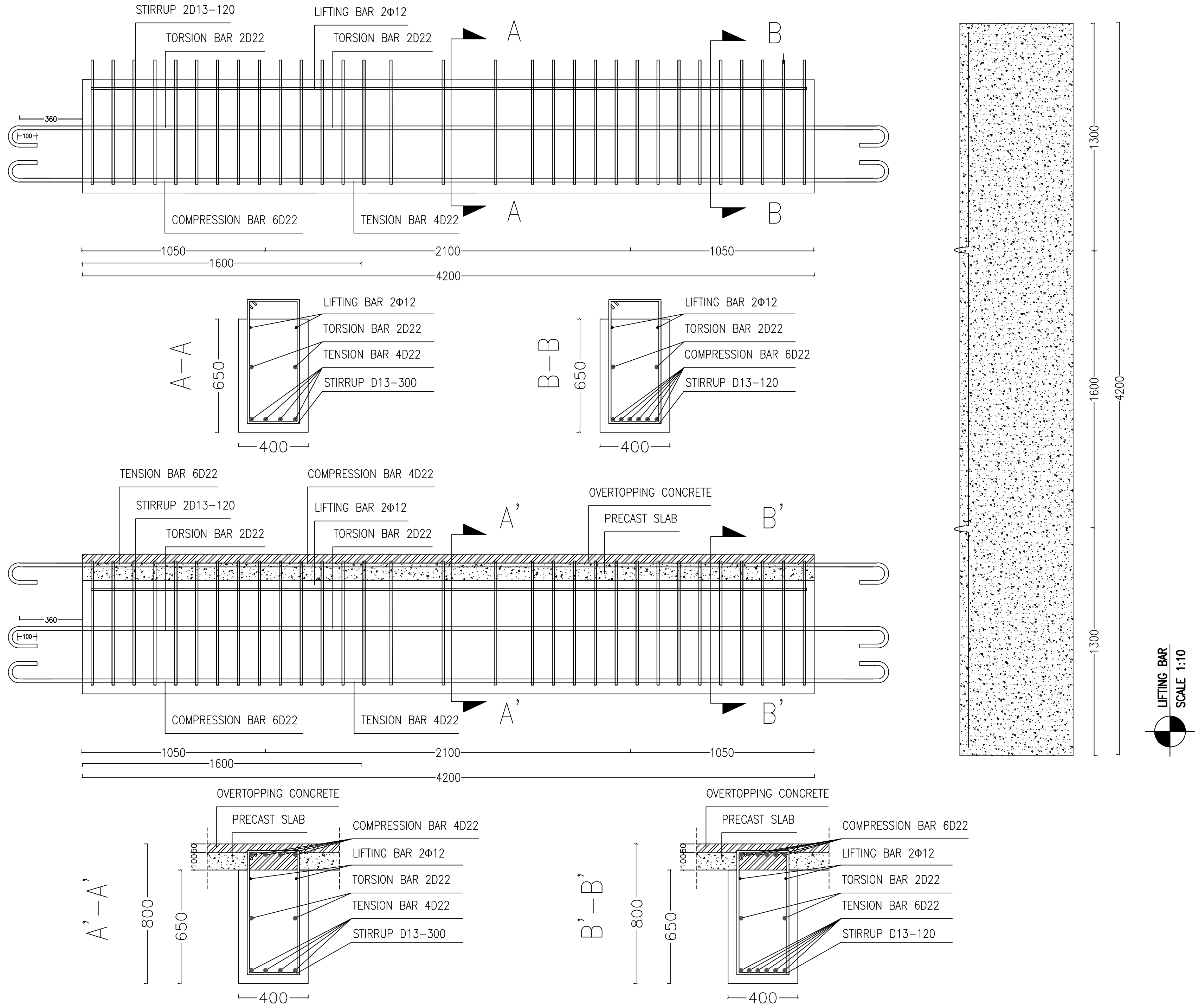




PRECAST TRANSVERSAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIL2 (FLOOR)
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

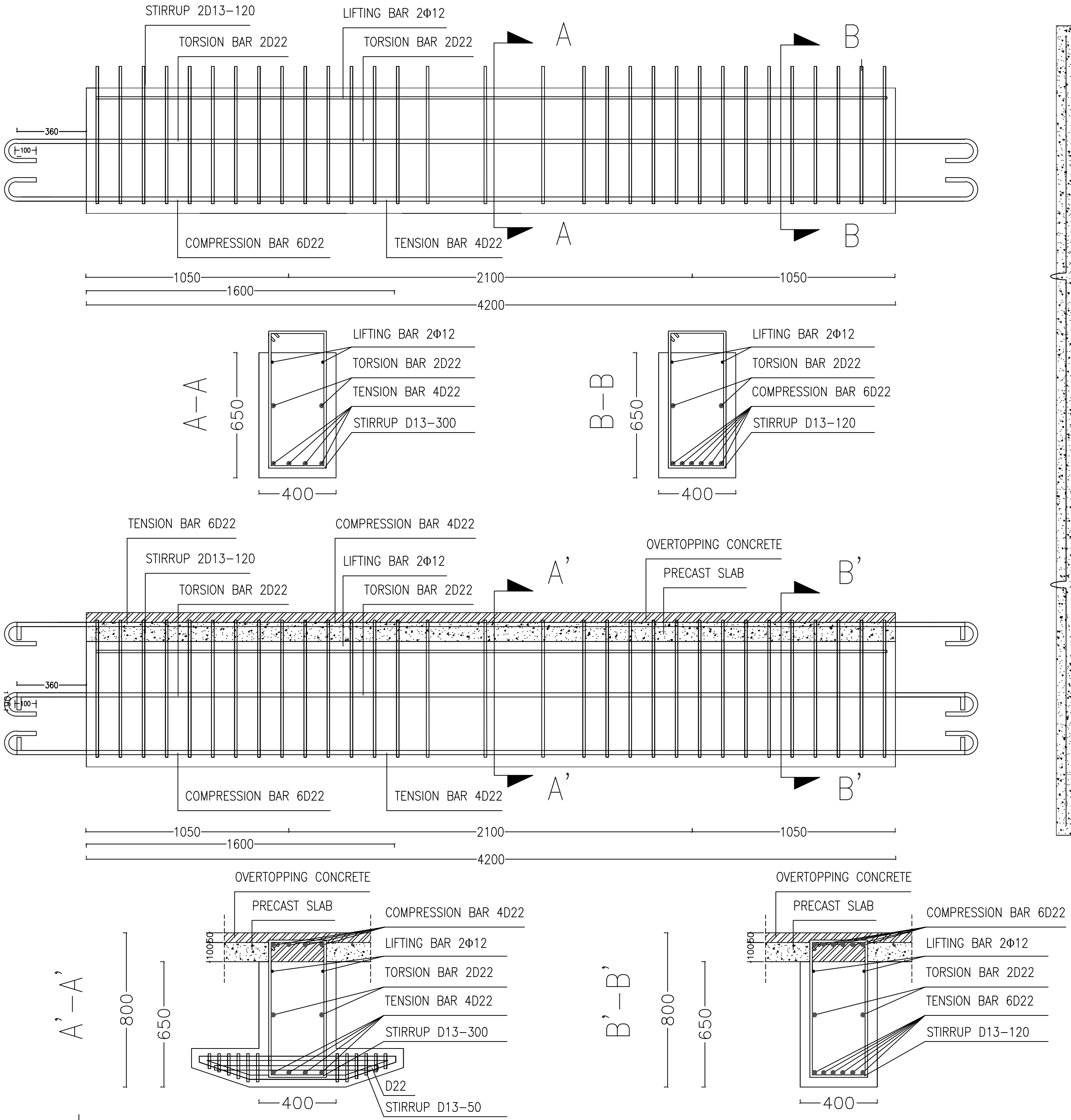
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
41	61



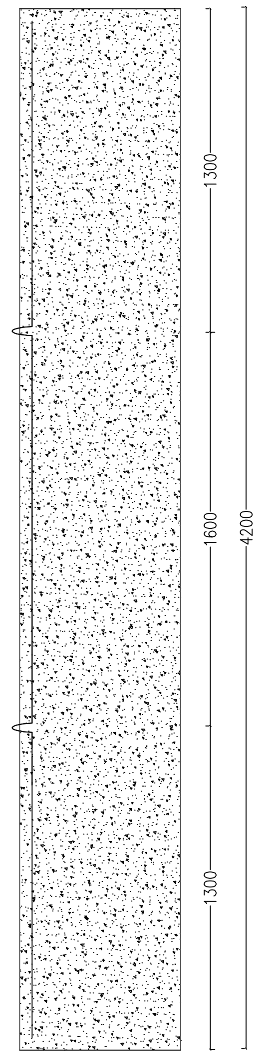
PRECAST TRANSVERSAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIL3 (FLOOR)
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
42	61



PRECAST INTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIL1 (FLOOR)
SCALE 1:10

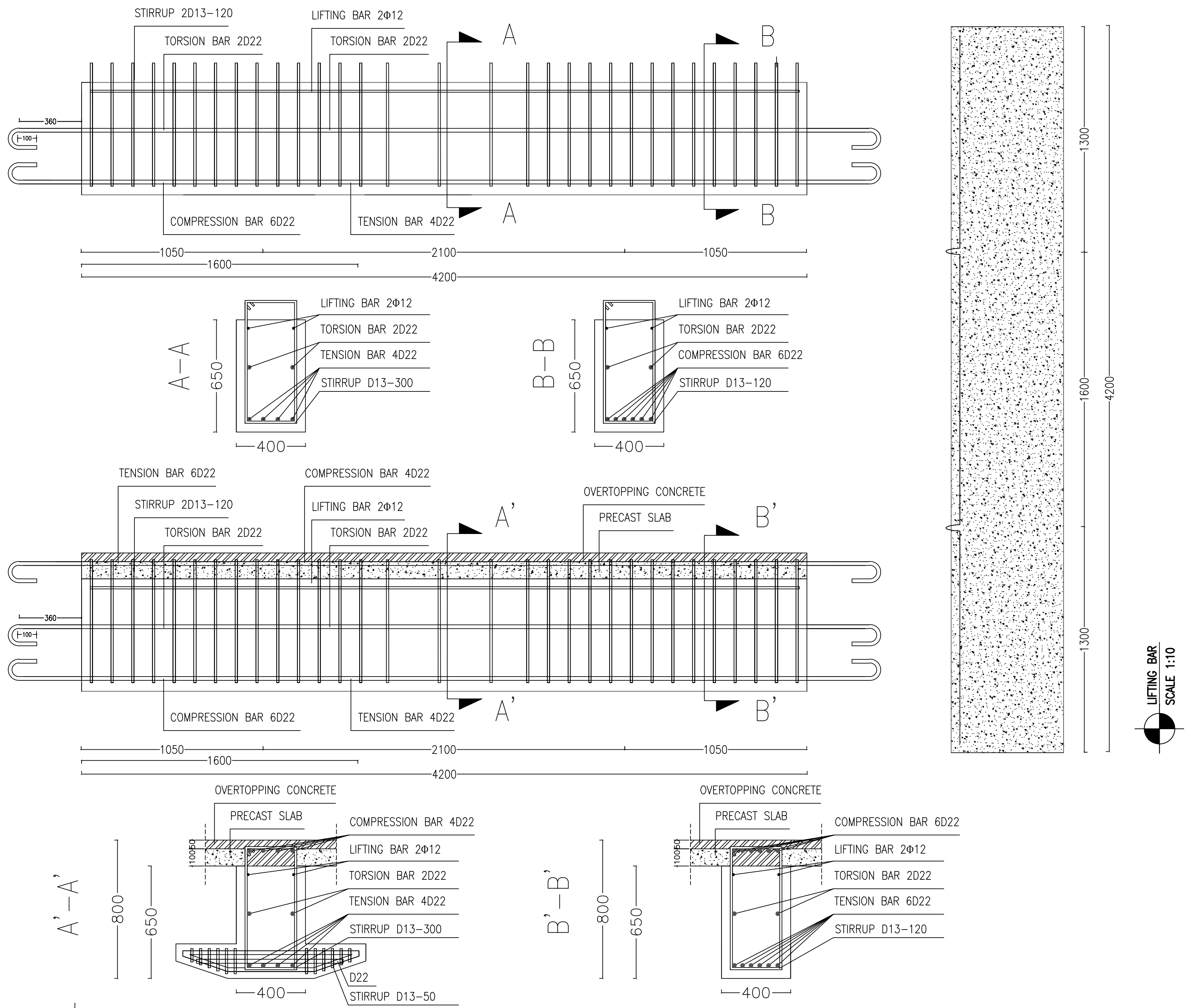


LIFTING BAR
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME
FINAL PROJECT
TECHNICAL DRAWING
STRUCTURAL
SCALE
1:10
ADVISOR LECTURER
DATA IRANATA, ST, MT, PhD.
Dr. Ir. DJOKO IRAWAN, MS.
STUDENT's NAME
SOCA FAHREZA I.
STUDENT's ID NUMBER
3112100025

NUMBER	TOTAL
43	61

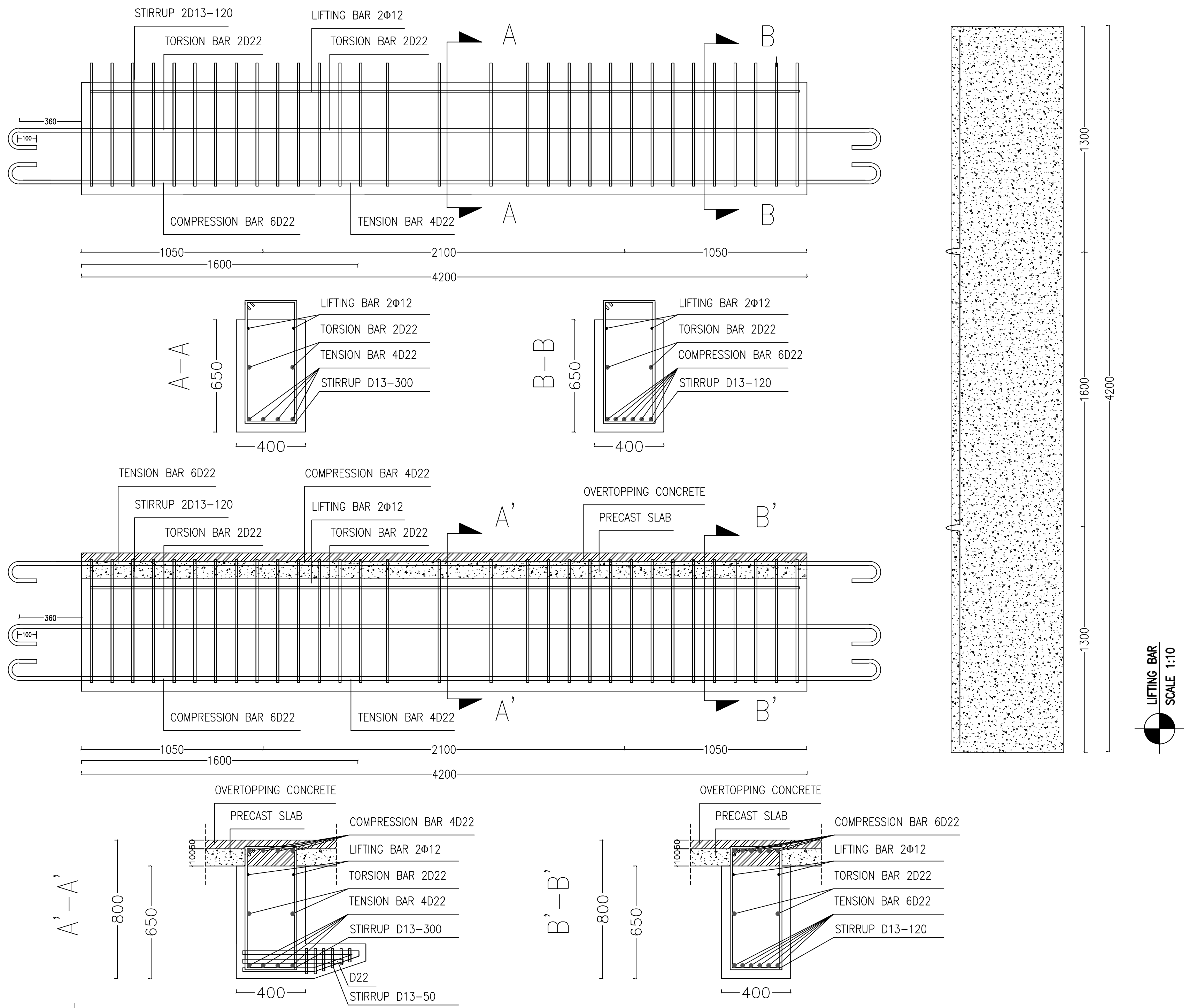


PRECAST INTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIL3 (FLOOR)

SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

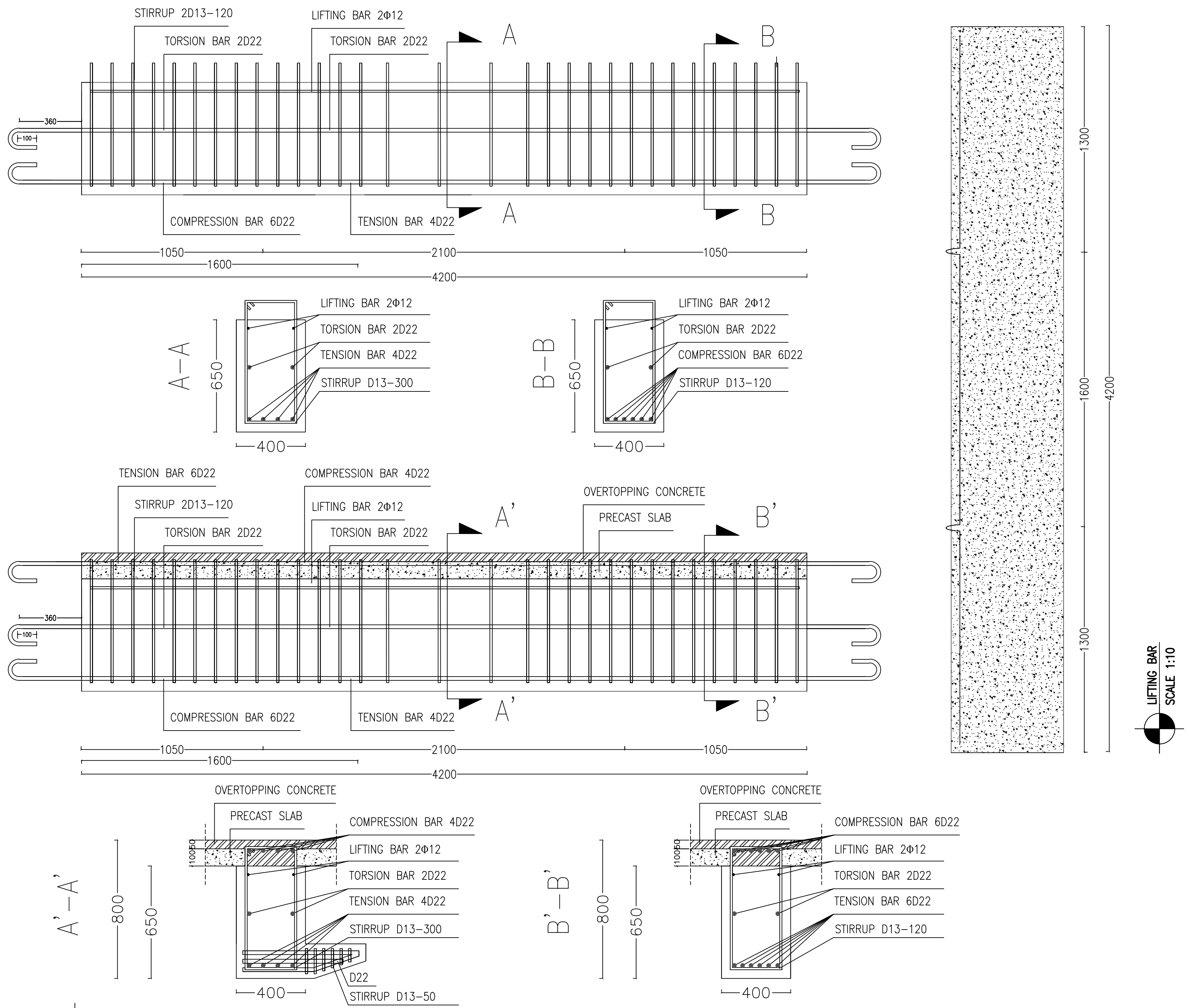
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
44	61



PRECAST EXTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIL1 (FLOOR)
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
45	61

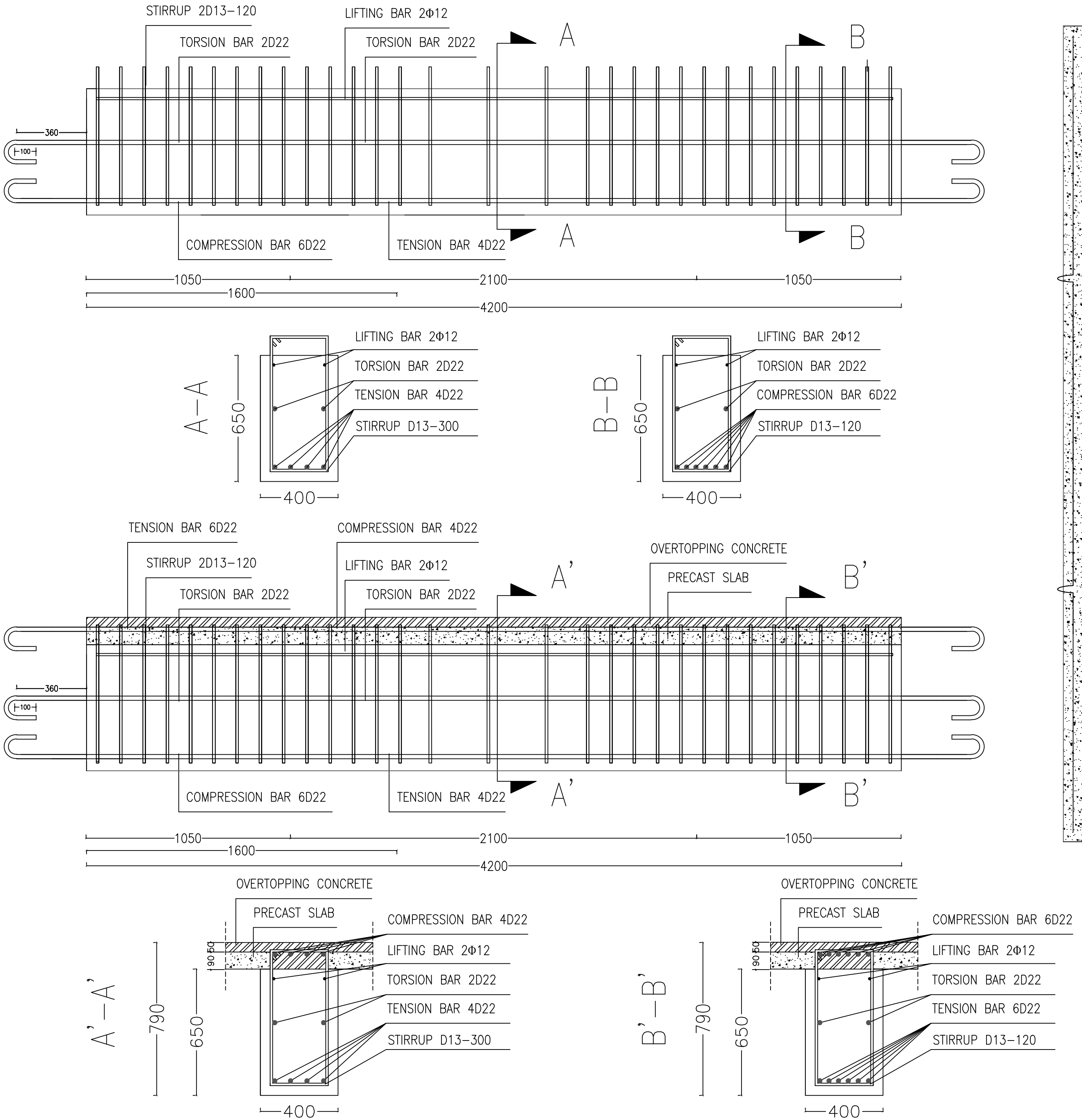


PRECAST EXTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIL3 (FLOOR)

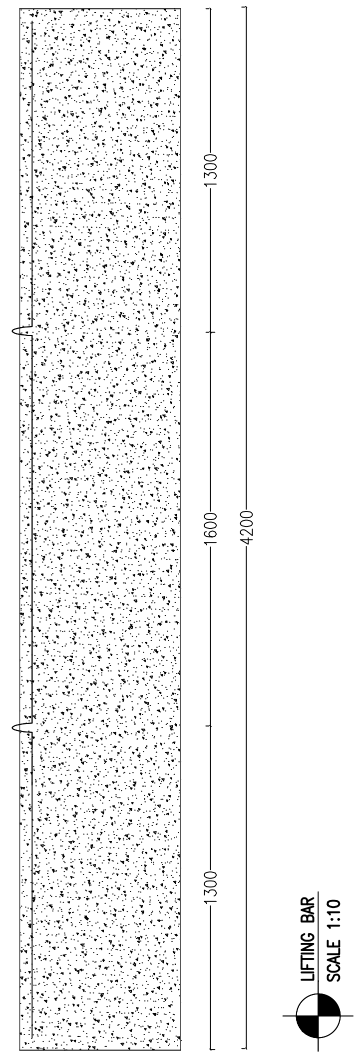
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
46	61



PRECAST TRANSVERSAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIA1 (ROOF)
SCALE 1:10



LIFTING BAR
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME

FINAL PROJECT

TECHNICAL DRAWING

STRUCTURAL

SCALE

1:10

ADVISOR LECTURER

DATA IRANATA, ST, MT, PhD.

Dr. Ir. DJOKO IRAWAN, MS.

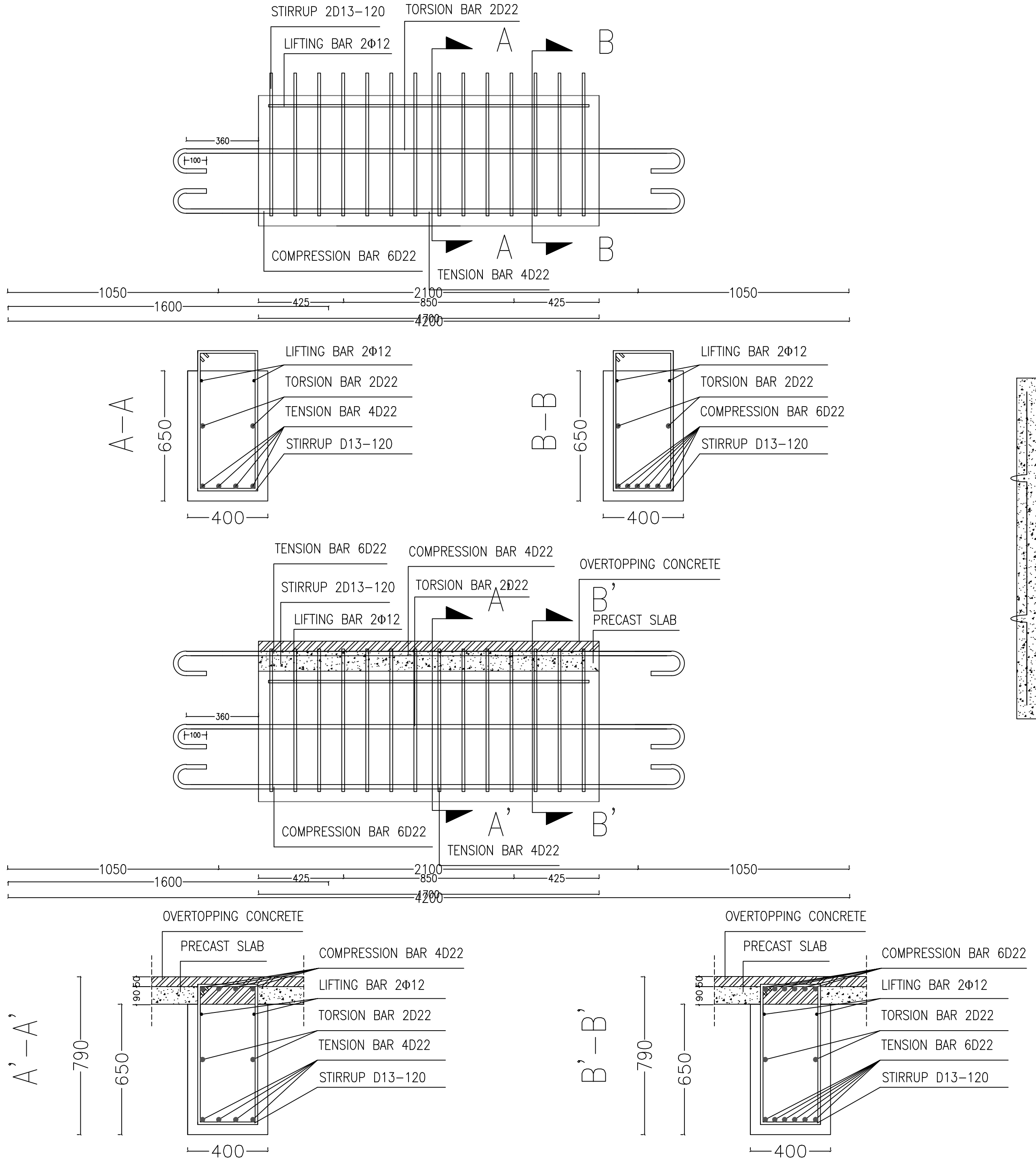
STUDENT's NAME

SOCA FAHREZA I.

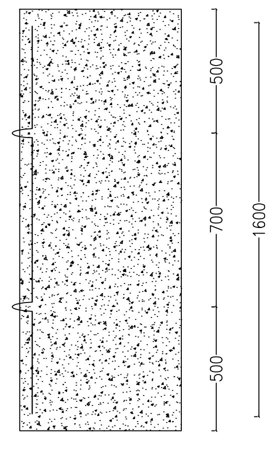
STUDENT's ID NUMBER

3112100025

NUMBER	TOTAL
47	61



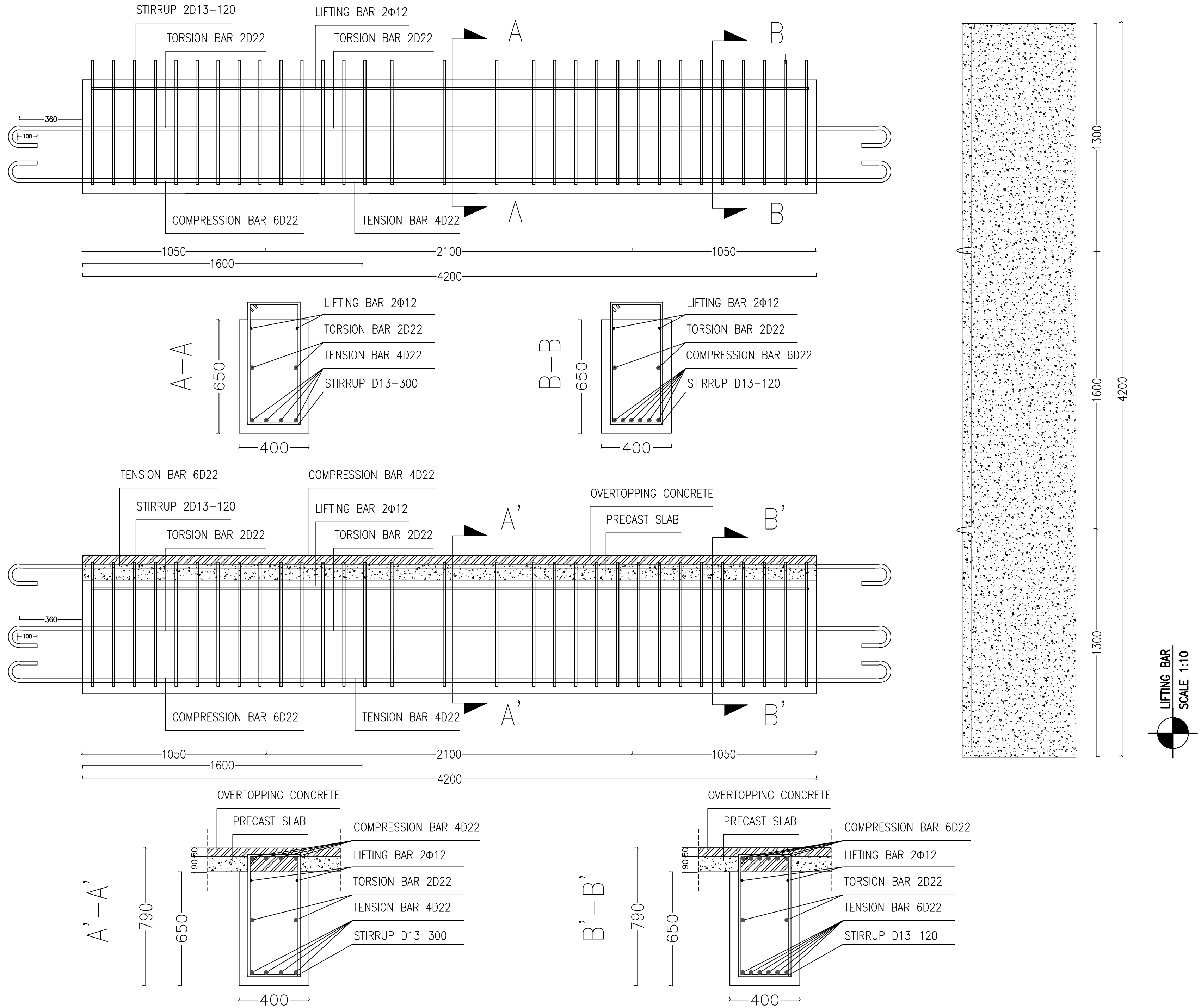
PRECAST TRANSVERSAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIA2 (ROOF)
SCALE 1:10



LIFTING BAR
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:10	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
48	61



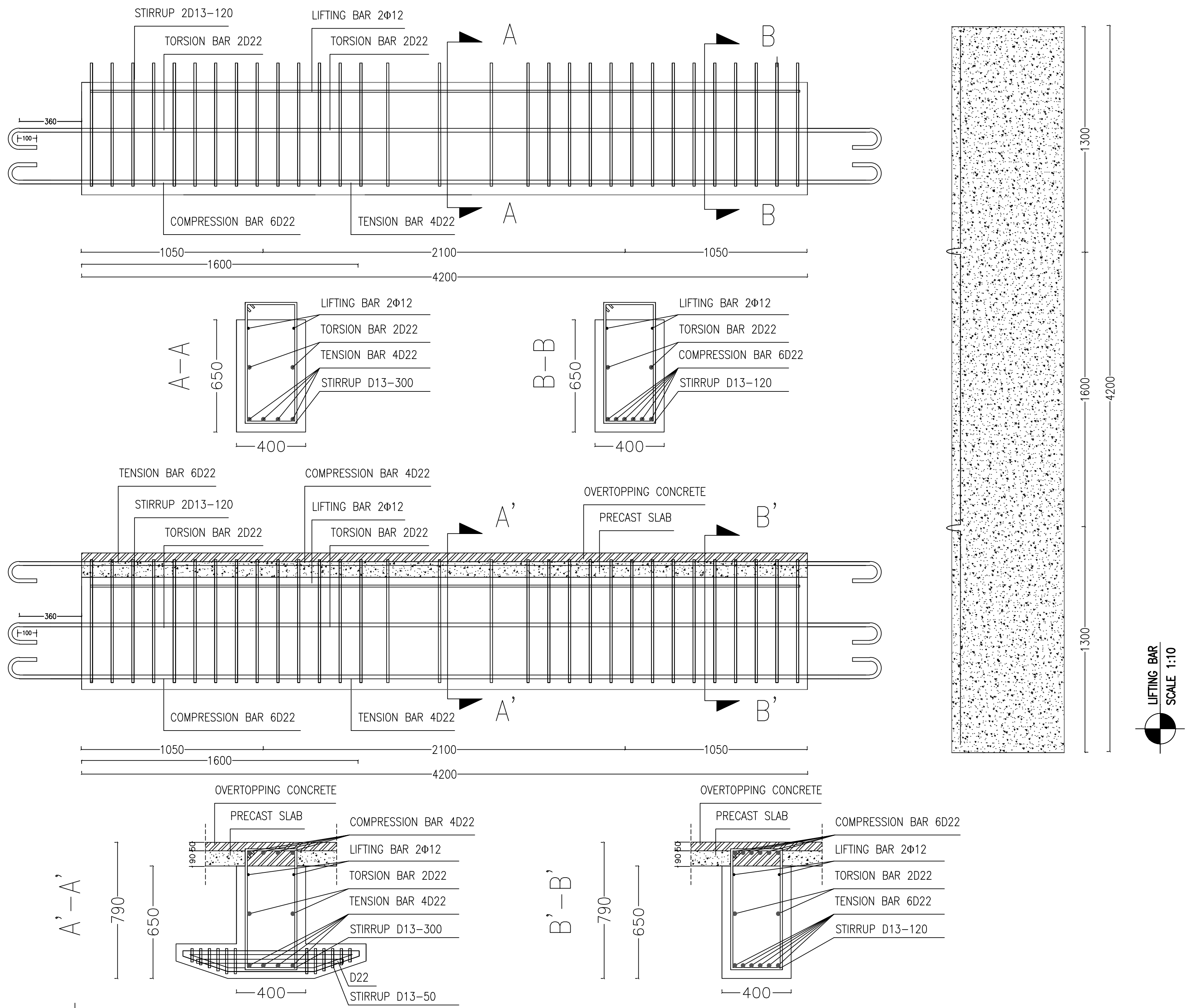
PRECAST TRANSVERSAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIA3 (ROOF)

SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME
FINAL PROJECT
TECHNICAL DRAWING
STRUCTURAL
SCALE
1:100
ADVISOR LECTURER
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.
STUDENT's NAME
SOCA FAHREZA I.
STUDENT's ID NUMBER
3112100025

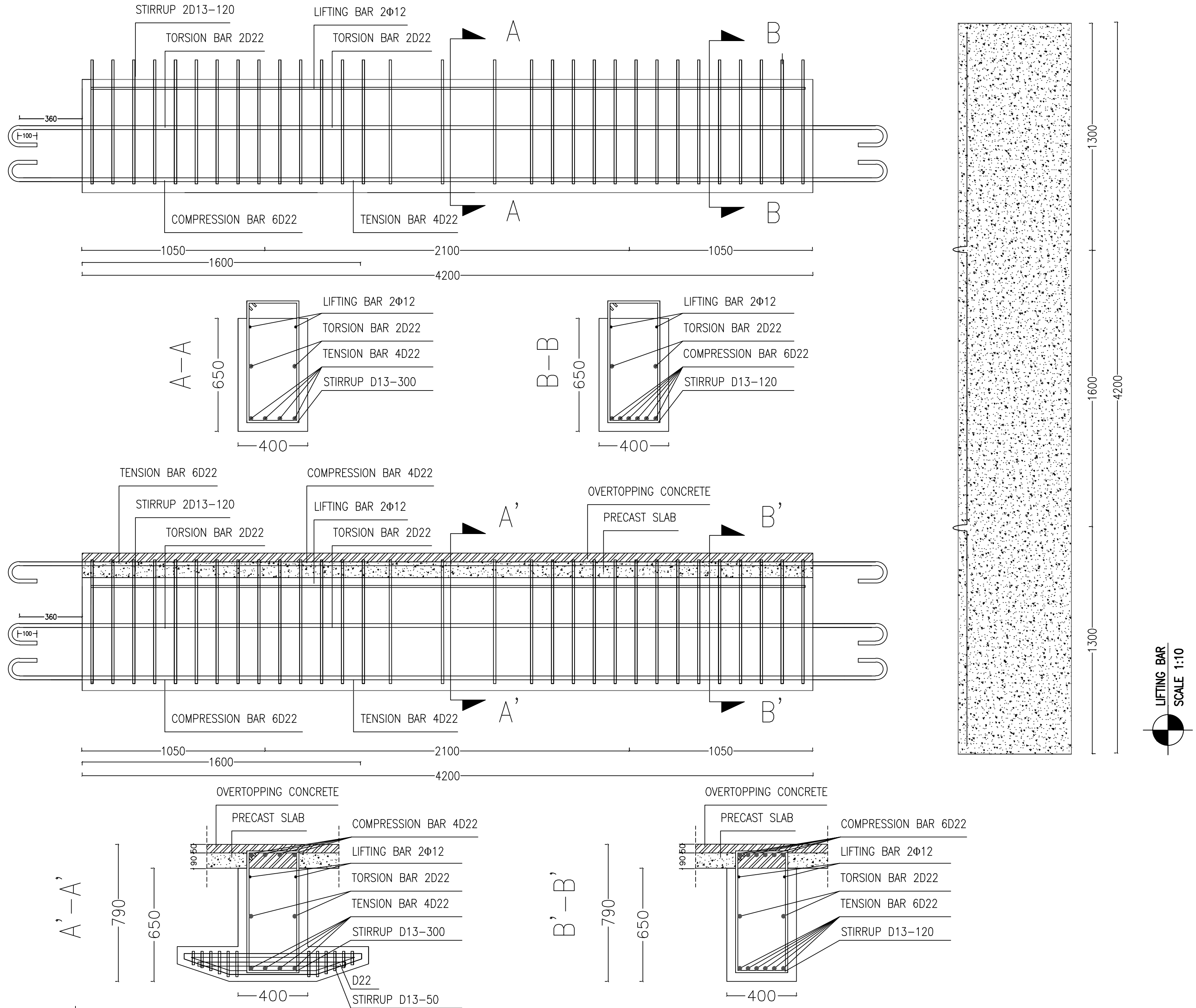
NUMBER	**TOTAL**
49	61



PRECAST INTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIA1 (ROOF)
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

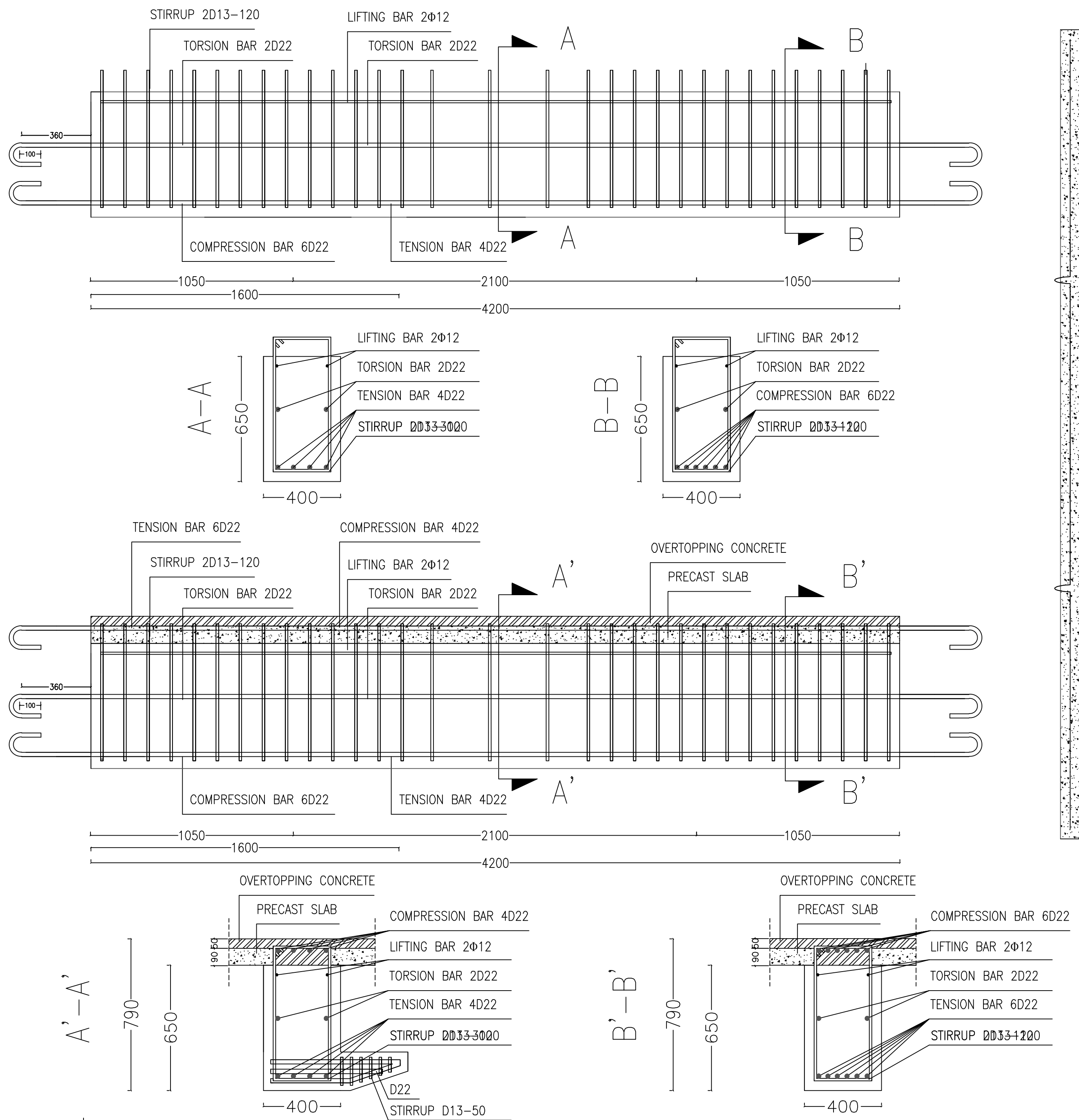
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
50	61



PRECAST INTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIA3 (ROOF)
SCALE 1:10

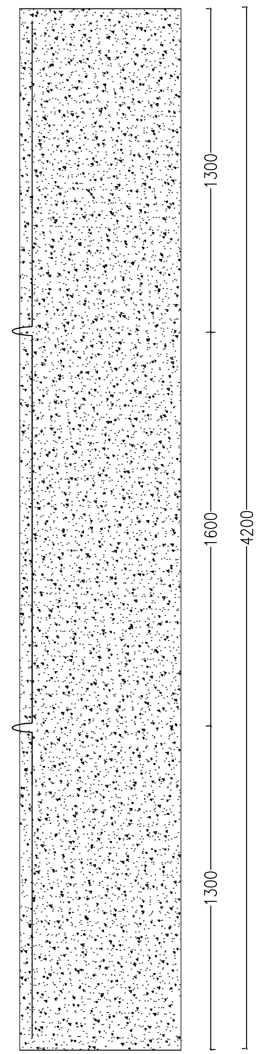
DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
51	61



PRECAST EXTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIA1 (ROOF)

SCALE 1:10

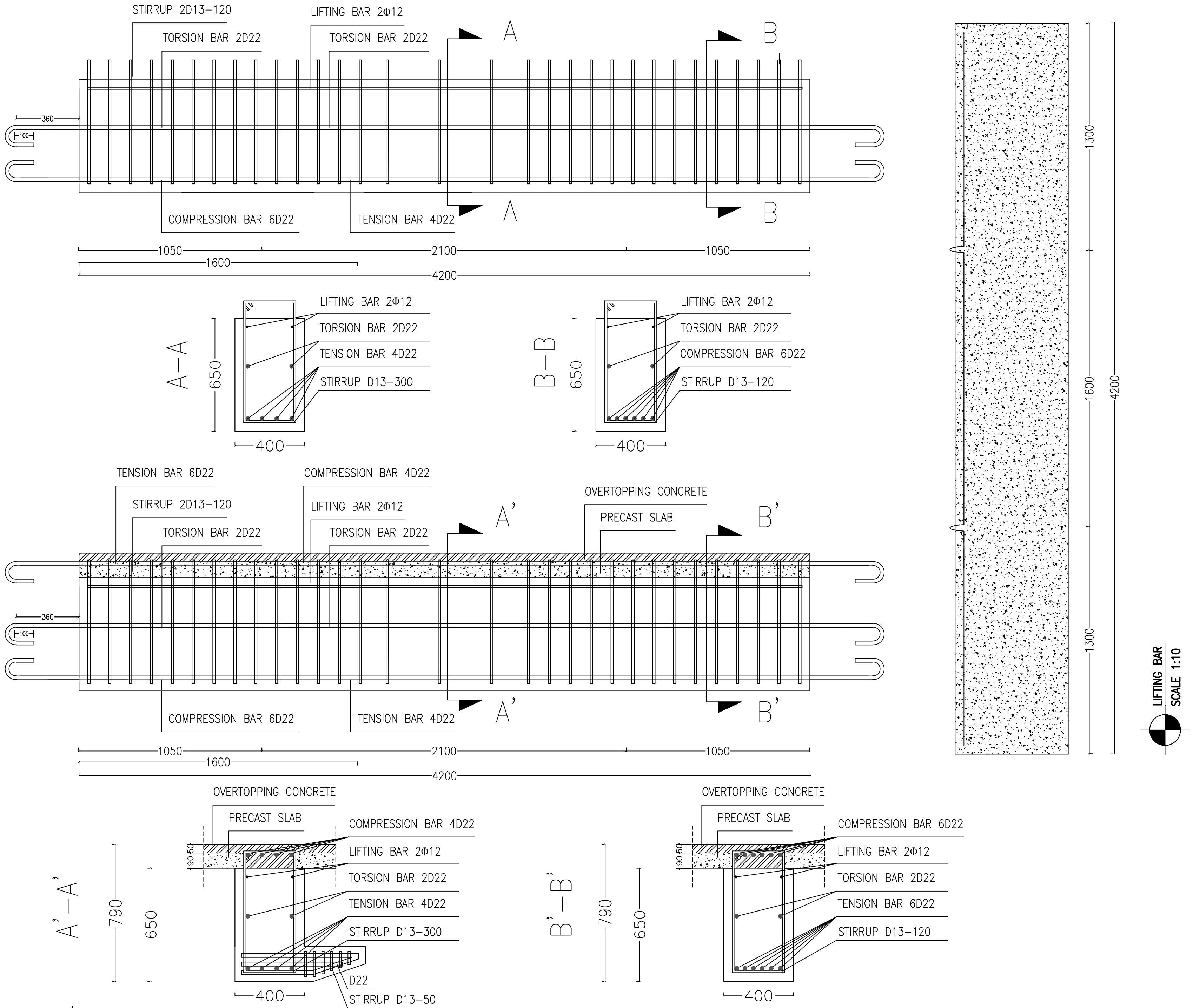


LIFTING BAR

SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
52	61

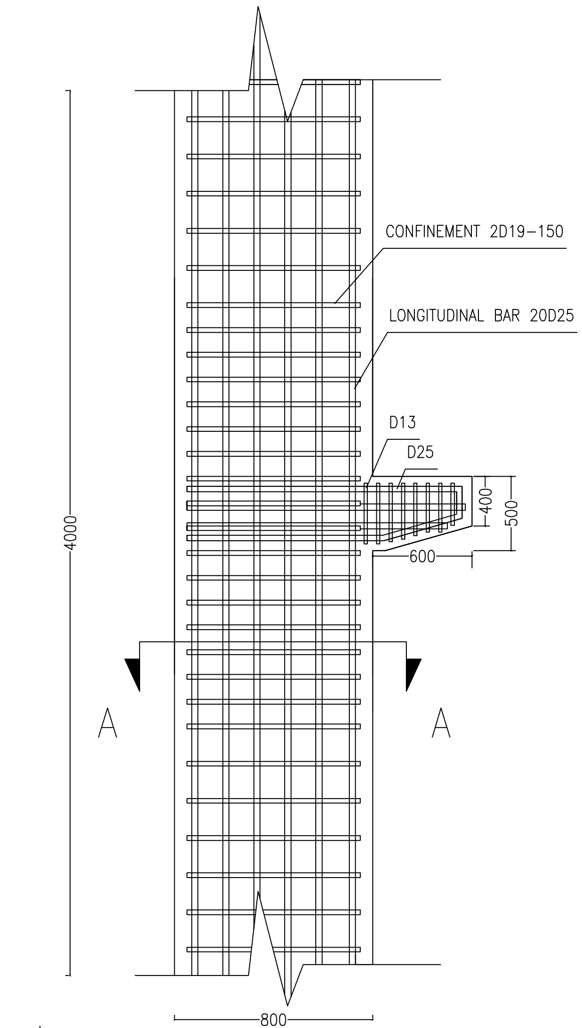


PRECAST EXTERIOR LONGITUDINAL PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT FOR BIA3 (ROOF)

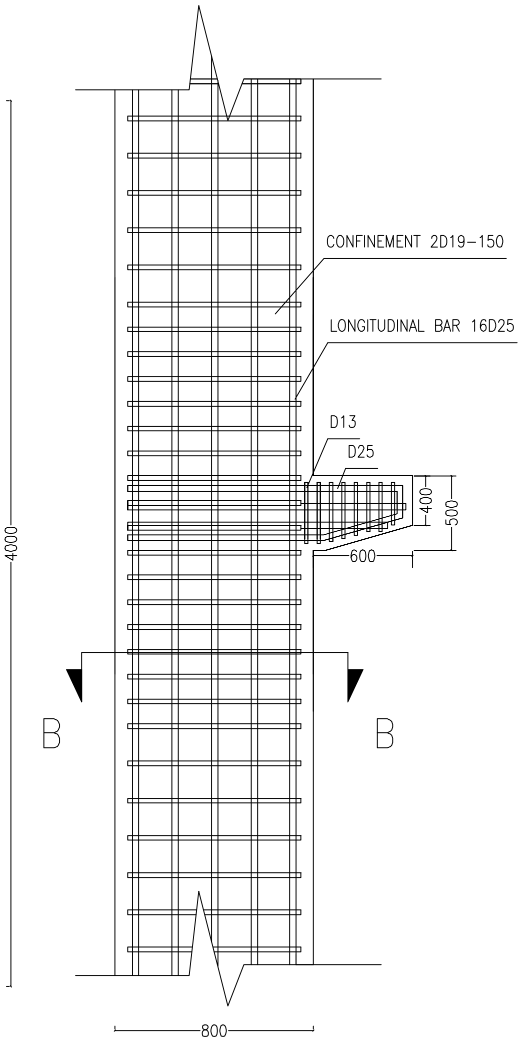
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

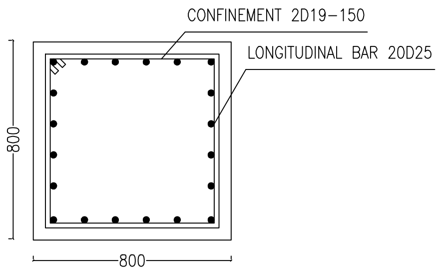
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
53	61



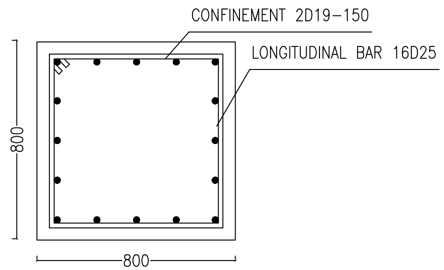
DETAILING OF REINFORCEMENT OF CAST IN PLACE PILLAR TYPE K1
SCALE 1:10



DETAILING OF REINFORCEMENT OF CAST IN PLACE PILLAR TYPE K2
SCALE 1:10



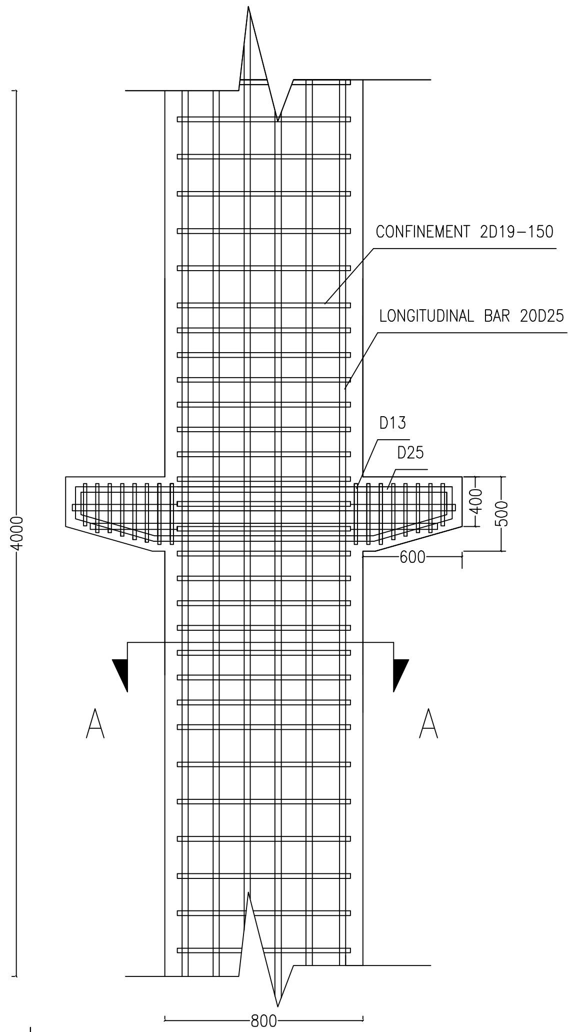
SECTION CUT A–A
SCALE 1:10



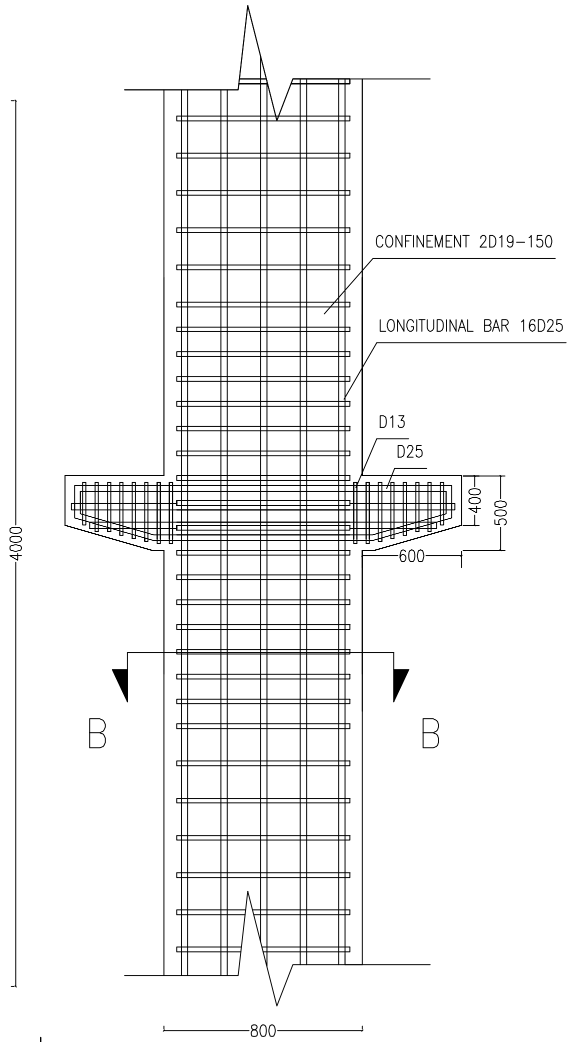
SECTION CUT B–B
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

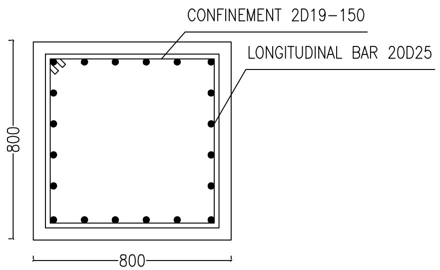
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD.	
Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
54	61



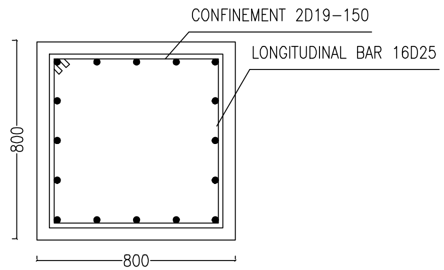
DETAILING OF REINFORCEMENT OF CAST IN PLACE PILLAR TYPE K1
SCALE 1:10



DETAILING OF REINFORCEMENT OF CAST IN PLACE PILLAR TYPE K2
SCALE 1:10



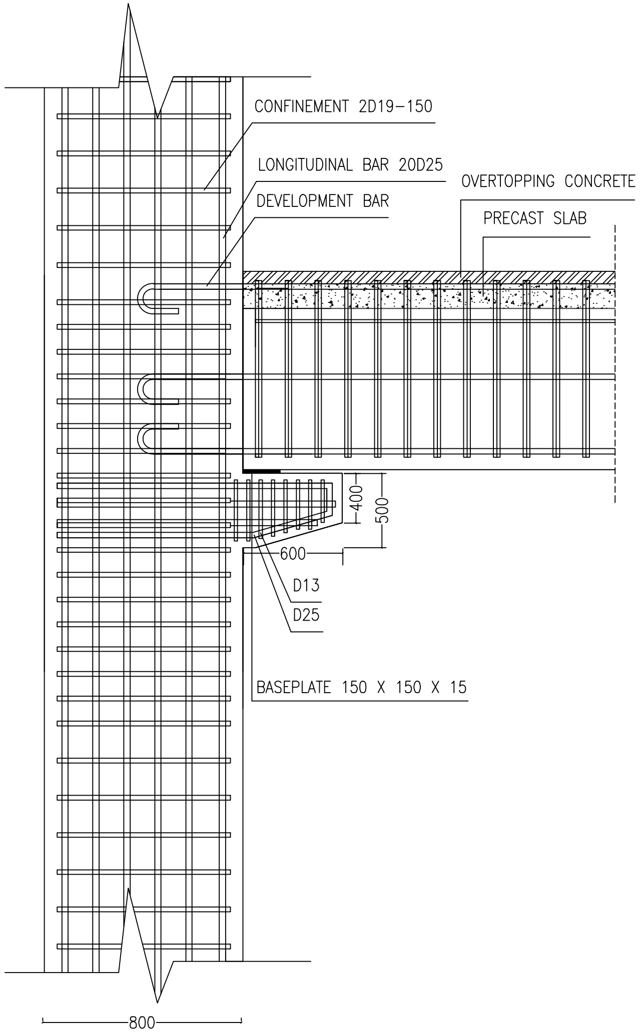
SECTION CUT A–A
SCALE 1:10



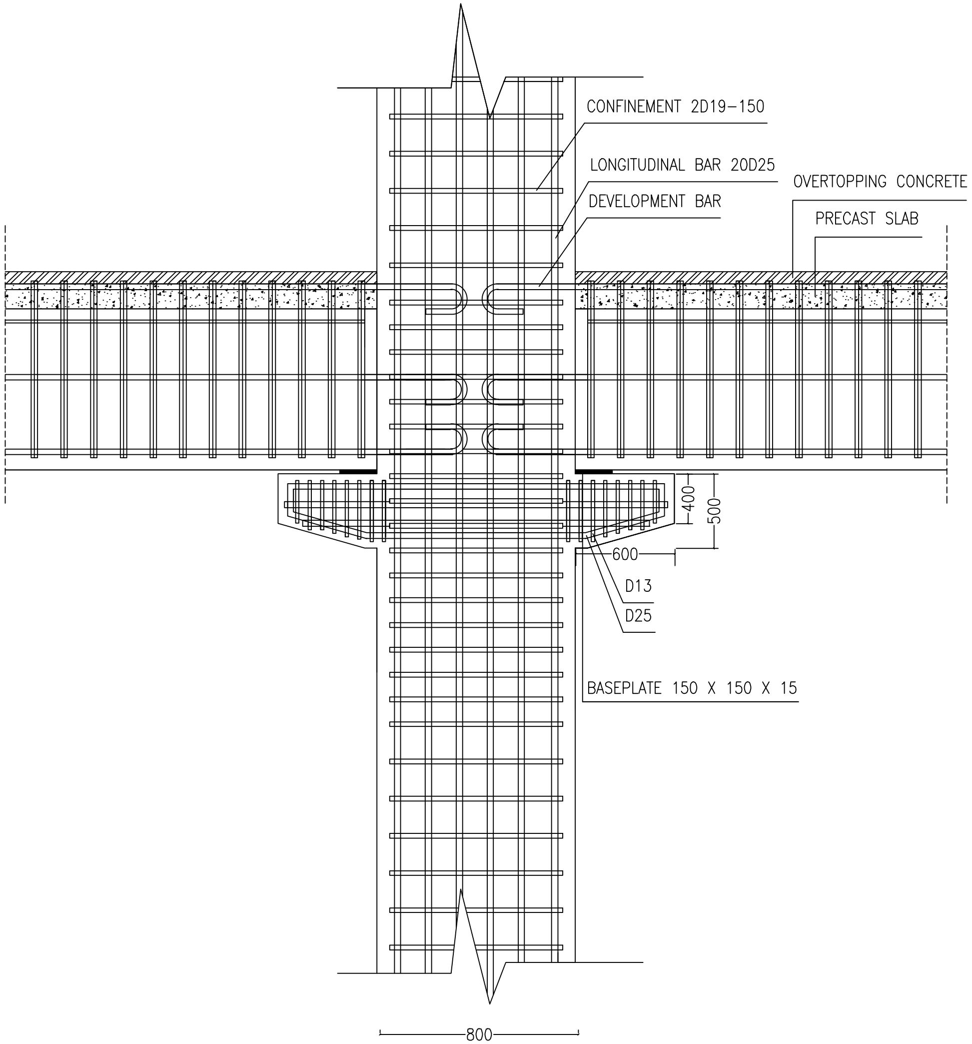
SECTION CUT B–B
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
55	61



DETAILING OF CONNECTION OF EXTERIOR PILLAR AND PRIMARY BEAM
SCALE 1:10



DETAILING OF CONNECTION OF INTERIOR PILLAR AND PRIMARY BEAM
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
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STRUCTURAL	
SCALE	
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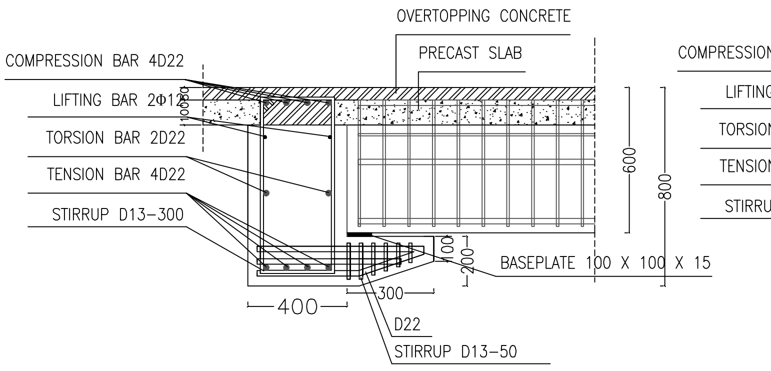
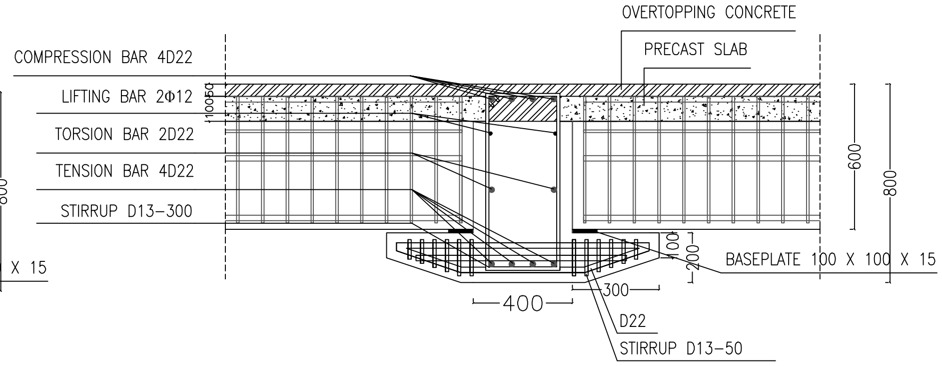
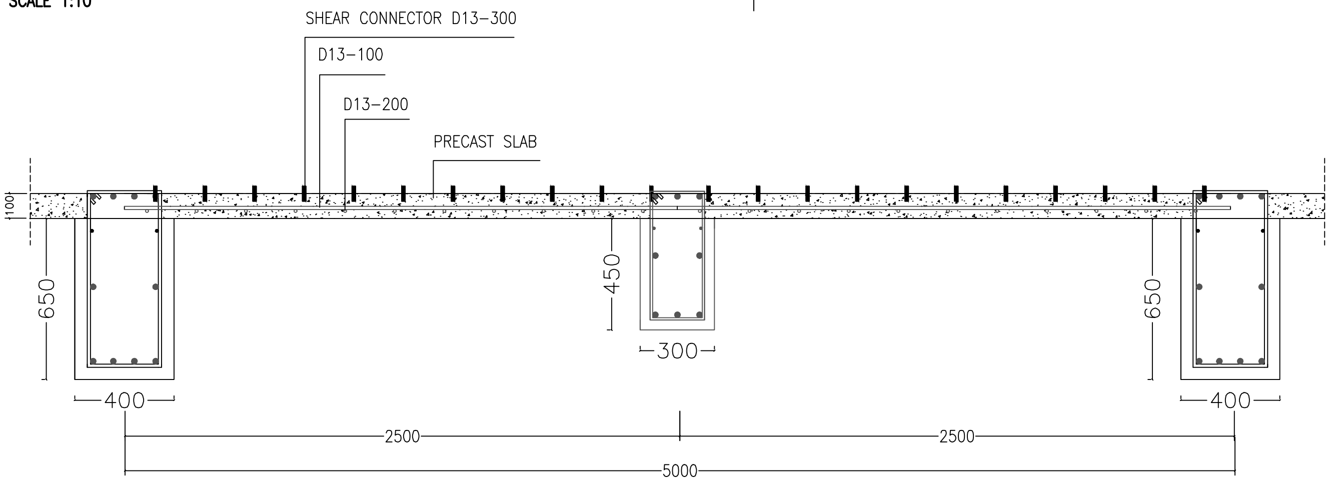
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
56	61



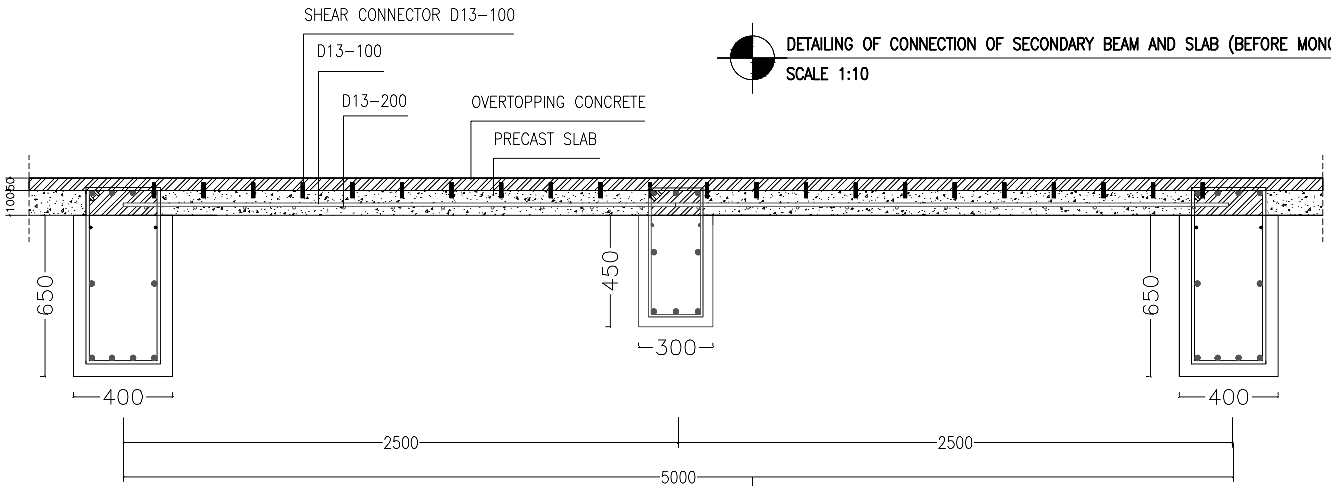
DETAILING OF CONNECTION OF EXTERIOR PRIMARY BEAM AND SECONDARY BEAM
SCALE 1:10



DETAILING OF CONNECTION OF INTERIOR PRIMARY BEAM AND SECONDARY BEAM
SCALE 1:10



DETAILING OF CONNECTION OF SECONDARY BEAM AND SLAB (BEFORE MONOLITH)
SCALE 1:10



DETAILING OF CONNECTION OF SECONDARY BEAM AND SLAB (AFTER MONOLITH)
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
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STRUCTURAL	
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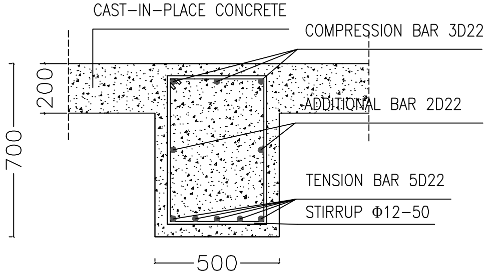
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
57	61



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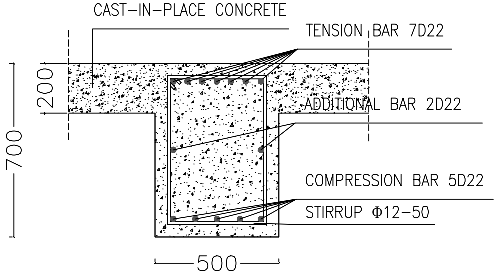
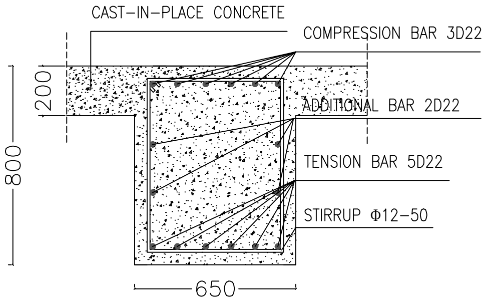
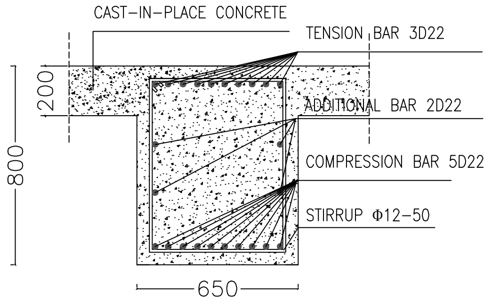


SUPPORT

BASEMENT SECONDARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT
SCALE 1:10



FIELD

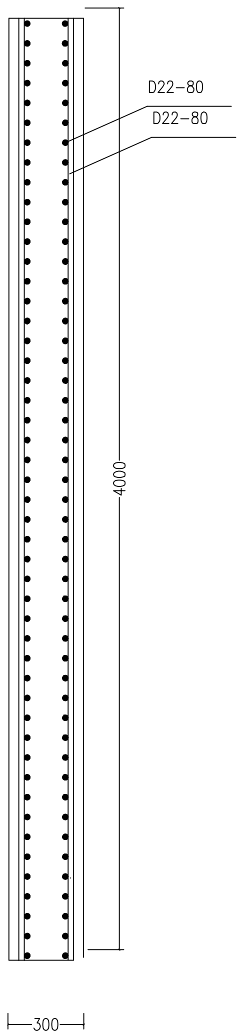


SUPPORT

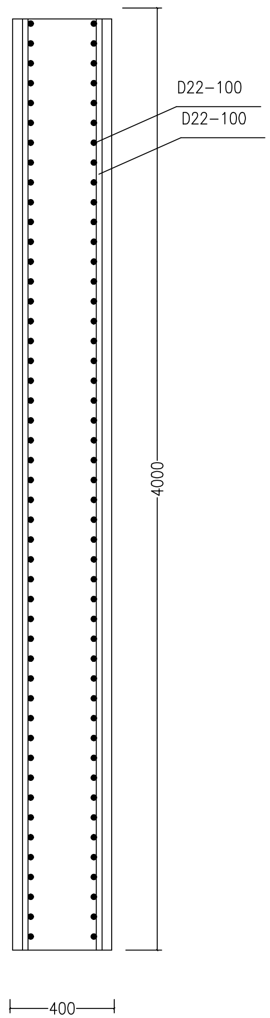
BASEMENT PRIMARY BEAM'S REINFORCEMENT DETAIL BLUEPRINT
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
58	61



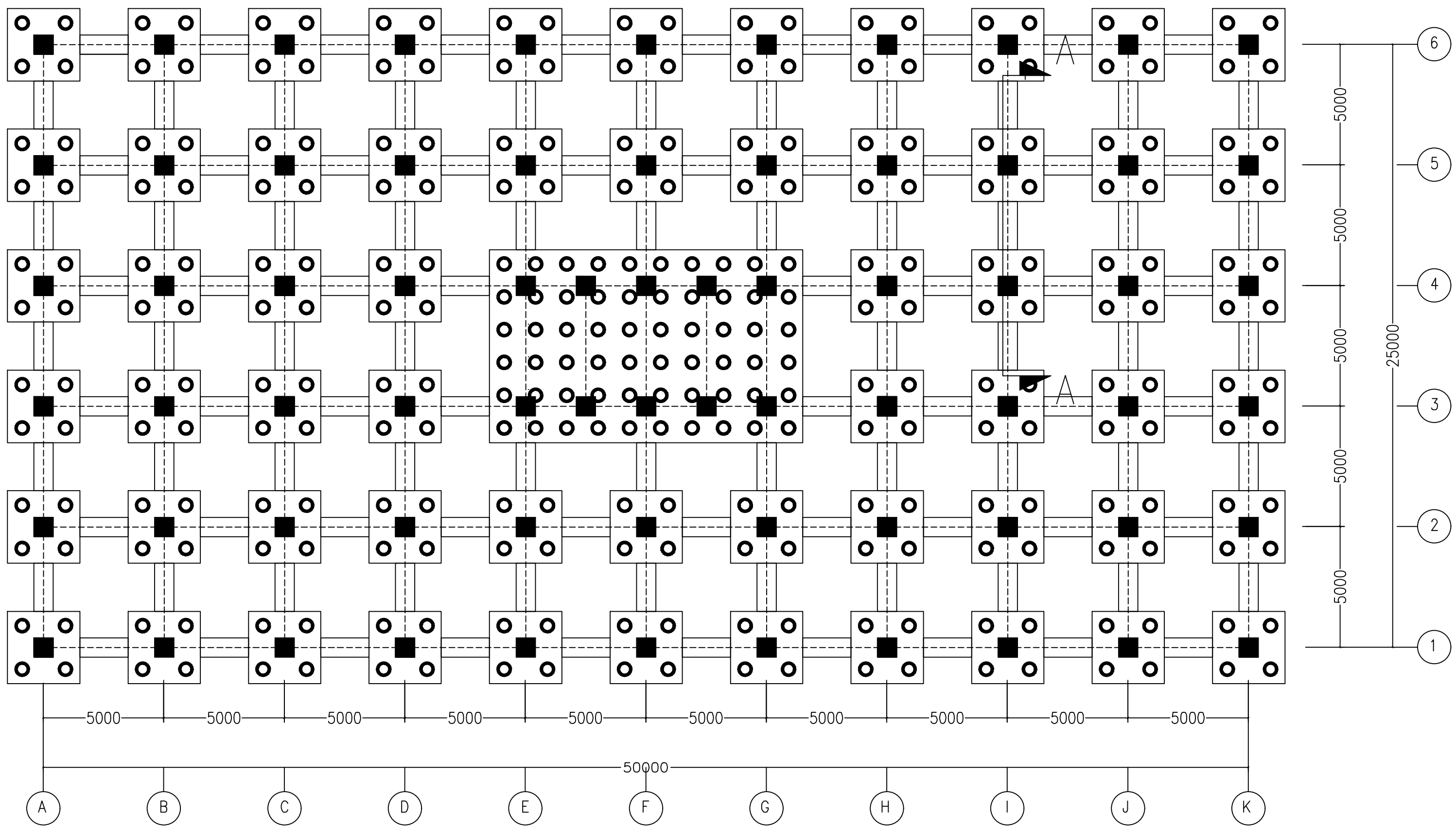
DETAILING OF REINFORCEMENT OF BASEMENT RETAINING WALL
SCALE 1:10



DETAILING OF REINFORCEMENT OF SHEAR WALL
SCALE 1:10

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

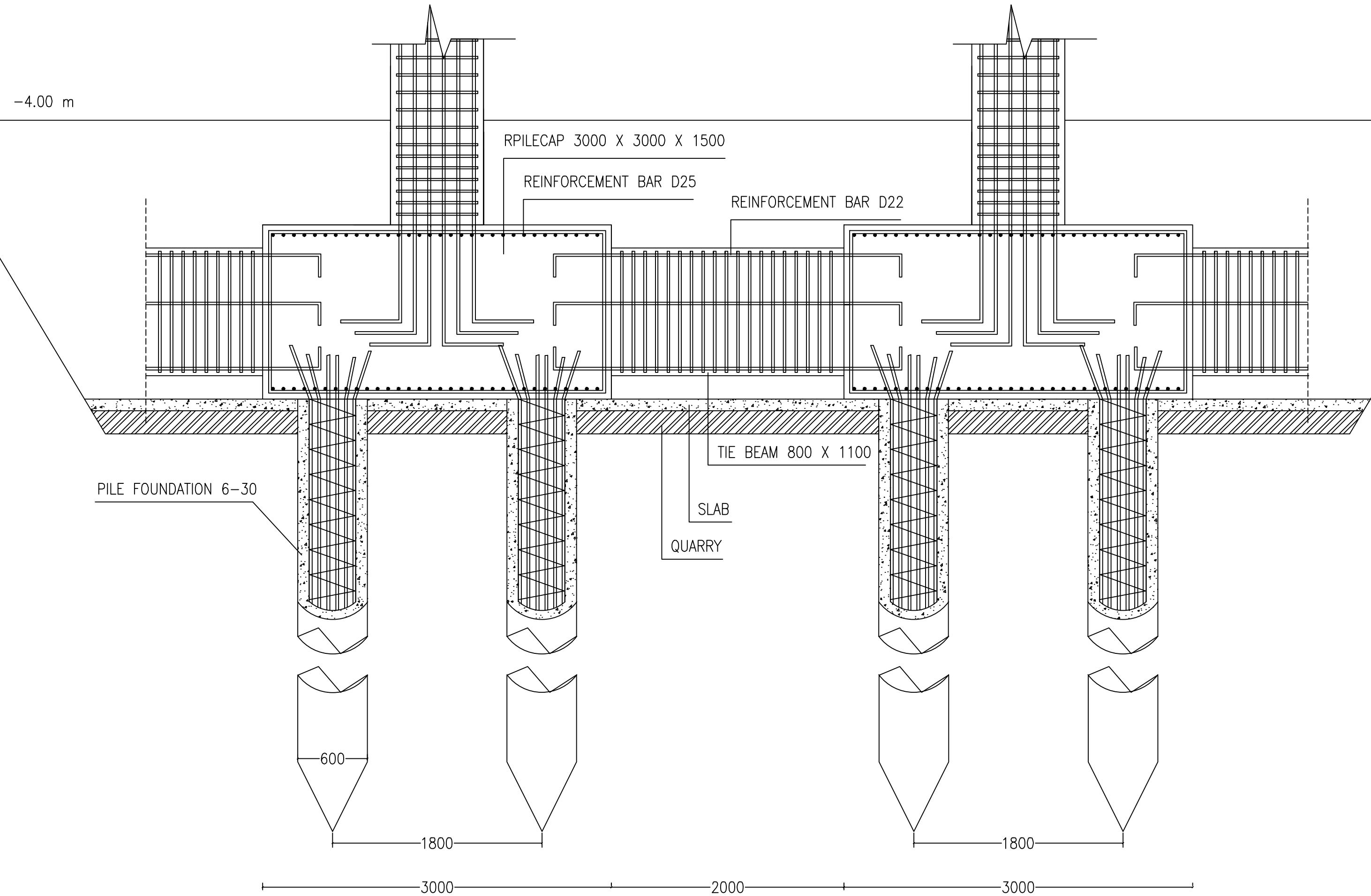
PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
59	61



PILECAP BLUEPRINT
SCALE 1:100

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
60	61



SECTION CUT A-A
SCALE 1:20

DEPARTEMENT OF CIVIL ENGINEERING
FACULTY OF CIVIL ENGINEERING AND PLANNING
TENTH OF NOVEMBER INSTITUTE OF TECHNOLOGY SURABAYA
2016

PROJECT's NAME	
FINAL PROJECT	
TECHNICAL DRAWING	
STRUCTURAL	
SCALE	
1:100	
ADVISOR LECTURER	
DATA IRANATA, ST, MT, PhD. Dr. Ir. DJOKO IRAWAN, MS.	
STUDENT's NAME	
SOCA FAHREZA I.	
STUDENT's ID NUMBER	
3112100025	
NUMBER	**TOTAL**
61	61

LABORATORIUM MEKANIKA TANAH & BATUAN

JURUSAN TEKNIK SIPIL

FAKULTAS TEKNIK SIPIL DAN PERENCANAAN - ITS

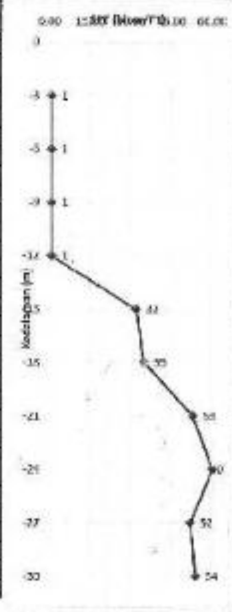
Kampus ITS Keputih Sukolilo Surabaya 60111, Telp. 031 5994251-55 Psw 1140 Telp/Fax 031 592 8601, e-mail : tanah.its@gmail.com

LEGEND : PASIR, LEMPUNG, LANAU, KERIKIL, BATU BARA, UNDISTURBED SAMPLE, MAT

KLIENT	: PT. KOPPELLAND		TANGGAL	:	22 - 23 Agustus 2016
PROYEK	: Perencanaan Pembangunan Apertemen		MASTER BOR	:	Ropii Cs
LOKASI	: Jl. Kenjeran No.504, Hotel Puspa Asri,, Surabaya		ELEVASI	:	±0.00 m - MT
TITIK BOR No.	: BH - 5 (Lima)		MAT	:	- 1,0 m

KEDALAMAN (meter)	DESKRIPSI TANAH	SPT/15 cm: 15	15	15	Jumlah (Blow/ft)
0 – 1	PASIR KERIKIL BERLANAU (COKLAT)				
1 – 5	LEMPUNG BERLANAU (ABU-ABU)	1/45			1
5 – 7	PASIR BERLANAU (ABU-ABU)	1/45			1
7 – 15	LEMPUNG BERLANAU (ABU-ABU)	1/45			1
		1/45			1
15 – 19	LEMPUNG BERPASIR BERLANAU (COKLAT)	6	14	18	32
		9	16	19	35
19 – 21	LEMPUNG BERPASIR BERKERIKIL (COKLAT)				
21 – 26	LEMPUNG PASIR BERKERIKIL (COKLAT)	21	25	28	53
		>60			60
26 – 30	PASIR BERLEMPUNG BERKERIKIL (COKLAT)	19	22	30	52
30	LEMPUNG BERPASIR (ABU-ABU)	20	24	30	54

SPT (blow/feet)

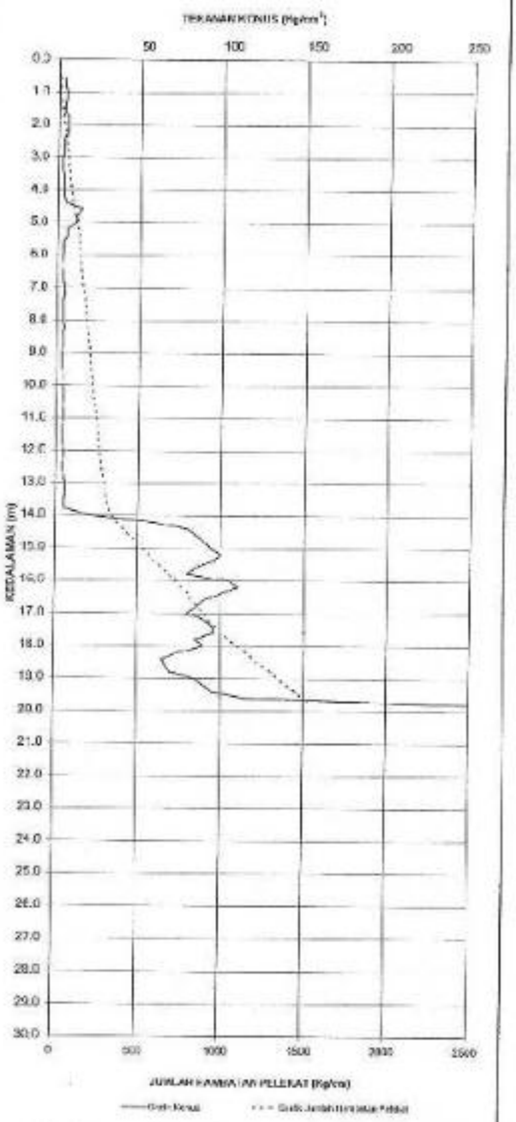


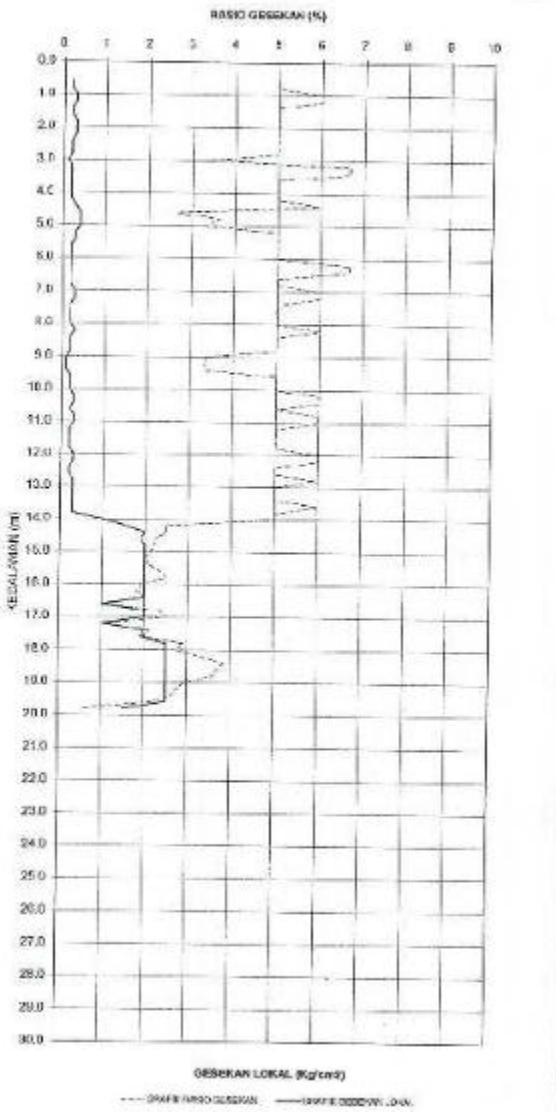
LABORATORIUM MEKANIKA TANAH & BATUAN

JURUSAN TEKNIK SIPIL
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN - ITS
Kampus ITS, Keputih Sukolilo Surabaya 60111,
Telp. 031 5994251 – 55, Psw. 1140,

GRAFIK SONDIR

TITIK	:	S-3	MASTER SONDIR	:	Agus Cs
LOKASI	:	Jl. Kenjeran No.504, Hotel Puspa Asri,, Surabaya	TANGGAL	:	19-Aug-2016
ELEVASI	:	± 0,00 (dari muka tanah setempat)			



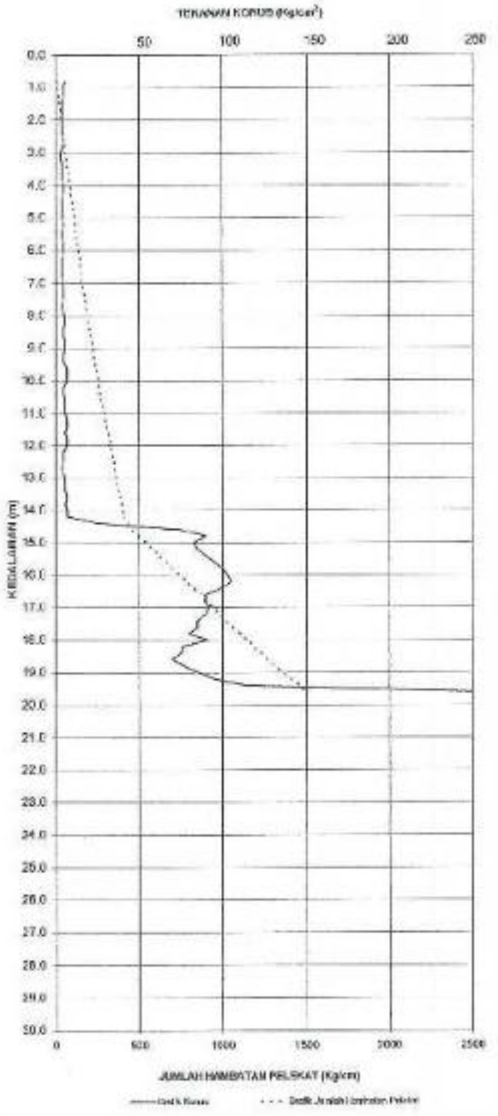


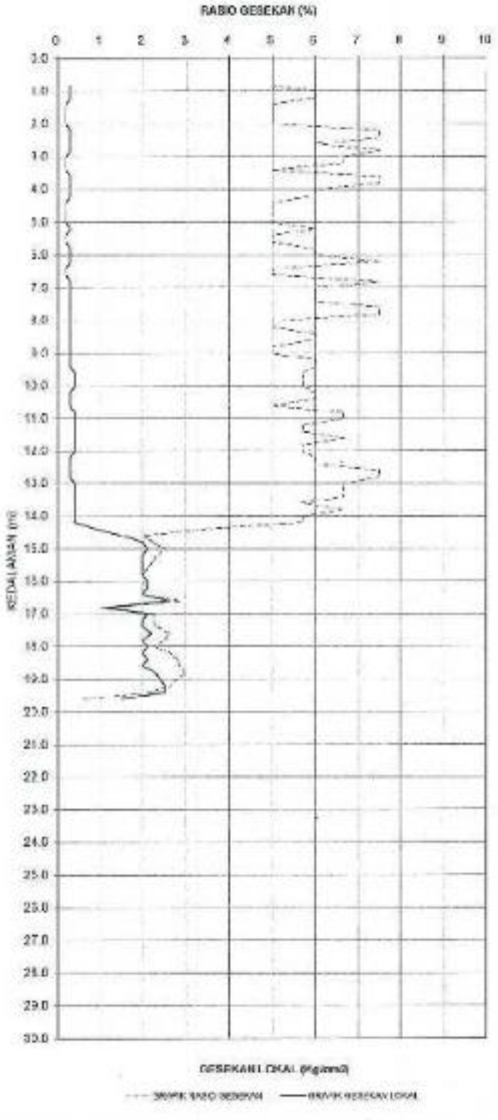
LABORATORIUM MEKANIKA TANAH & BATUAN

JURUSAN TEKNIK SIPIL
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN - ITS
Kampus ITS Keputih Sukolilo Surabaya 60111,
Telp. 031 5994251 – 55, Psw. 1140,

GRAFIK SONDIR

TITIK	:	S-4	MASTER SONDIR	:	Agus Cs
LOKASI	:	Jl. Kenjeran No.504, Hotel Puspa Asri., Surabaya	TANGGAL	:	19-Aug-2016
ELEVASI	:	± 0,00 (dari muka tanah setempat)			



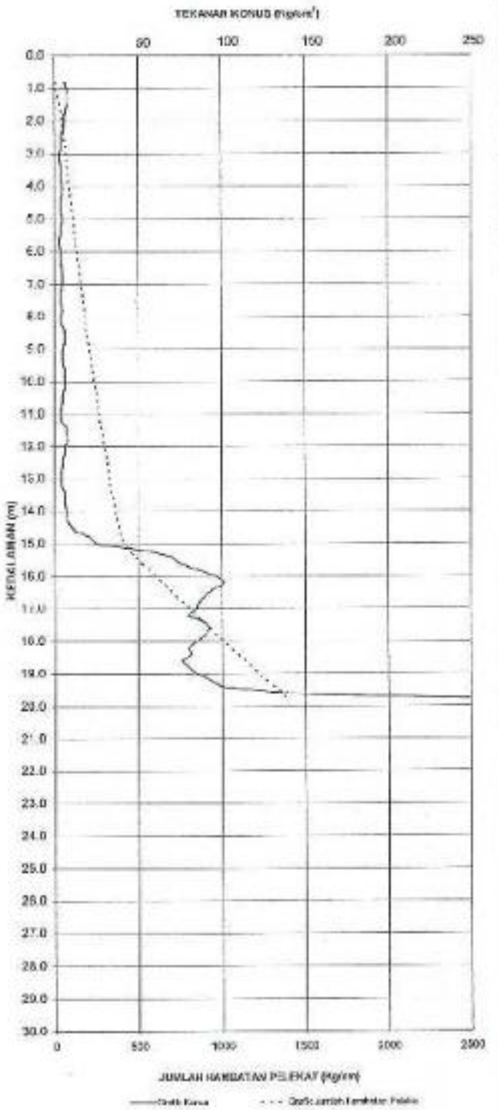


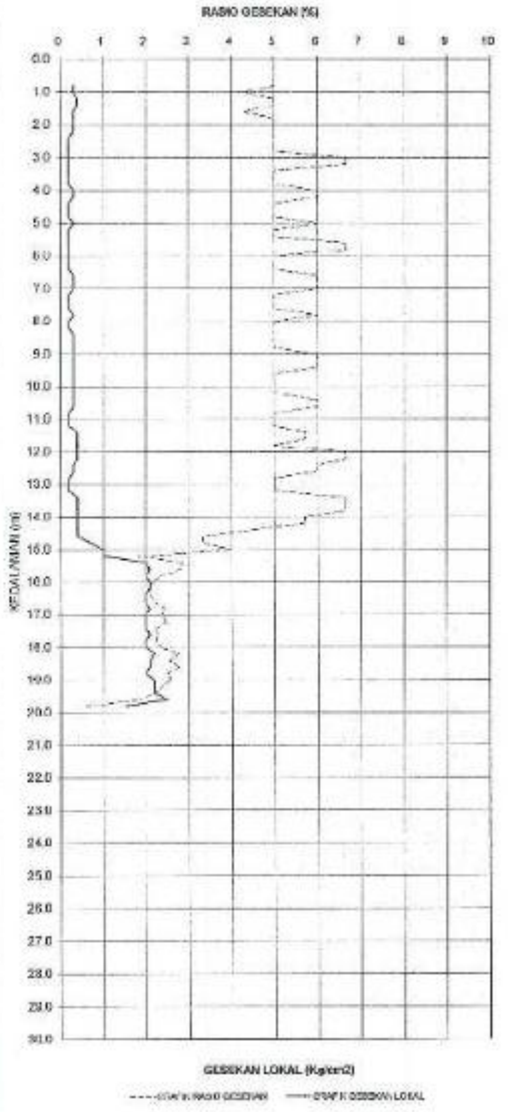
LABORATORIUM MEKANIKA TANAH & BATUAN

JURUSAN TEKNIK SIPIL
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN - ITS
Kampus ITS, Keputih Sukolilo Surabaya 60111,
Telp. 031 5994251 – 55, Psw. 1140,

GRAFIK SONDIR

TITIK	:	S-5	MASTER SONDIR	:	Agus Cs
LOKASI	:	Jl. Kenjeran No.504, Hotel Puspa Asri,, Surabaya	TANGGAL	:	20-Aug-2016
ELEVASI	:	± 0,00 (dari muka tanah setempat)			





LABORATORIUM MEKANIKA TANAH & BATUAN
JURUSAN TEKNIK SIPIL
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN - ITS
Kampus ITS, Keputih Sukolilo Surabaya
Telp. 031 5994251 - 55 Psw. 1140,
Telp/Fax: 031 5928601, e-mail: tanah.its@gmail.com

REKAP HASIL TEST LABORATORIUM

KLIEN : PT. KOPEL LAHAN ANDALAN
PROYEK : PEMBANGUNAN APARTEMEN
LOKASI : JL. KENJERAN 504 SURABAYA

TITIK BOR : BH - 5
MASTER BOR : Ropil Cs

DEPTH (Meter)	VOLUMETRIC + GRAVIMETRIC								CONSOLIDATION		
	Gs	e	Sr	Wc	n	γt	γd	γsat	Pp	Cc	Cv
-7.50	2.637	1.567	100.00	59.42	61.04	1.638	1.027	1.636	1.07	0.841	5.20E-03
-15.00	2.597	1.197	100.00	46.09	54.48	1.727	1.182	1.727	*	*	*
-21.50	2.615	0.879	100.00	33.61	46.78	1.860	1.392	1.859	*	*	*
-30.00	2.729	0.772	100.00	28.29	43.57	1.976	1.540	1.976	*	*	*

DEPTH (Meter)	SIEVE ANALYSIS			ATTERBERG LIMITS			DIRECT TEST		Unconfined test		TRIAXIAL CD		VANE TEST		k (cm/sec)
	G	S	S+Cl	LL	PL	IP	c	φ	C_u	$φ_u$	C_d	$φ_d$	Su	Cu	
-7.50	0.00	11.09	88.91	63.45	28.65	34.80	*	*	*	0	*	*	*	0.35	*
-15.00	0.00	32.77	67.23	53.28	30.75	22.53	*	*	1.53	0	*	*	*	*	*
-21.50	12.35	30.87	56.78	42.87	29.68	13.19	*	*	2.31	0	*	*	*	*	*
-30.00	0.00	29.45	70.55	59.37	31.25	28.12	*	*	2.55	0	*	*	*	*	*

REMARK

G = Gravel (%)
S = Sand (%)
S + Cl = Silt + Clay (%)

e = Void ratio
Gs = Specific Gravity
n = Porosity (%)
Sr = Degree of saturation (%)
W_c = Water content (%)
$γ_t$ = Moisture density (gr/cc)
$γ_{sat}$ = Saturated density (gr/cc)
$γ_d$ = Dry density (gr/cc)

LL = Liquid Limit (%)
PL = Plastic Limit (%)
IP = Plastic Index (%)

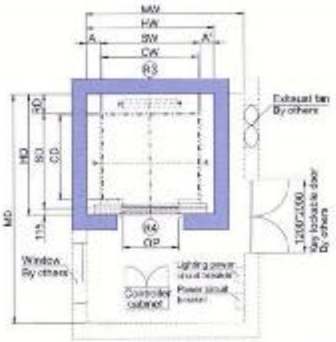
Cc = Compression index
Cv = Coefficient of Consolidation (cm^2/det)
Pp = Preconsolidation Pressure (kg/cm^2)
* = Not test

k = Coeficient of permeability (cm/sec)

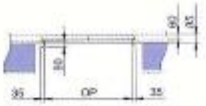
C = Cohesion of direct shear (kg/cm^2)
C_d = Drained cohesion triaxial test(kg/cm^2)
C_u = Undrained cohesion (kg/cm^2)
Su = Strength of vane test (kg/cm^2)
φ = Angle of internal friction direct shear test (degree)
φd = Angle of internal friction drained triaxial test (degree)
φu = Angle of internal friction undrained triaxial test (degree)

LAYOUT - CENTER OPENING (capacity 450-1000kg)

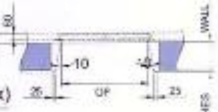
HOISTWAY & MACHINE ROOM PLAN (Simplex)



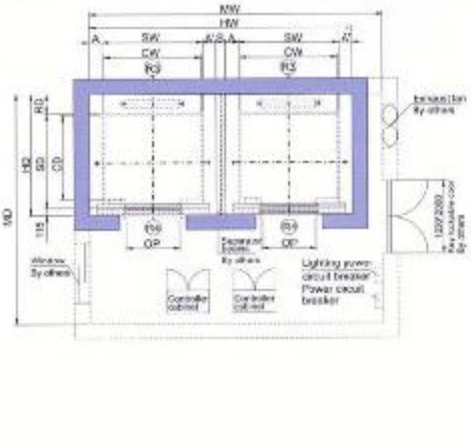
NARROW JAMB DETAIL



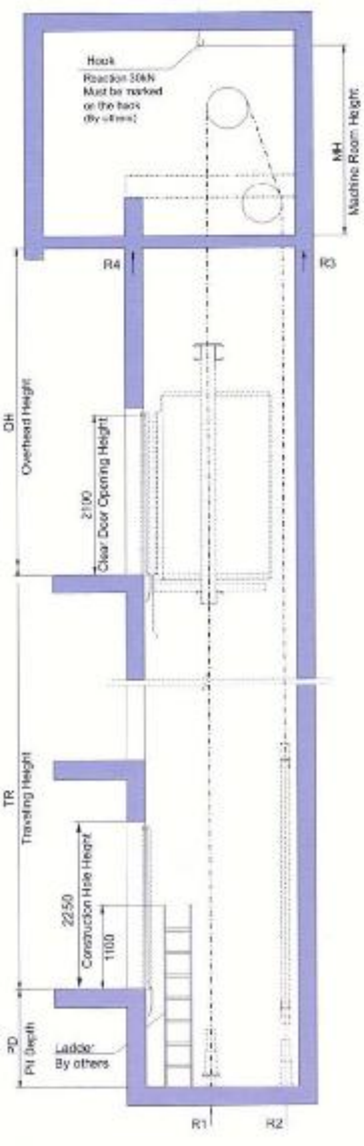
WIDE JAMB DETAIL



HOISTWAY & MACHINE ROOM PLAN (Duplex)



HOISTWAY ELEVATION



OVERHEAD, PIT DEPTH & MACHINE ROOM HEIGHT

Speed (m/min)	Overhead OH (mm)	Pit Depth PD (mm)	Machine Room Height MH (mm)
60	4400	1300	2500
90	4500	1400	2500
105	4600	1500	2500

DATA SHEET-CENTER OPENING (capacity 450-1000kg)

PLANNING GUIDE FOR DIMENSIONS

Unit: mm

Speed (m/min)	Capacity		Entrance	Car Size		Dimensions			Hoistway Size				Machine Room Size				Machine Room		Pit Reaction	
	Person	Load (kg)	Center Opening (mm)	Inside	Outside	in Hoistway			Simplex		Duplex		Simplex		Duplex		Reaction Load(kg)		Load(kg)	
				CW*CD	KW*KD	A&A	RD	S	HW	HD	HW	HD	MW	MD	MW	MD	R3	R4	R1	R2
60	6	450	800	1400×850	1450×1015	200	320	150	1850	1450	3850	1450	2350	3450	4350	3450	3600	2330	5300	4400
	7	550	800	1400×1000	1450×1165	200	320	150	1850	1600	3850	1600	2350	3600	4350	3600	4000	2600	6200	5100
	8	630	800	1400×1100	1450×1265	200	320	150	1850	1700	3850	1700	2350	3700	4350	3700	4200	2800	6700	5500
	9	680	800	1400×1200	1450×1365	200	320	150	1850	1800	3850	1800	2350	3800	4350	3800	4900	3100	7200	5800
	10	750	800	1400×1300	1450×1465	200	320	150	1850	1900	3850	1900	2350	3900	4350	3900	5300	3300	7800	6300
	10	800	800	1400×1350	1450×1515	200	320	150	1850	1950	3850	1950	2350	3950	4350	3950	5600	3500	8300	6700
	12	900	900	1600×1300	1650×1465	225	320	150	2100	1900	4350	1900	2600	3900	4850	3900	6100	3800	9200	7400
	13	1000	900	1600×1400	1650×1585	225	320	150	2100	2000	4350	2000	2600	4000	4850	4000	6700	4100	10100	8100

Unit: mm

Speed (m/min)	Capacity		Entrance	Car Size		Dimensions			Hoistway Size				Machine Room Size				Machine Room		Pit Reaction	
	Person	Load (kg)	Center Opening (mm)	Inside	Outside	in Hoistway			Simplex		Duplex		Simplex		Duplex		Reaction Load(kg)		Load(kg)	
				CW*CD	KW*KD	A&A	RD	S	HW	HD	HW	HD	MW	MD	MW	MD	R3	R4	R1	R2
90 105	7	550	800	1400×1000	1450×1135	200	320	150	1850	1600	3850	1600	2350	3500	4350	3600	4400	2800	6200	5100
	8	630	800	1400×1100	1450×1265	200	320	150	1850	1700	3850	1700	2350	3700	4350	3700	4700	3000	6700	5600
	9	680	800	1400×1200	1450×1365	200	320	150	1850	1800	3850	1800	2350	3500	4350	3800	4900	3100	7200	5800
	10	750	800	1400×1300	1450×1465	200	320	150	1850	1900	3850	1900	2350	3900	4350	3900	5300	3300	7800	6300
	10	800	800	1400×1350	1450×1515	200	320	150	1850	1950	3850	1950	2350	3960	4350	3950	5600	3500	8300	6700
	12	900	900	1600×1300	1650×1465	225	320	150	2100	1900	4350	1900	2600	3900	4850	3900	6100	3800	9200	7400
	13	1000	900	1600×1400	1650×1585	225	320	150	2100	2000	4350	2000	2600	4000	4850	4000	6700	4100	10100	8100

ELECTRICAL DESIGN GUIDE

380V

Speed (m/min)	Capacity		Motor Capa. (kW)	MCCB Capacity of Building (A)		Power Supply Capacity(KVA)		Lead-in Wire Size (mm²)		Earth Wire Size(mm²)	Heat Output (KJ/H)
	Person	Load (kg)		Simplex	Duplex	Simplex	Duplex	Simplex	Duplex		
60	6	450	5.5	20	35	5.9	13.8	4	6	4	2970
	7	550	6.4	20	40	7.9	15.8	6	10	6	3456
	8	630	7.5	25	45	8.7	17.4	6	10	6	4050
	9	680	7.5	25	45	8.7	17.4	6	10	6	4050
	10	750	9	30	55	10.6	21.2	6	16	6	4860
	11	800	9	30	55	10.6	21.2	6	16	6	4860
	12	900	11	35	63	13.2	26.4	6	10	6	5940
	13	1000	11	35	63	13.2	26.4	6	16	6	5940
90	6	450	7.5	25	45	8.7	17.4	6	16	6	4050
	7	550	9	30	63	10.6	21.2	6	16	6	4860
	8	630	11	35	63	13.2	26.4	6	16	6	5940
	9	680	11	35	63	13.2	26.4	6	16	6	5940
	10	750	15	45	98	17.4	34.8	10	16	10	8100
	11	800	15	45	98	17.4	34.8	10	16	10	8100
	12	900	15	45	98	17.4	34.8	10	16	10	8100
	13	1000	18.5	80	110	21.7	43.4	16	28	16	9990
105	6	450	11	35	63	13.2	26.4	6	16	6	5940
	7	550	11	35	63	13.2	26.4	6	16	6	5940
	8	630	11	35	63	13.2	26.4	6	16	6	5940
	9	680	15	45	98	17.4	34.8	10	16	10	8100
	10	750	15	45	98	17.4	34.8	10	16	10	8100
	11	800	15	45	98	17.4	34.8	10	16	10	8100
	12	900	18.5	80	110	21.7	43.4	16	28	16	9990
	13	1000	18.5	80	110	21.7	43.4	16	28	16	9990

ICP PILES

HIGH PERFORMANCE

PRETENSIONED SPUN HIGH STRENGTH CONCRETE PILES

ICP PILES

INTRODUCTION

INDUSTRIAL CONCRETE PRODUCTS SDN BHD (ICP) is the first commercial manufacturer of PRETENSIONED SPUN CONCRETE PILES (ICP PC PILES) in Malaysia. Presently, ICP is the largest manufacturer in South East Asia.

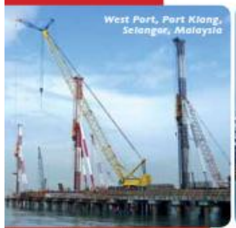
The Company was incorporated in Malaysia on 6 April 1977 and commenced business in September 1977.

In September 1993, ICP started manufacturing HIGH PERFORMANCE Pretensioned Spun High Strength Concrete Piles (ICP PHC PILES) which offers an economical foundation system with consistent and superior quality compared to the ordinary concrete piles. With the vast experience in the manufacturing of pretensioned spun concrete piles and utilising the latest concrete technology, ICP is the market leader in concrete piles. ICP has recently introduced higher grade piles of Grade 90 into the market in 2011.

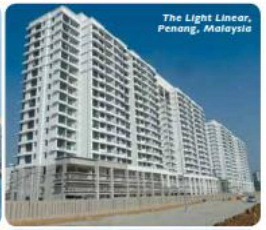
ICP Piles are circular in cross-section and are manufactured in sizes ranging from diameter 250mm to 1,200mm with standard lengths varying from 6m to 46m in single pieces. For the large diameter piles, ICP has pile joining facilities which are able to pre-join the piles up to 66m. This shall provide good quality welding and most importantly expedite the piling progress at site.

ICP Piles have been used extensively as foundation piles for power stations, highrise buildings, civil engineering works, bridges, marine structures, harbours, schools and government projects, etc.

ICP Piles are exported to Brunei, Singapore, Bangladesh, Sri Lanka, the Middle East, Indonesia, Pakistan, Philippines, Vietnam, Myanmar, USA, Canada, New Zealand, Maldives and Samoa. ICP Piles are also supplied to many projects in Southern China from our plant in Jiangmen, China.


West Port, Port Klang, Selangor, Malaysia


Tg Bin Power Plant, Johor, Malaysia


The Light Linear, Penang, Malaysia


Megayard Project, Tuas, Singapore


OMA 1, Commercial cum Residential Complex, Vancouver, Canada

GENERAL SPECIFICATION

STANDARDS

ICP PHC Piles comply with MS 1314:Part 4:2004 and also generally comply with JIS A 5337:1987. ICP PHC Piles are modified to suit BS 8004:1986 – Foundations and BS 8110:1997 – Structural Use of Concrete. Concrete complies with SS EN 206-1:2009 - specification of concrete.

MATERIALS

Aggregates – Coarse aggregates shall be 20mm granite. Fine aggregates shall be clean river sand or washed mining sand.
Cement – Portland cement comply with MS 522:2007.
Prestressing Steel – High frequency induction heat treated bars manufactured to JIS G 3137:1994 or equivalent.
Spiral Wire – Hard drawn wire.

CONCRETE STRENGTH

Minimum concrete cube strength:

at transfer of prestress	30 N/mm²
at 28 days - Grade 80 pile	80 N/mm²
- Grade 90 pile	90 N/mm²

JOINT

The joint is designed to have the same performance as the main body particulary in respect of bending strength. All ICP PHC Piles have steel extension plates for splicing.

LIFTING POINTS

For piles up to 12m length, piles shall be lifted by using steel hooks at both ends. For piles exceeding 12m, piles shall be lifted by wrapping wire ropes around the piles at the marked lifting points.

PILE SHOE

All ICP PHC Piles will be supplied either open ended, with a flat shoe or with an X-pointed shoe.

CURING

After casting, the piles are steam cured. When the concrete reaches the specified transfer strength, the piles are demoulded, marked and checked for quality. The piles can normally be transported and driven after three days from the date of casting, or when the cube strength reaches 70 N/mm².

IDENTIFICATION

All ICP PHC Piles have the typical markings as follows:

(logo)	Logo as Trademark
ICP	Company's Initial
MS1314: Part 4	Malaysian Standard
60B800	Pile Size and Class
130917	Date of Cast (yy/mm/dd)
001L3	Serial No & Factory Code
12E	Pile Length and Type

Other markings if used, S for Starter (flat shoe or X-pointed shoe)

STANDARD LENGTHS

ICP PHC Piles are available in lengths of 6m to 46m subject to certain limitations.

DELIVERY

ICP Standard Piles are normally available ex-stock. Custom made piles usually takes two to three weeks from date of confirmed order.

TECHNICAL DATA

Technical data of our standard piles are given in the tables on the next page. Please note that the axial loads represent the structural capacities of the piles. Actual working load will depend on the soil conditions and the pile slenderness ratio. Appropriate reduction of axial loads should be applied for
a) marine structures;
b) piles subjected to bending;
c) high upstand;
d) piles driven through very poor top stratum;
e) raking piles, etc.

DEFINITIONS

ICP PHC Piles: High Performance Pretensioned Spun High Strength Concrete Piles
MS: Malaysian Standard
JIS: Japanese Industrial Standard
BS: British Standard
SS EN: Singapore Standard

PROPERTIES OF ICP PILES

GRADE 80 PILES

CLASS A (EFFECTIVE PRESTRESS ≥ 4.0 N/mm²)

Nominal Diameter	Nominal Thickness	Length	Nominal Weight	Prestressing Bar 7.1mm	Prestressing Bar 9.0mm	Prestressing Bar 10.7mm	Area of Concrete	Section Modulus	Bending Moment Cracking	Bending Moment Ultimate	Recommended Max Structural Axial Working Load (For a short strut)	Effective Prestress
mm	mm	m	kg/m	no.	no.	no.	mm²	x1000 mm³	kNm	kNm	ton	N/mm²
300	60	6-12	118	6	-	-	45,239	2,373	21.3	34.8	87	4.9
350	60	6-12	142	8	-	-	54,664	3,533	32.9	54.1	104	5.3
400	65	6-12	178	8	-	-	68,408	5,106	42.7	61.8	132	4.3
450	70	6-12	217	10	-	-	83,566	7,113	60.4	86.9	161	4.5
500	80	6-12	274	12	-	-	105,558	9,888	82.3	115.9	204	4.3
600	90	6-12	375	-	12	-	144,199	16,586	148.8	222.5	276	5.0

CLASS B (EFFECTIVE PRESTRESS ≥ 5.0 N/mm²)

Nominal Diameter	Nominal Thickness	Length	Nominal Weight	Prestressing Bar 7.1mm	Prestressing Bar 9.0mm	Prestressing Bar 10.7mm	Area of Concrete	Section Modulus	Bending Moment Cracking	Bending Moment Ultimate	Recommended Max Structural Axial Working Load (For a short strut)	Effective Prestress
mm	mm	m	kg/m	no.	no.	no.	mm²	x1000 mm³	kNm	kNm	ton	N/mm²
250	55	6-12	88	6	-	-	33,694	1,435	14.9	29.0	63	6.4
300	60	6-12	118	7	-	-	45,239	2,383	22.9	40.6	86	5.6
350	70	6-12	160	9	-	-	61,575	3,778	35.6	60.8	117	5.4
400	80	6-15	209	12	-	-	80,425	5,643	53.7	92.7	153	5.5
450	80	6-15	242	-	8	-	92,991	7,624	70.0	111.2	177	5.2
500	90	6-15	301	-	10	-	115,925	10,518	95.9	154.5	221	5.1
600	100	6-15	408	-	14	-	157,080	17,546	163.1	259.6	299	5.3
700	110	6-18	530	-	20	-	203,889	27,131	265.1	432.6	386	5.8
800	120	6-18	667	-	24	-	256,354	39,455	376.8	593.3	487	5.5
900	130	6-18	818	-	28	-	314,473	54,942	510.3	778.7	599	5.3
1000	140	6-18	983	-	-	24	378,248	74,036	688.9	1042.8	720	5.3
1200	150	6-18	1286	-	-	36	494,801	120,188	1198.5	1877.1	934	6.0

CLASS C (EFFECTIVE PRESTRESS ≥ 7.0 N/mm²)

Nominal Diameter	Nominal Thickness	Length	Nominal Weight	Prestressing Bar 7.1mm	Prestressing Bar 9.0mm	Prestressing Bar 10.7mm	Prestressing Bar 12.6mm	Area of Concrete	Section Modulus	Bending Moment Cracking	Bending Moment Ultimate	Recommended Max Structural Axial Working Load (For a short strut)	Effective Prestress
mm	mm	m	kg/m	no.	no.	no.	no.	mm²	x1000 mm³	kNm	kNm	ton	N/mm²
250	55	6-12	88	7	-	-	-	33,694	1,443	16.2	33.8	63	7.2
300	60	6-12	118	10	-	-	-	45,239	2,416	28.0	37.9	84	7.6
350	70	6-15	160	-	8	-	-	61,575	3,826	43.0	86.5	114	7.2
400	80	6-15	209	-	12	-	-	80,425	5,748	69.7	148.3	147	8.1
450	80	6-15	242	-	12	-	-	92,991	7,734	86.9	166.9	173	7.2
500	90	6-18	301	-	15	-	-	115,925	10,670	120.3	231.7	215	7.3
600	100	6-30	408	-	-	14	-	157,080	17,761	196.4	365.0	292	7.0
700	110	6-46	530	-	-	20	-	203,889	27,498	320.6	608.3	376	7.6
800	120	6-46	667	-	-	24	-	256,354	39,966	454.7	834.3	475	7.4
900	130	6-46	818	-	-	28	-	314,473	55,622	617.7	1095.0	584	7.1
1000	140	6-46	983	-	-	36	-	378,248	75,188	864.5	1564.3	699	7.5
1200	150	6-36	1286	-	-	46	-	494,801	121,361	1378.6	2398.6	916	7.3
1000	140	6-46	983	-	-	-	34	378,248	76,247	961.1	2051.9	688	8.6
1200	150	6-36	1286	-	-	-	46	494,801	123,457	1583.1	3331.3	898	8.8

GRADE 90 PILES

CLASS A (EFFECTIVE PRESTRESS ≥ 4.0 N/mm²)

Nominal Diameter	Nominal Thickness	Length	Nominal Weight	Prestressing Bar 7.1mm	Prestressing Bar 9.0mm	Prestressing Bar 10.7mm	Area of Concrete	Section Modulus	Bending Moment Cracking	Bending Moment Ultimate	Recommended Max Structural Axial Working Load (For a short strut)	Effective Prestress
mm	mm	m	kg/m	no.	no.	no.	mm²	x1000 mm³	kNm	kNm	ton	N/mm²
300	60	6-12	118	6	-	-	45,239	2,373	21.9	34.8	98	4.9
350	60	6-12	142	8	-	-	54,664	3,533	33.8	54.1	118	5.3
400	65	6-12	178	8	-	-	68,408	5,106	43.9	61.8	149	4.3
450	70	6-12	217	10	-	-	83,566	7,113	62.2	86.9	182	4.5
500	80	6-12	274	12	-	-	105,558	9,888	84.8	115.9	231	4.3
600	90	6-12	375	-	12	-	144,199	16,586	152.9	222.5	313	5.0

CLASS B (EFFECTIVE PRESTRESS ≥ 5.0 N/mm²)

Nominal Diameter	Nominal Thickness	Length	Nominal Weight	Prestressing Bar 7.1mm	Prestressing Bar 9.0mm	Prestressing Bar 10.7mm	Area of Concrete	Section Modulus	Bending Moment Cracking	Bending Moment Ultimate	Recommended Max Structural Axial Working Load (For a short strut)	Effective Prestress
mm	mm	m	kg/m	no.	no.	no.	mm²	x1000 mm³	kNm	kNm	ton	N/mm²
250	55	6-12	88	6	-	-	33,694	1,435	15.3	29.0	72	6.4
300	60	6-12	118	7	-	-	45,239	2,383	23.5	40.6	97	5.6
350	70	6-12	160	9	-	-	61,575	3,778	36.5	60.8	133	5.4
400	80	6-15	209	12	-	-	80,425	5,643	55.0	92.7	173	5.5
450	80	6-15	242	-	8	-	92,991	7,624	71.9	111.2	201	5.2
500	90	6-15	301	-	10	-	115,925	10,518	98.5	154.5	251	5.1
600	100	6-15	408	-	14	-	157,080	17,546	167.3	259.6	339	5.3

(Subject to change without prior notice)

FORMULA FOR AXIAL LOAD

Based on BS 8004: 1986, the maximum allowable axial stress that may be applied to a pile acting as a short strut should be one quarter of (specified works cube strength at 28 days less the prestress after losses)

$$N = f_{ca} \times A$$
$$= 1/4\,(f_{cu} - f_{pe}) \times A$$

Where, N = maximum allowable axial load

A = cross section area of concrete

f_{ca} = permissible compressive strength of concrete

f_{cu} = specified compressive strength of concrete

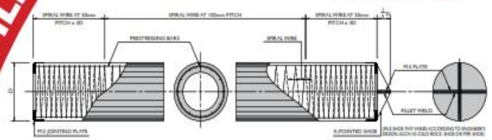
f_{pe} = effective prestress in concrete

ICP PILES

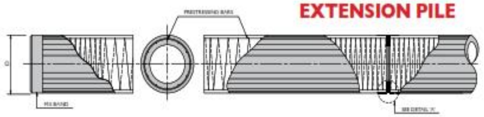
SECTIONAL DETAILS & BONDING INTO PILE CAP

SECTIONAL DETAILS OF ICP PILES

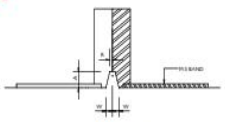
STARTER PILE



EXTENSION PILE



JOINT WELDING DETAILS

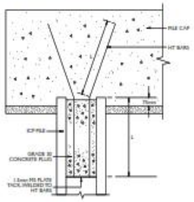


DETAIL 'A'

Dia Of Pile D	Throat Thickness A	W	Root R
mm	mm	mm	mm
250	8.5	4.0	2.0
300	8.5	4.0	2.0
350	8.5	4.0	2.0
400	10	4.5	2.0
450	10	4.5	2.0
500	12	5.0	2.0
600	12	5.0	2.0
700	14	6.0	2.0
800	14	6.0	2.0
900	14	6.0	2.0
1000	14	6.0	2.0
1200	14	6.0	2.0

BONDING ICP PILES INTO PILE CAP

Dia Of Pile	HT Bars Quantity	HT Bars Dia.	HT Bars L
mm	mm	mm	mm
250	4	12	500
300	4	12	500
350	5	12	550
400	5	12	700
450	5	16	800
500	6	16	900
600	8	16	1000
700	8	20	1200
800	8	20	1400
900	10	25	1500
1000	12	25	1500
1200	20	25	2000



As the PC bars are bonded with concrete, ICP Piles may be cut off at any point. The piles need not be stripped down to expose the bars and can be bonded to the pile cap as shown in the above sketch. If the piles are not subjected to tensile loads, the recommended H.T. bars are considered adequate.

MANUFACTURING PROCESS

CAGE MAKING/ MOULD SETTING

PC bars in coil form are straightened and cut to correct lengths. The ends are warm-headed to form button heads. The bars are passed through the cage forming machine where spiral wire is automatically spot-welded at the correct spacings. End plates are fitted to the cage. The whole cage is then placed onto the bottom half mould.

CONCRETE FEEDING

Concrete from the computerised batching plant is discharged into a feeding hopper. Concrete is then fed into the bottom half mould. The top half mould is then bolted to the bottom half.

STRESSING

The PC bars are stressed against the mould through a central shaft and stressing plate. The stressing is being carried out in a single operation. This ensures uniformity of stress in all the PC bars and hence straightness of the pile.

PILE SPINNING

The pile is then compacted by the centrifugal spinning machine. Spinnig process squeezes out excess water, thus increases the concrete strength.

STEAM CURING

The pile is sent to the steam tank for rapid curing process in order to achieve the required transfer strength for early demoulding.

DEMOULDING

After demoulding, final QC inspection is carried out according to the specification.

ICP PILES

PROJECT PHOTOS

ICP Piles have been used extensively as foundation piles for power stations, highrise buildings, civil engineering works, bridges, marine structures, harbours, schools and government projects, etc. The piles are widely used in local construction industry and overseas, namely to Brunei, Singapore, Bangladesh, Sri Lanka, the Middle East, Indonesia, Pakistan, Philippines, Vietnam, Myanmar, USA, Canada, New Zealand, Maldives and Samoa.


Aden Flour Mill Jetty at Aden, Yemen


Saigon International Container Terminal, Vung Tau, Vietnam


Club Med, Maldives


Condominium at Amber Road, Singapore


Marina Bay St Petersburg, Florida, USA


One Kilometer Container Terminal at Chittagong Port, Bangladesh


Sabah Oil & Gas Terminal, Sabah

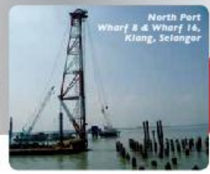

Tg Agas Maritime Industrial Park, Pahang


Aerial View of Completed Piers at 2nd Penang Bridge


Haiwan Petrochemical Port, Dongguan, China


Gurney Paragon, Gurney Drive, Penang


Ocean View Condominium, Butterworth, Penang


North Port Wharf 8 & Wharf 16, Klang, Selangor


CT6 Wharf (B17) Second 300m at Westport, Pulau Indah, Port Klang, Selangor


Janamanjung Power Station, Manjung, Perak

XCP400

起重特性 Load diagrams

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~15.96	20	25	30	35	40	45	50	55	60	65	70	75	80
80	IV	起重量(t) Load	20	15.28	11.66	9.3	7.64	6.41	5.46	4.71	4.1	3.59	3.16	2.79	2.48	2.2
	II		10			9.96	8.33	7.13	6.2	5.46	4.86	4.36	3.94	3.58	3.27	3.0

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.5~20.38	25	30	35	40	45	50	55	60	65	70
70	IV	起重量(t) Load	20	15.74	12.67	10.51	8.91	7.68	6.69	5.9	5.23	4.68	4.2
	II		10				9.65	8.43	7.46	6.67	6.02	5.47	5.0

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~20.83	25	30	35	40	45	50	55	60
60	IV	起重量(t) Load	20	16.15	13	10.8	9.16	7.89	6.89	6.08	5.4
	II		10				9.92	8.67	7.68	6.87	6.2

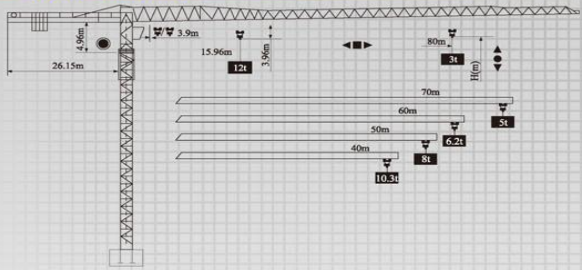
起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~21.51	25	30	35	40	45	50
50	IV	起重量(t) Load	20	16.77	13.52	11.24	9.54	8.24	7.2
	II		10					9.03	8.0

起重臂 (m) Jib	倍率 Fall	起重幅度 (m) Range	3.3~22.84	25	30	35	40
40	IV	起重量(t) Load	20	18.01	14.54	12.11	10.3
	II		10				

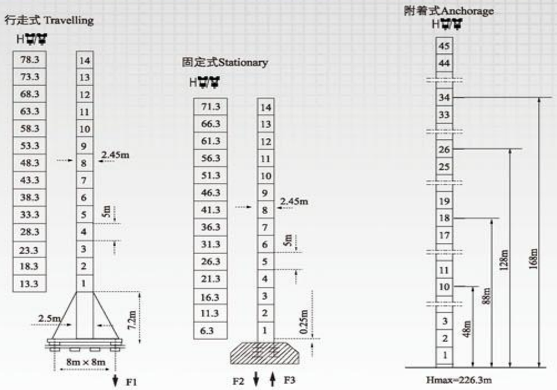
传动机构 Mechanisms

名称 Items	机构代码 Mechanism	工作速度 (m/min) Speed		起重量 (t) Hoist Weight	容绳量 (m) Rope Capacity	电动机 (kw) Motor
起升 Hoisting	122LVF50	双绳 2 fall	0-40	10	750 >750*	90
			0-72	5		
			0-80	2.5		
		四绳 4 fall	0-20	20		
			0-36	10		
			0-40	5		
变幅 Trolleying	X185K	15-30-60				堵转力矩 185Nm
	DVF185	0-60				
回转 Slewing	RCV185	0-0.7r/min				堵转力矩 3×185Nm
	RVF185					
行走 Travelling	RT443	16-32				6×2.6/5.2
电源 Power	380V/50Hz 440V/60Hz					

外形尺寸 DIMENSIONS



塔身截面 Mast 2.45m×2.45m

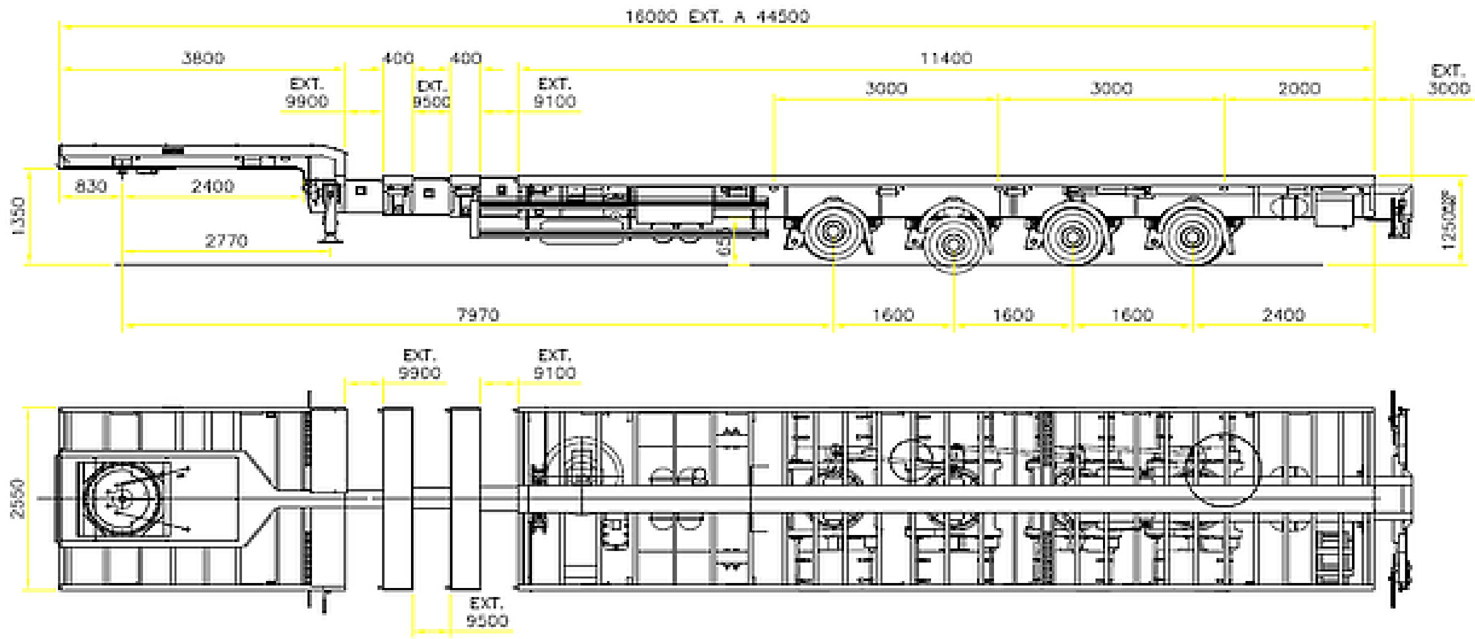


F1	●108t	■114t
自重	138t	

F2	●188t	■205t
F3	●116t	■140t
自重	116t	

F=固定反力 Reactions
● 工作状态 In service
■ 非工作状态 Out of service

自重 Without load and ballast with longest jib and maximum height



* DIRECCIÓN AUTOMÁTICA DESDE 5ª RUEDA
* SUSPENSIÓN HIDRÁULICA
* 16+1 RUEDAS 245/70 R17,5

CODIGO	MARCA	UNDS.	DENOMINACION	MATERIAL	PESO	OBSERVACIONES

A4 DIBUJADO | COMPROBADO

FECHA 05-08

FIRMA D.S.G.

ESCALA 1/75

DNV ISO 9001

INDUSTRIAS Laneko S.A.L

TRAYL-ONA

PLATAFORMA REBAJADA

EXT. 4E S/H 16-44,5 mt.

TP08070000A

SUSTITUIDO POR:

SUSTITUYE A:

KEMENTERIAN RISET, TEKNOLOGI DAN PENDIDIKAN TINGGI
INSTITUT TEKNOLOGI SEPULUH NOPEMBER
JURUSAN TEKNIK SIPIL
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN
Gedung Teknik Sipil Lt. 2 Kampus ITS Sukolilo, Surabaya 60111
Telp : 031-5946094 / Fax : 031-5947284
E-mail : jurusantsipilits@gmail.com
http://ce.its.ac.id / Twitter : @jtsits

SURAT PERJANJIAN MULAI MENGERJAAN TUGAS AKHIR (SP - MMTA)
Nomor : 054267 / IT2.3.I.1/PP.05.02.00/2016

Berdasarkan hasil ujian seminar Proposal Tugas Akhir periode Juni 2016 Semester Genap 2015/2016, dan setelah menyerahkan perbaikan Proposal Tugas Akhirnya, maka mahasiswa yang tercantum di bawah ini :

Nama	:	Soca Fahreza I.
NRP	:	3112100025
Judul Tugas Akhir	:	Redesigning The Building's Structure Of Hotel Novotel THE SAMATOR Surabaya By Using Precast Concrete
Pembimbing TA	:	Data Iranata, ST.MT Ir. Djoko Irawan, MS.
Tanggal Ujian Proposal TA	:	15 Juni 2016
Tanggal Penyerahan Proposal (yang sudah direvisi)	:	21 Juli 2016
Nilai Proposal	:	75

dinyatakan dapat memulai mengerjakan Tugas Akhirnya di bawah bimbingan Dosen yang telah ditetapkan.

Proses pembimbingan berlaku maksimal selama satu semester, terhitung mulai tanggal **13 Juli 2016** sampai dengan tanggal **16 Januari 2017** (buku Tugas Akhir sudah masuk).

Apabila Tugas Akhir tersebut tidak dapat diselesaikan dalam waktu yang telah ditentukan, maka :

a. Bila kemajuan penyusunan Tugas Akhir telah mencapai ≥ 75% akan diberikan perpanjangan waktu satu semester
b. Bila kemajuan penyusunan Tugas Akhir < 75%, diharuskan membuat Proposal Tugas Akhir dengan judul yang baru dan dipresentasikan di depan Team Dosen Penguji.

Demikian Surat Perjanjian ini dibuat untuk dipergunakan sebagai syarat proses pengerjaan Tugas Akhir

Surabaya, 31 AUG 2016
Menyetujui :
Ketua Program Studi S1 Teknik Sipil,

Dr.techn. Umboro lasminto, ST, M.Sc.
NIP. 197212021998021001

Menyetujui :
Mahasiswa,

Soca Fahreza I.
NRP. 3112100025

Menyetujui :
Dosen Pembimbing I,

Data Iranata, ST.MT
NIP. 198004302005011002

Menyetujui :
Pembimbing II,

Ir. Djoko Irawan, MS.
NIP. 195902131987011001

KEMENTERIAN RISET, TEKNOLOGI, DAN PENDIDIKAN TINGGI
INSTITUT TEKNOLOGI SEPULUH NOPEMBER
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FAKULTAS TEKNIK SIPIL DAN PERENCANAAN
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Telp : 031-5946094 / Fax : 031-5947284
E-mail : jurusantsipilits@gmail.com
http://ce.its.ac.id / Twitter : @jtsits

PERSETUJUAN PERPANJANGAN WAKTU MENGERJAKAN TUGAS AKHIR

Nomor : 003845 /IT2.VI.4.1/PP.05.02.00/2017

Sehubungan dengan belum selesainya pengerjaan Tugas Akhir selama 1 (satu) semester, dan berdasarkan hasil penilaian progres pengerjaan Tugas Akhir oleh Dosen Pembimbing yang telah mencapai 75%, maka mahasiswa yang tercantum dibawah ini :

N a m a	:	*Soca Fahreza I.*
N R P	:	*3112100025*
Judul Tugas Akhir	:	*Redesigning The Building's Structure Of Hotel Novotel THE SAMATOR Surabaya By Using Precast Concrete*
Pembimbing Tugas Akhir	:	*Data Iranata, ST. MT. PhD* *Ir. Djoko Irawan, MS.*
SP-MMTA Nomor	:	*054267/IT2.3.I.1/PP.05.02.00/2016*
Masa berlaku SP-MMTA	:	*13 Juli 2016 sampai dengan 16 Januari 2017*

disetujui untuk menyelesaikan Tugas Akhirnya dengan tambahan waktu 1 (satu) semester atau sampai **17 Juli 2017.**

SPMMTA yang dikeluarkan tetap tunduk kepada **Peraturan Akademik**, terutama tentang batas waktu studi.

Demikian Surat Persetujuan ini dibuat untuk dipergunakan sebagai syarat perpanjangan penyelesaian pengerjaan Tugas Akhir.

Surabaya, 20 JAN 2017

Menyetujui :
Mahasiswa,

Soca Fahreza I.
NRP 3112100025

Ketua Program Studi S1 Teknik Sipil,

KEMENTERIAN RISET, TEKNOLOGI DAN PENDIDIKAN TINGGI INSTITUT TEKNOLOGI SEPULUH NOPEMBER FAKULTAS TEKNIK SIPIL DAN PERENCANAAN JURUSAN TEKNIK SIPIL

Dr. techn. Umboro Lasminto, ST.,MSc
NIP. 197212021998021001

Tembusan :
- *Yth. Dosen Pembimbing*

UKAS

PROGRAM STUDI S-1 JURUSAN TEKNIK SIPIL FTSP - ITS

LEMBAR KEGIATAN ASISTENSI TUGAS AKHIR (WAJIB DIISI)

Jurusan Teknik Sipil lt.2, Kampus ITS Sukolilo, Surabaya 601111

Telp.031-5946094, Fax.031-5947284

Form AK/TA-04
rev01

GlobalGROUP ISO 9001 | UKAS MANAGEMENT SYSTEMS 0039

NAMA PEMBIMBING	: Data Iranata, ST, MT, PhD.
NAMA MAHASISWA	: Soca Fahreza I.
NRP	: 3112100025
JUDUL TUGAS AKHIR	: Redesigning The Building's Structure of Hotel Novotel THE SAMATOR Surabaya By Using Precast Concrete
TANGGAL PROPOSAL	: 15 Juni 2016
NO. SP-MMTA	: 054267/IT2.3.I.1/PP.05-02.00/2016

NO	TANGGAL	KEGIATAN		PARAF ASISTEN
		REALISASI	RENCANA MINGGU DEPAN	
1.	15/09/2016	Prelim	- Perbaiki prelim - pelat	
2.	29/09/2016	perbaiki ldl, fc	- perbaiki - lanjut balok anak	
3.	14/11/2016	fc pakai SNI	- Gambar - lanjut tangga & lift	
4.	20/4/2017	Struktur primer + sambungan	- Basement	
5.	2/5/2017	Basement + pondasi	- pondasi - perbaiki	
6.	17/5/2017	Pondasi	- perbaiki hitungan - Gambar	
7.	30/5/2017	Gambar pondasi	- Cek Gambar - perbaiki	
8.	1/6/2017	Gambar	- perbaiki	

KEMENTERIAN RISET, TEKNOLOGI, DAN PENDIDIKAN TINGGI
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Telp : 031-5946094 / Fax : 031-5947284
E-mail : jurusantsipilits@gmail.com
http://ce.its.ac.id / Twitter : @jtsits

SURAT PERJANJIAN MULAI MENGERJAAN TUGAS AKHIR (SP - MMTA)
Nomor : 054867 / IT2.3.I.1/PP.05.02.00/2016

Berdasarkan hasil ujian seminar Proposal Tugas Akhir periode Juni 2016 Semester Genap 2015/2016, dan setelah menyerahkan perbaikan Proposal Tugas Akhirnya, maka mahasiswa yang tercantum di bawah ini :

Nama	:	Soca Fahreza I.
NRP	:	3112100025
Judul Tugas Akhir	:	Redesigning The Building's Structure Of Hotel Novotel THE SAMATOR Surabaya By Using Precast Concrete
Pembimbing TA	:	Data Iranata, ST.MT Ir. Djoko Irawan, MS.
Tanggal Ujian Proposal TA	:	15 Juni 2016
Tanggal Penyerahan Proposal (yang sudah direvisi)	:	21 Juli 2016
Nilai Proposal	:	75

dinyatakan dapat memulai mengerjakan Tugas Akhirnya di bawah bimbingan Dosen yang telah ditetapkan.

Proses pembimbingan berlaku maksimal selama satu semester, terhitung mulai tanggal **13 Juli 2016** sampai dengan tanggal **16 Januari 2017** (buku Tugas Akhir sudah masuk).

Apabila Tugas Akhir tersebut tidak dapat diselesaikan dalam waktu yang telah ditentukan, maka :

a. Bila kemajuan penyusunan Tugas Akhir telah mencapai ≥ 75% akan diberikan perpanjangan waktu satu semester
b. Bila kemajuan penyusunan Tugas Akhir < 75%, diharuskan membuat Proposal Tugas Akhir dengan judul yang baru dan dipresentasikan di depan Team Dosen Penguji.

Demikian Surat Perjanjian ini dibuat untuk dipergunakan sebagai syarat proses pengerjaan Tugas Akhir

Surabaya, 31 AUG 2016

Menyetujui : Mahasiswa,	Menyetujui : Ketua Program Studi S1 Teknik Sipil,
Soca Fahreza I. **NRP. 3112100025**	**Dr.techn. Umboro lasminto, ST, M.Sc.** **NIP. 197212021998021001**
Menyetujui : Dosen Pembimbing I,	Menyetujui : Pembimbing II,
Data Iranata, ST.MT **NIP. 198004302005011002**	**Ir. Djoko Irawan, MS.** **NIP. 195902131987011001**

KEMENTERIAN RISET, TEKNOLOGI, DAN PENDIDIKAN TINGGI
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JURUSAN TEKNIK SIPIL
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN
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E-mail : jurusantsipilits@gmail.com
http://ce.its.ac.id / Twitter : @jtsits

PERSETUJUAN PERPANJANGAN WAKTU MENGERJAKAN TUGAS AKHIR

Nomor : 003845 /IT2.VI.4.1/PP.05.02.00/2017

Sehubungan dengan belum selesainya pengerjaan Tugas Akhir selama 1 (satu) semester, dan berdasarkan hasil penilaian progres pengerjaan Tugas Akhir oleh Dosen Pembimbing yang telah mencapai **75%**, maka mahasiswa yang tercantum dibawah ini :

N a m a	:	*Soca Fahreza I.*
N R P	:	*3112100025*
Judul Tugas Akhir	:	*Redesigning The Building's Structure Of Hotel Novotel THE SAMATOR Surabaya By Using Precast Concrete*
Pembimbing Tugas Akhir	:	*Data Iranata, ST. MT. PhD* *Ir. Djoko Irawan, MS.*
SP-MMTA Nomor	:	*054267/IT2.3.I.1/PP.05.02.00/2016*
Masa berlaku SP-MMTA	:	*13 Juli 2016 sampai dengan 16 Januari 2017*

disetujui untuk menyelesaikan Tugas Akhirnya dengan tambahan waktu 1 (satu) semester atau sampai **17 Juli 2017.**

SPMMTA yang dikeluarkan tetap tunduk kepada **Peraturan Akademik**, terutama tentang batas waktu studi.

Demikian Surat Persetujuan ini dibuat untuk dipergunakan sebagai syarat perpanjangan penyelesaian pengerjaan Tugas Akhir.

Surabaya, 20 JAN 2017

Menyetujui :
Mahasiswa,

Soca Fahreza I.
NRP 3112100025

Ketua Program Studi S1 Teknik Sipil,

Dr. techn. Umboro Lasminto, ST.,MSc
NIP 197212021998021001

KEMENTERIAN RISET, TEKNOLOGI DAN PENDIDIKAN TINGGI INSTITUT TEKNOLOGI SEPULUH NOPEMBER FAKULTAS TEKNIK SIPIL DAN PERENCANAAN JURUSAN TEKNIK SIPIL

Tembusan :
- *Yth. Dosen Pembimbing*

PROGRAM STUDI S-1 JURUSAN TEKNIK SIPIL FTSP - ITS
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Jurusan Teknik Sipil lt.2, Kampus ITS Sukolilo, Surabaya 601111
Telp.031-5946094, Fax.031-5947284

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rev01

NAMA PEMBIMBING	: Ir. Djoko Irawan, MS
NAMA MAHASISWA	: Soca Fahreza I.
NRP	: 3112100025
JUDUL TUGAS AKHIR	: Redesigning the Building's Structure of Hotel Novotel THE SAMATOR Surabaya By Using Precast Concrete
TANGGAL PROPOSAL	: 15 Juni 2016
NO. SP-MMTA	: 054267/IT2.3.I.1/PP.05.02.00/2016

NO	TANGGAL	KEGIATAN: REALISASI	KEGIATAN: RENCANA MINGGU DEPAN	PARAF ASISTEN
1.	15/09/2016	Dicek preum kolom, balok anak (sebelum overtopping) → beban ~~ate~~ pelat, dsb. Gambar penampang dibesarkan. → kontrol lendutan balok. Cek pelat, dan kolom, yang basement pedestalnya jangan digambar (ditiadakan), notasi bedakan	• perbaiki preum • pelat	[signature]
2	29/09/2016	f'c = 3 hari → f'c = 30 MPa. perbaiki ld, f'c, pelat.	• perbaiki • lanjut balok anak	[signature]
3	14/11/2016	f'crack → pakai SNI Gambar PBI 1971	• perbaiki • lanjut tangga & lift • Gambar peraturan harus konsisten	[signature]
4.	13/1/17	tangga SAP	• perbaiki tangga • SAP	[signature]
5	2/02/17	SAP	struktur utama	[signature]
6	31/03/17		buat sketsa gambar. komponen balok di setiap perlu tumpuan	[signature]
7	26/05/17	Gambar pondasi		[signature]
8	16/06/17	Gambar	- perbaiki Ramp & DK. - tulang pelat - Shear Connector	[signature]

BIOGRAPHY

Soca Fahrzea Isma'i was born in Mojokerto, September 1st 1994. He is the son of Drs. Khabib Isma'i and Yuliati. He is the first son of two brothers, currently lives in Surabaya.

He had his elementary school at SDN Beringin 477 Surabaya (2000 - 2006), junior high school at SMPN 26 Surabaya (2006 – 2009), and senior high school at SMAN 11 Surabaya (2009 – 2012). Currently studying at Institut Teknologi Sepuluh Nopember Surabaya in Civil Engineering Departement. The author can be contacted via email socafahreza@yahoo.co.id