FINAL PROJECT (RC14-1501)

REDESIGN FOUNDATION OF CROWN PROJECT CIKARANG WITH PRECAST PRESTRESSED SLAB ON GROUND AND MACHINE FOUNDATION

NATALIA INDAH PERMATA PUTRI
NRP 3111 100 155

Advisors:
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CIVIL ENGINEERING DEPARTMENT
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FINAL PROJECT

To Accomplish a Requirement to Obtain
The Bachelor Degree of Engineering
in
Bachelor Degree of Civil Engineering Program
Faculty of Civil and Planning Engineering
Institut Teknologi Sepuluh Nopember

by:
NATALIA INDAH PERMATA PUTRI
NRP. 3111 100 155

KEMENTERIAN RISET, TEKNOLOGI, DAN PENDIDIKAN TINGGI
INSTITUT TEKNOLOGI SEPULUH NOPEMBER
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN
JURUSAN TEKNIK SIPIL

Had been approved by Final Project Advisors:

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SURABAYA
JULY, 2015

REDESIGN FOUNDATION OF CROWN PROJECT CIKARANG WITH PRECAST PRESTRESSED SLAB ON GORUND AND MACHINE FOUNDATION

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ABSTRACT

This Crown Factory project is located in a good soil. Therefore, it will be easier to design with precast prestressed slabs on grade. The design includes the thickness of slabs and the needed prestressed post-tensioned tendon and reinforcement. The precast panel will be evaluated partly; it means each panel won't influence another panel. So, every panel will be connected with contraction joint and silien as a glue connector. Because of that, this slab is considered as secondary structure. Hence, it's a needed to design structural foundation as part of resisting external forces such as earthquake, wind, and rain. The structural foundation includes reinforcement pile cap and pile.

Not only the design, this final thesis project also identifies the appropriate precast erection method, especially for slab, and calculating the loss of prestressed that occurs from the erection.

Furthermore, this thesis will be analyzed the foundation of machine that considered as dynamic foundation. The design will includes calculating of pile cap and pile.

Keywords: *Soil investigation, slabs on grade, SAFE software, prestressed, post-tensioned, reinforcement, erection method, dynamic foundation, pile.*

FOREWORD

First of all the writer would like to thank God, Jesus Christ-the most inspiration, that the writer can finish this final project report of ”Redesign Foundation of Crown Project with Precast Prestressed Slab on Ground and Machine Foundation”. The writer herself cannot finish this report without any support and assistance from others. I would like to say thank for everyone, especially for:

1. Both of my parents, mom and dad, and my brother, Daniel, who will be a pilot soon, and my twin as well, Natasha, who always take an adventure with me. Thanks for the support, your pray, and love
2. Prof. Tavio as my first advisor who gave big support and encouraged me to write this final project in English although I made many mistakes.
3. Mr. Ananta as my second advisor who gave many contributions, experiences, and knowledge.
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7. All of my best friends from kindergarten till now who couldn’t I say one by one. Thanks for every moment.
8. All other people that the writer cannot mention here one by one that helped him finishing this project.

The writer realizes that this report still needs to be improved. However, the writer hopes that this report will be useful for whom it may concern.

Surabaya, July 2015
Natalia Indah Permata Putri

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CHAPTER 1
INTRODUCTION

1.1 Background

Based on the book of an old theory of architect, there are three primary criteria of a good building (Virtruvius, 2006) – durability,convenience, andaesthetics. The three of them have an equal weight, none is more important than the others. In other words, a good structure meets the demands of durability that depend on the strength or stability, convenience means functionality or usefulness, and esthetic. But, that cost will determine the continuityof the construction. Will it stops or continues?Therefore, this thesis will explain one of the recenttechnologiesin civil engineering that willhelp to solve issues in construction world. *PrecastPrestressed Slab on Grade*for example.Prestressedconcrete is too expensive for most people, because of the high quality material, such as high strength steel and high strength concrete, whereas, many advantages of prestressedcan cover the high cost of prestressed concrete.

Prestressed concrete is no longer a strange type of design. It is rather an extension and modification ofreinforcement concrete with high strength steel and concrete (Lin and Burns, 1981).By prestressing and anchoring the steel against the concrete, we produce desirable stresses and strains in both materials.As a result, it has the ability to resist the more load or crack. Beside of the material, curing is also an important thing to make a durable concrete. Curing in precast is much easier and better control then cast in situ.

High costs of prestressed concrete is probably the most common viewpoint among engineers, whereas, the cost will be reduced with some points. First, reducingthe thickness of floor or slab can reduce theoverall building height (especially for high rise building, to avoid the strong of wind load). Second, using precast construction can reduce the total weight of the structure resultingpile can be reduced too.Third, reducing the formwork cost (as long as we use the same dimension of precast). Fourth, lower construction costs. Construct with precast save much time than concrete in-situ. Last, it considerable lower costs of maintenance because of the longer service life.

1.2 Statements of Problem

1. How to design one-way-slab with prestress?
2. How to erect the precast from fabricated area to project area?
3. How to control prestressed concrete strength with occurred load?
4. How to design machine foundation?)Both pile cap and pile)

1.3 Objectives

1. To design the dimension of precast
2. To analyze the post tensioned prestressed
3. To analyze the loss of prestressed
4. To design the appropriate foundation of machine
5. To design the appropriate foundation of each column

1.4 Scopes of Work

1. Soil bearing capacity will be calculated based on soil investigation report that had been investigated by Suryacipta Industrial Estate.
2. The upper structural calculating had been done by the vendor, PT. Bluescope Buildings.
3. The frequency of machine is supposed not disturbing, so there is no calculating of machine amplitude.
4. The previous designs of steel columns are supposed being able to resist earthquake moment. There is no analysis of steel strength.
5. Precast panel are analyzed as partly panel, so there are no joints calculating.
6. There is no comparative study between the previous and recently designin economic aspect.

CHAPTER 2
LITERATURE REVIEW

2.1 Soil Investigation

2.1.1 N-SPT

With N correction:

1. Toward Groundwater (N’) according to Terzaghi& Peck

$$N' = 15+0.5(N-15), \text{ for } N>15 \qquad (1\text{-}1)$$

N’ = 1.25 for gravel or sandy gravel

2. Toward Soil Overburden Pressure (N_2):

$$N_2 = \frac{4.N_1}{1+(0.4\,.\rho_0)} \text{ if } \rho_0 \leq 7.5\ ton/m^2 \qquad (1\text{-}2)$$

$$N_2 = \frac{4.N_1}{3.25+(1.4\times\rho_0)} \text{ if } \rho_0 \geq 7.5\ ton/m^2 \qquad (1\text{-}3)$$

ρ_0 = vertical soil pressure at a depth which is reviewed. N_2 value is should be $\leq 2N_1$, if the correction is obtained that $N_2 > 2N_1$, use $N_2 = N_1$ ($\rho o = \gamma t \times h$)

2.1.2 Pile Foundation

Piles are structural members that are made of steel, concrete, or timber. They are used to build pile foundations, which are deep and which cost more than shallow foundations. Despite the cost, the use of piles often is necessary to ensure structural safety (Das, Seventh Edition, 2007).

2.1.2.1 Estimating Pile Length

Piles can be divided into three major categories, depending on their lengths and mechanism of load transfer to the soil:

1. Point bearing piles
2. Friction piles

3. Compaction piles

1. Point bearing piles

If soil-boring records establish the presence of bedrock or rocklike material at a site within the reasonable depth, pile can be extended to the rock surface. In this case, the ultimate capacity of the piles depends on the load bearing capacity of the under-lying material. This piles are called point bearing capacity.

Piles with pedestals can be constructed on the bed of the hard stratum, and the ultimate pile load may be expressed as

$$Q_u = Q_p + Q_s \tag{1-4}$$

where :

Q_p = load carried at the pile point

= qp x Ap

= α x Np x K x Ap (1-5)

qp=point stress pile

Ap = section area pile

Np = SPT average for 4B upper till 4B bellow pile (B is pile diameter)

K =Soil characteristic coefficient

Q_s= load carried by skin friction developed at the side of the pile (caused by shearing resistance between the soil and the pile)

= qs x As

$$= \beta \times \left(\frac{Ns}{3} + 1\right) \times As \tag{1-6}$$

β = Shaft coefficient intermediate soils for driven pile = 1

Ns = SPT average for planted pile, boundary $3 \leq N \leq 50$

As = Luasselimuttiangtertanam

qs=Teganganakibatgesertiang

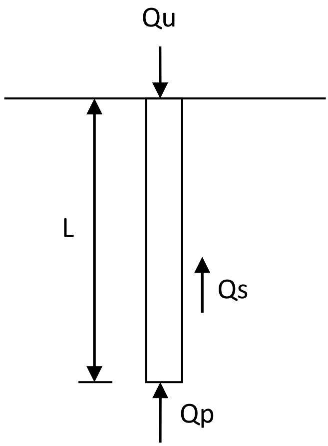

Figure 2.1 Forces that work on poimt bearing piles

2. *Friction piles*

When no layer of rocklike material is present at a reasonable depth at a site, point bearing piles become very long and uneconomical. These piles are called *friction piles*, because the most of their resistance is derived from skin friction.

$$Q_u = Q_s$$

The length of friction piles depend on the shear strength of the soil, the applied load, and the pile size.

3. *Compaction Piles*

Under certain circumstances, piles are driven in granular soils to achieve proper compaction of soil close to the ground surface. These piles are called compaction piles. The compaction length depends on factors such as; the relative density of the soil before compaction, the desired relative density of the soil after compaction, and the requires depth of compaction.

2.1.2.3 Maximum Load of Every Pile

To calculate or check how many pile will be needed, analyzing the strength of each pile is a must. As the formula bellow

$$P_{max} = \frac{\mathrm{V}}{\mathrm{n}} + \frac{\mathrm{M_x \times Y_{max}}}{\sum \mathrm{Y_2}} + \frac{\mathrm{M_y \times X_{max}}}{\sum \mathrm{X_2}} \qquad (1\text{-}7)$$

Where:

P_{max}	= Maximum load for one pile
Σ P	= Total axial load occurred
Mx	= Moment in X direction
My	= Moment in Y direction
Xmax	= Absistiangpancangterjauhterhadapgaris beratkelilingtiang
Ymax	= Ordinattiangpancangterjauhterhadapgaris beratkelilingtiang
$\sum X^2$	= Jumlahkuadratabsistiangpancangterhadap garisberatkelompoktiang
$\sum Y^2$	= Jumlahkuadratordinattiangpancangterhadap garisberatkelompoktiang
n	= total of pile = 48

2.1.2.3 Group Efficiency

In most cases, piles are used in groups to transmit the structural load to the soil. A pile cap is constructed over group piles. The cap can be contacted with the ground or well above the ground.

The efficiency of the load-bearing capacity of the group pile may be defined as

$$\eta = \sqrt{\frac{\mathrm{Qb}^2}{\mathrm{Qb}^2 + \mathrm{nQ1}^2}} \qquad (1\text{-}8)$$

2.2 Precast Slab Concrete

All slab dimension are based on SNI 7833:2012, Tata Cara PerancanganBetonPracetakdanBetonPrateganguntukBangunanGedung.

2.2.1 Slab Thickness

Slam thickness will be considered base on their type and dimension. PTI has had the standard of thickness

Table 2.1 Maximum Span-to-Depth Ratios for Post-Tensioned Flat Slabs (Post Tensioning Institute)

One-way slab	48
Two-way slab	45
Two-way slab with drop panel	50
Two way-slab with two-way beams	55
Waffle (5 x 5 grid)	35
Beams b=h/3	20
Beams b=3h	30

2.2.2 Decking Concrete (d)

According to SNI 7843:2012 chap. 4.6.2.3.3, tolerance of concrete decking is based on the thickness of slab

Table 2.2 Tolerance of d

Slab thickness	Tolerance of d
$d \leq 200$mm	±10mm
$d \geq 200$mm	± 13mm

2.3 Prestressing

Because of high creep and shrinkage losses in concrete, effective prestressing can be achieved by using very high strength steels in the range of 1,862 MPa or higher. Such high strength steels are able to counterbalance these losses in the surrounding concrete and have adequate leftover stress levels to sustain the required prestressing force.

Prestressing reinforcement can be in the form of single wires, strands composed of several wires twisted to form a single element, and high strength bars.

2.3.1 ACI Maximum Permissible Stresses in Concrete and Reinforcement

Following are definitions of some important mathematical term used in calculating.

f_{py} = specified yield strength of prestressing tendons (MPa)
f_y = specified yield strength of non-prestressed reinforcement(MPa)
f_{pu} = specified tensile strength of prestressing tendons (MPa)
f'_c = specified compressive strength of concrete (MPa)
f'_{ci} = compressive strength of concrete at time of initial prestress

2.3.1.1 Concrete Stresses in Flexure

Stresses in concrete immediately after prestress transfer (before time dependent prestress losses) shall not exceed the following:

a) Extreme fiber stress in comparison $0.60f'_{ci}$
b) Extreme fiber stress in tension except as permitted in (c) $3\sqrt{f'ci}$
c) Extreme fiber stress in tension at ends of simply

Where computed tensile stresses exceed these values, bonded auxiliary reinforcement (non-prestresses or prestressed) shaal be provided in the tensile zone to resist the total tensile force in concrete computed under the assumption of an uncracked section.

Stresses in concrete at service loads (after allowance for all prestress losses) shall not exceed the following:

a) Extreme fiber stress in compression due to prestress plus sustained load, where sustained dead load and live load are a large part of the total service load $0.45f'_c$

b) Extreme fiber stress in compression due to prestress plus total load, id the live load is transient $0.60f'_{ci}$

c) Extreme fiber stress in tension in precompressed tensile zone $6\sqrt{f'c}$

d) Extreme fiber stress in tension inprecompressed tensile zone of member (except way slab systems), where analysis based on transformed cracked section and on bilinear moment-deflection relationship shows that immediate and long-time deflection comply with the ACI definition requirements and mimimum concrete cover requirements $12\sqrt{f'c}$

2.3.1.2 Prestressing Steel Stresses

Tensile stress in prestressing tendons shall not exceed the following:

a) Due to jacking force $0.94f_{py}$, but not greater than the lesser of $0.80f_{pu}$ and the maximum value recomendedby the manufacturer of prestressing tendons or anchorages.

b) Immidiately after prestress transfer $0.82f_{py}$, but not greater than of $0.74f_{pu}$

c) Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage $0.70f_{pu}$

2.3.2 Prestressing System and Anchorage

2.3.2.1 Pretensioning

Prestressing steel is pretensioned against independent anchorages prior to the placement of concrete around it. Such anchorages are supported by large and stable bullheads to support the exceedingly high concentrated forces applied to the individual tendons. Prestressing can be accomplished by prestressing individual strands, or all the strands at one jacking operation.

2.3.2.2 Post Tensioning

In post-tensioning, the strands, wires, or bars are tensioned after hardening of the concrete. The strands are placed in the longitudinal ducts within the precast concrete element. The prestressing force is transferred through end anchorages. The tendons of strands should not be bonded or grouted prior to full prestressing.

2.3.2.3 Jacking System

One of fundamental components of a prestressing operation is the jacking system applied, i.e., the manner in which the prestressing force is transferred to the steel tendons.

2.3.3 Loss of Prestress

It is a well established fact that the initial prestressing force applied to the concrete element undergoes a progressive process over a period of approximately five years. Consequently, it is important to determine the level of prestressing force at each loading stage, from the stage of transfer of the prestressing force to the concrete to the various stages of prestressing available at the service load, up to the ultimate. Essentially, the reduction in the prestressing force can be grouped into two categories:

- Immediate elastic loss during the fabrication or construction process, including elastic shorthening, anchorage loasses, and frictional losses.
- Time dependent losses such as creep, shrinkage, and those due to temperature effects and steel relaxation, all of which are determinable at the service load limit stage of stress in the prestressed concrete element.

A summary of the sources of the separate prestressing losses and the stagesof their occurance is given in Table 2.3. From this table, the total loss in prestress can be calculated for pretemsioned and post-tensioned members as follows:

Table2.3 Types of Prestress Loss

Type of prestress loss	Stage of occurrence: Pretensioned members	Stage of occurrence: Post-tensioned members	Tendon stress loss: During time interval (t_i, t_j)	Tendon stress loss: Total or during life
Elastic shortening of concrete *(ES)*	At transfer	At sequential jacking	. . .	Δf_{pES}
Relaxation of tendons *(R)*	Before and after transfer	After transfer	$\Delta f_{pR}(t_i, t_j)$	Δf_{pR}
Creep of concrete *(CR)*	After transfer	After transfer	$\Delta f_{pC}(t_i, t_j)$	Δf_{pCR}
Shrinkage of concrete *(SH)*	After transfer	After transfer	$\Delta f_{pS}(t_i, t_j)$	Δf_{pSH}
Friction *(F)*	. . .	At jacking	. . .	Δf_{pF}
Anchorage seating loss *(A)*	. . .	At transfer	. . .	Δf_{pA}
Total	Life	Life	$\Delta f_{pT}(t_i, t_j)$	Δf_{pT}

2.3.2.1 Elastic Shortening of Concrete

Concrete shortens when a prestressing force is applied. As the tendons that are bonded to the adjacent concrete simultaneously shorten, they lose part of the prestresseing force that they carry.

a. Pretensioned Element

For pretensioned (precast) elements, the compressive force imposed on the beam by the tendon results in the longitudinal shorteningof the beam.

$$\Delta f_{pES} = E_s \,\epsilon_{ES} = \frac{E_s P_i}{A_c E_c} = \frac{nP_i}{A_c} = nf_{cs} \qquad (2\text{-}1)$$

b. Post-tensioned Element

In the post-tensioned beams, the elastic shortening loss varies from zero if all tendons are jacked simultaneously to half the value calculated in the pretensioned case if several sequential jacking steps are used, such as jacking two tendons at a time. If n is the number of tendons or pairs of tendons sequentially tensioned, then

$$\Delta f_{pES} = \frac{1}{n}\sum_{j=1}^{n} (\Delta f_{pES})j \qquad (2\text{-}2)$$

where j denotes the number of jacking operations. Note that the tendon that was tensioned last does not suffer any losses due to elastic shortening, while the tendon that was tensioned first suffers the maximum amount of loss.

2.3.2.2 Steel Stress Relaxation (R)

Stress relieved tendons suffer loss in the prestressing force due to constant elongation with time. The magnitude of the decrease in the prestress depends not only the duration of the sustained prestressing force, but also on the ratio $f_{pi}/f_{py}$of the initial prestress to the yield strength if the reinforcement. Such a loss in stress is termed stress relaxation.

The ACI 318-05 Code limits the tensile stressin the prestressing tendons to the following:

a) For stresses due to the tendon jacking force, $f_{pJ} = 0.94f_{py}$, but not greater than the lesserof $0.80f_{pu}$ and the maximum value recommended by the manufacturer of the tendons and anchorages.
b) Immediately after prestress transfer, $f_{pi} = 0.82f_{py}$ but not greater than $0.74f_{pu}$
c) In the post-tensioned tendons, at the anchorages and couplers immediately after the force transfer = $0.74f_{pu}$

The range of values of f_{py} is given by the following:

- Prestressing bars: $f_{py} = 0.8f_{pu}$
- Stress relieved tendons: $f_{py} = 0.85f_{pu}$
- Low relaxation tendons: $f_{py} = 0.9f_{pu}$

The ACi method use the separate contributions of elastic shortening, creep, and shrinkage in the evaluation of the steel stress relaxation loss by means of the equation

$$\Delta f_{pR} = K_{re} - J\Delta(f_{pES} + f_{pCR} + f_{pSH} \times C \qquad (2\text{-}3)$$

The values of K_{re}, J, and C are given in Table 2.4

Table 2.4 Values of C

f_{pi}/f_{pu}	Stress-relieved strand or wire	Stress-relieved bar or low-relaxation strand or wire
0.80		1.28
0.79		1.22
0.78		1.16
0.77		1.11
0.76		1.05
0.75	1.45	1.00
0.74	1.36	0.95
0.73	1.27	0.90
0.72	1.18	0.85
0.71	1.09	0.80
0.70	1.00	0.75
0.69	0.94	0.70
0.68	0.89	0.66
0.67	0.83	0.61
0.66	0.78	0.57
0.65	0.73	0.53
0.64	0.68	0.49
0.63	0.63	0.45
0.62	0.58	0.41
0.61	0.53	0.37
0.60	0.49	0.33

Source: Post-Tensioning Institute.

Table 2.4 Values of C

Type of tendon[a]	K_{RE}	J
270 Grade stress-relieved strand or wire	20,000	0.15
250 Grade stress-relieved strand or wire	18,500	0.14
240 or 235 Grade stress-relieved wire	17,600	0.13
270 Grade low-relaxation strand	5,000	0.040
250 Grade low-relaxation wire	4,630	0.037
240 or 235 Grade low-relaxation wire	4,400	0.035
145 or 160 Grade stress-relieved bar	6,000	0.05

[a]In accordance with ASTM A416-74, ASTM A421-76, or ASTM A722-75.

Source: Prestressed Concrete Institute.

2.3.2.2 Creep Loss (CR)

Experimental work over the past half century indicates that flow in materials occurs with time when load or stress exists. This lateral flow or deformation due to the longitudinal stress is termed creep.

The ACI-ASCE Committee expression for evaluating creep loss has essentially the same format as bellow:

$$\Delta f_{pCR} = nK_{CR}(f_{cs} - f_{csd}) \qquad (2\text{-}4)$$

where :

K_{CR} = 2.0 for pretensioned members
= 1.6 for post-tensioned members

f_{cs} = stress in concrete at level of steel cgs immediately after transfer

f_{csd} = stress in concrete at level of steel cgs due to all superimposed dead loads applied after prestressing is accomplished

n = modular ratio = $\frac{E_{ps}}{E_c}$

2.3.2.2 Shrinkage Loss (SH)

As with concrete creep, the magnitude of the shrinkage of concrete is affected by several factors. Size and shape of the member also effect shrinkage. Approximately 80% of shrinkage takes place in the first year of life of the structure.

For post-tensioned members, the loss in prestressing due to shrinkage is somewhat less since some shrinkage has already taken place before post-tensioning. If the relative humidity is taken as a percent value and the V/S ratio effect is considered, the PCI general expression for loss in prestressing due to shrinkage becomes

$$\Delta f_{pSH} = 8.2 \times 10^{-6} K_{SH} E_{ps} (1 - 0.006\frac{V}{S})(100 - RH) \qquad (2\text{-}5)$$

where the K_{SH} is shown in Table 2.5

Table2.5 Values of Ksh for Post-Tensioned Members

Time from end of moist curing to application of prestress, days	1	3	5	7	10	20	30	60
K_{sh}	0.92	0.85	0.80	0.77	0.73	0.64	0.58	0.45

Source: Prestressed Concrete Institute.

2.3.2.2 Loss due to Friction (F)

Loss of prestressing occurs in post-tensioning members due to friction between the tendons and the surrounding concrete ducts. it is influenced by:

a) *Curvature effect* = Tendon form or alignment
b) *Wooble effect* = The local deviations in the alignment

Assuming that the prestress force between the start of the curved portion and its end is small, it is sufficient accurate to use the initial tension for the entire curve, and can be simplified to yield:

$$\Delta f_{pf} = -f_1(\mu\alpha + KL) \qquad (2\text{-}6)$$

Table 2.6 Wooble and Curvature Friction Coefficients

Type of tendon	Wobble coefficient, K per foot	Curvature coefficient, μ
Tendons in flexible metal sheathing		
Wire tendons	0.0010–0.0015	0.15–0.25
7-wire strand	0.0005–0.0020	0.15–0.25
High-strength bars	0.0001–0.0006	0.08–0.30
Tendons in rigid metal duct		
7-wire strand	0.0002	0.15–0.25
Mastic-coated tendons		
Wire tendons and 7-wire strand	0.0010–0.0020	0.05–0.15
Pregreased tendons		
Wire tendons and 7-wire strand	0.0003–0.0020	0.05–0.15

Source: Prestressed Concrete Institute.

`2.3.2.2 Anchorage Seating Losses (A)

Anchorage seating losses occur in post-tensiones members due to the seating of wedges in the anchors when the jacking force is transferred to the anchorage.

$$\Delta f_{pA} = \frac{\Delta_A}{L} E_{ps} \qquad (2\text{-}7)$$

2.4 Mild Steel Reinforcement

Mild-steel reinforcement will be design to resist moment. The top reinforcement will resist negative moment from erection, and the bottom reinforcement will resist positive moment from service load.

There are some variables will be needed to calculate mild steel reinforcement:

(based SNI 2847:2013 chap. 10.2.7.3)

$$\beta_1 = 0.85 - 0.05\left(\frac{fc-28}{7}\right) \qquad (2\text{-}8)$$

(based on Appendix B.8.4.2 SNI 2847:2013)

$$\rho_b = \frac{0{,}85 \times \beta_1 \times f_c'}{400} \times \left(\frac{600}{600+f_y}\right) \qquad (2\text{-}9)$$

(based on Appendix B.10.3.3 SNI 2847:2013)

ρ_{max} = $0.75\rho_b$

(based on SNI 2847:2013 chap. 10.5.1)

$$\rho_{min1} = \frac{0,25 \times \sqrt{f_c'}}{f_y} \qquad (2\text{-}10)$$

$$\rho_{min2} = \frac{1.4}{fy} \qquad (2\text{-}11)$$

(based on SNI 2847:2013 chap. 7.12.2.1)

$\rho_{shrinkage}$ = 0.002

(based on SNI 2857:2013 chap. 7.12.2.1)

reduction factor for flexural reinforcement, ϕ= 0.9

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CHAPTER 4
SLAB ON GROUND DESIGN

4.1 Preliminary Design

Crown project has a building that is used to be office, storage room, and production place. Because of the wide area (almost 3,500m^2), it will be faster to design the foundation with precast slab-on-ground. Figure 4.1 shows the side plan of precast that will be constructed.

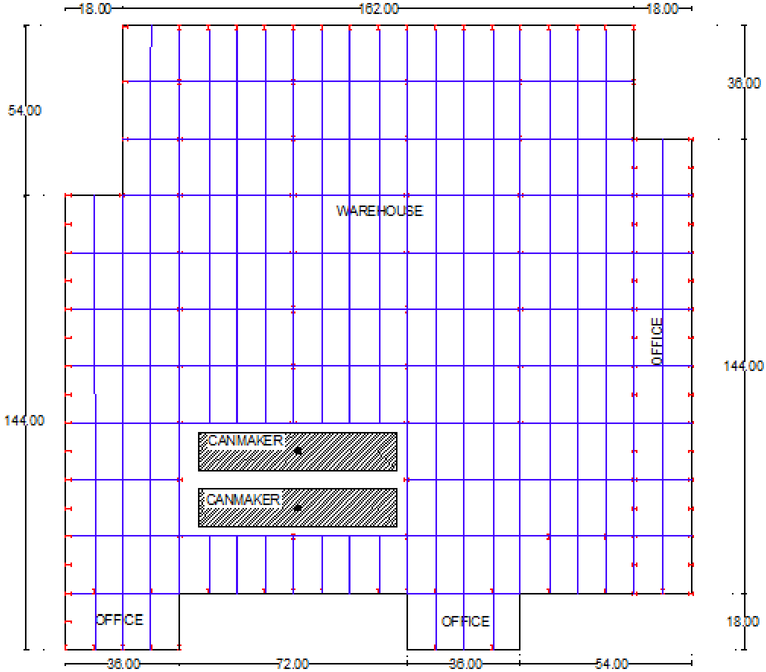


Figure 4.1 Side Plan of Precast

Warehouse and office rooms are planned to be constructed with precast as working floor, they just receive dead load and live load, while the earthquake load will be received by column towards by deep foundation (pile cap and pile), while, for canmaker machine foundation, it will be designed by dynamic foundation.

4.1.1 Slab Thickness

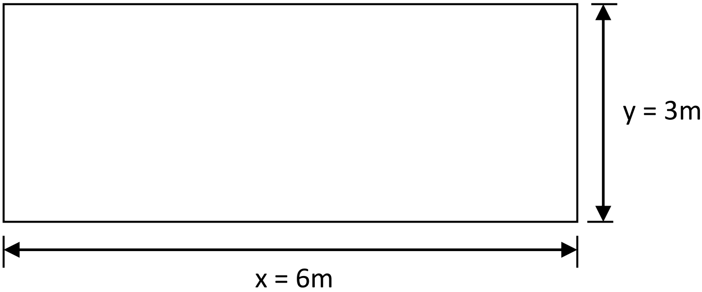


Figure 4.2 Precast slab design

Slab thickness will be considered based on their type and dimension. PTI has had the standard of thickness as shown in Table 4.1.

Table 4.1Maximum Span-to-Depth Ratios for Post-Tensioned Flat Slabs (Post Tensioning Institute)

One-way slab	48
Two-way slab	45
Two-way slab with drop panel	50
Two way-slab with two-way beams	55
Waffle (5 x 5 grid)	35
Beams b=h/3	20
Beams b=3h	30

Slab thickness, $h = \frac{600\text{cm}}{48} = 12.5\text{cm} \approx 25\text{cm}$

Thickness of slab will design 25 cm considered to the room for tendon and mild-steel reinforcement

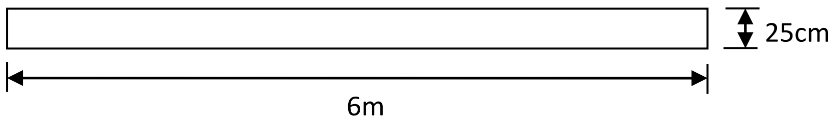


Figure 4.3 Precast slab thicknesses

4.1.2 Design Planning of Slab

A = b x h = 3 x 0.25 = 1.5 m^2 =750,000 mm^2

I $= \frac{1}{4} bh^3$

$= \frac{1}{4} \times 3{,}000 \times 250^3$

$= 1.172 \text{ x } 10^{10} mm^2$

Yt = top boundary = 125 mm

Yb = bottom boundary = 125 mm

E = 200,000 MPa

W_t $= \frac{I}{y_t} = \frac{1.172 \text{ x } 10^{10}}{125} = 93.75 \times 10^6$

W_b $= \frac{I}{y_b} = \frac{1.172 \text{ x } 10^{10}}{125} = 93.75 \times 10^6$

d = concrete cover = 25mm

There is no eccentricity (e=0) in this case, to prevent slab deflection right after installation and before service load.

`

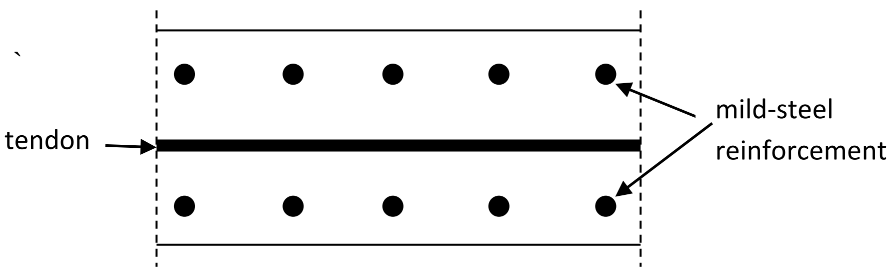


Figure 4.4 Eccentricity of prestress

4.1.3 Prestress Product:

Freyssinetprestress will be used with characteristics and specifications bellow:

- F range anchor, intended for the prestressing of thin elements (slab, concrete floor, etc.)
- Bonded internal prestressing
- Multi strand units 5F/13

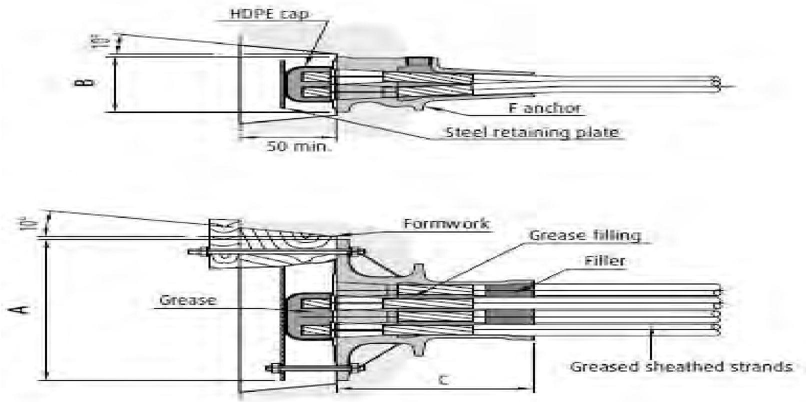


Figure 4.5Anchorage of Prestress

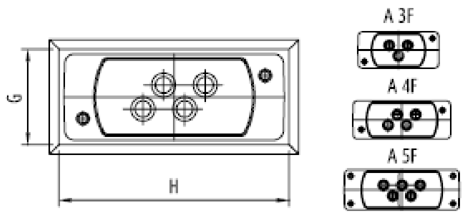


Figure 4.5 Cross Section of Anchorage

Table 4.2 Dimension of Anchorage

Units	A (mm)	B (mm)	C (mm)	G (mm)	H (mm)
A 3F 13/15	190	85	163	95	200
A 4F 13/15	230	90	163	100	240
A 5F 13/15	270	90	163	100	280

Table 4.3 Characteristic of Strands

Standard	Grade MPa	Nominal diameter (mm)	Nominal reinforcement cross-section (mm^2)	Nominal weight (kg/m)	Guaranteed breaking load (Fpk kN)	Elastic limit (Fp0.1 kN)
pr EN 10138-3	1,770	12.5	93	0.73	165	145
		12.9	100	0.78	177	156
		15.3	140	1.09	248	218
		15.7	150	1.18	265	234
	1,860	12.5	93	0.73	173	152
		12.9	100	0.78	186	164
		15.3	140	1.09	260	229
		15.7	150	1.18	279	246

- Nominal Diameter of Strand = 15.7 mm
- Nominal Steel Area of Strand = 150 mm^2
- Breaking Strength, f_{pu} = 1770 MPa

- Yielding Strength, f_{py} = 0.7 x f_{pu} = 1239 MPa
- Elasticity Modulus = 200,000 MPa

4.2 Erection Precast

When the slab is erected, it is supposed as simple beam. It will be lifted up by 4 points. These points are planted in the precast in distance of 0.207L from the edge of slab.

fc’ = 50 MPa = 500 kg/m^2

fy = 410 MPa = 4000 kg/m^2

b = 6 m; a = 3 m

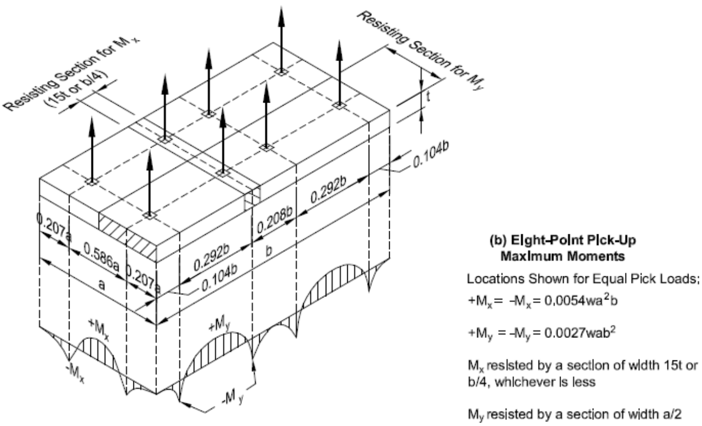


Figure 4.6 Erection Point Pick-up of Precast

4.3 Load and Load Combinations

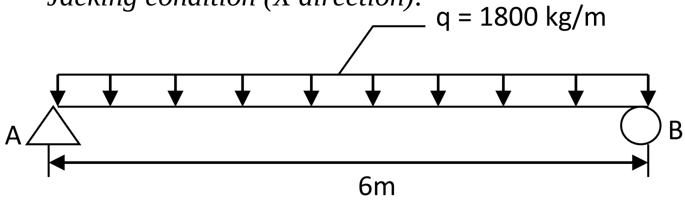
Precast accommodates dead load and live load occur on the slab on ground.

Dead Load (DL) = slab weight that adjusted to the slab thickness, occurred in jacking and erection

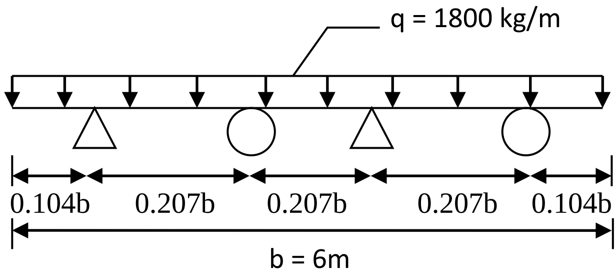
= 2400kg/m^3 x 0.25 m x 3m

= 1800 kg/m

Jacking condition (X direction):



Erection Condition (X direction):



Erection Condition (Y direction):

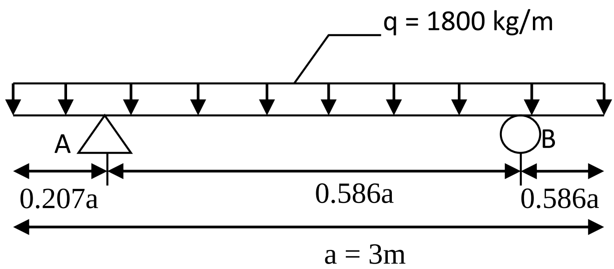
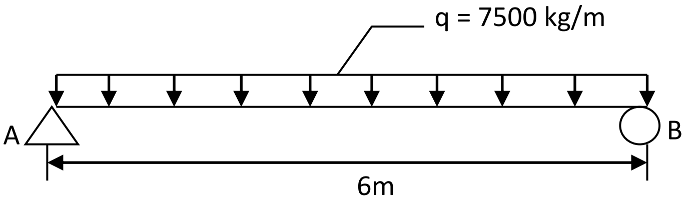


Figure 4.6 Dead Load

Live Load (LL) = vehicle, human, and another load that were approximated by consultant

$= 25 kN/m^2 x\ 3m$
$= 75\ kN/m^2 = 7500\ kg/m$

Sevice condition (X direction):



Service condition (Y direction):

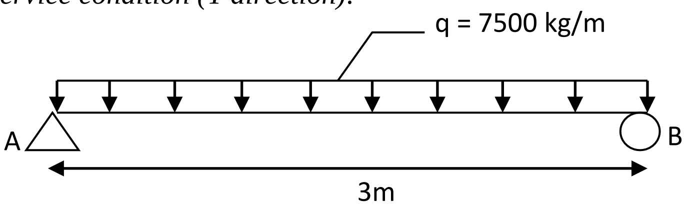


Figure 4.7 Live Load

Load combinations: (SNI 1726-2012 Tata caraperencanaanketahanangempauntukstrukturbangunangedungdan non-gedung), using ultimate stress combination:

a. 1.4D
b. 1.2D + 1.6L+0.5(Lr or R)
c. 1.2D + 1.6(Lr or R)+(L or 0.5W)
d. 1.2D+1.0W+L+0.5(Lr or R)
e. 1.2D+1.0E+L
f. 0.9D+1.0W
g. 0.9D+1.0E

4.4 Element Forces

There are two longitudinal section those will be observed, XZ direction and YZ direction. Element forces in XZ direction will be resisted by tendon and element forces in YZ direction will be resisted by mild steel reinforcement. There are three kinds of condition those will be observed:

1. Precast in fabric before erection (influenced by dead load of self weight) – Condition A
2. Precast when erected (dead load with erected point) – Condition B
3. Precast at service load (dead load and live load) – Condition C

4.4.1 X direction

This sub-chapter shows any kinds of element forces (shear and moment) that occurred in slab both in X direction at jacking, erection and service condition.

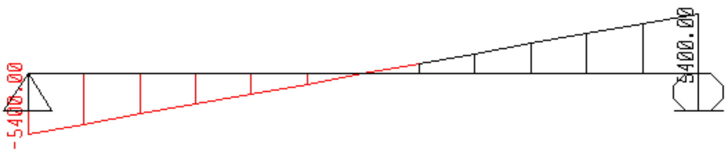


Figure 4.8 Shear Forces in Condition A (X Direction)

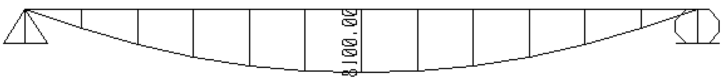


Figure 4.9 Moment Forces in Condition A (X Direction)

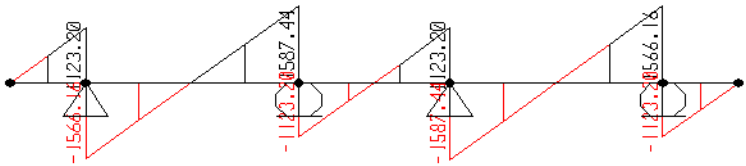


Figure 4.10 Shear Forces in Condition B (X Direction)

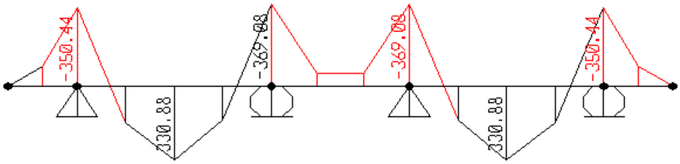


Figure 4.11Moment Forces in Condition B (X Direction)

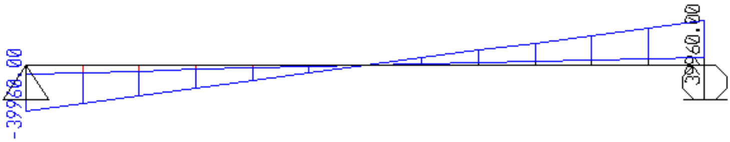


Figure 4.12 Shear Forces in Condition C (X Direction)

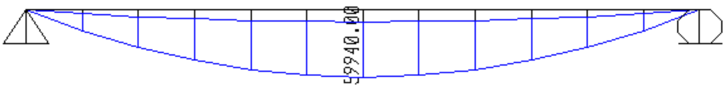


Figure 4.13Moment Forces in Condition C (X Direction)

4.4.2 Y direction

This sub-chapter shows any kinds of element forces (shear and moment) that occurred in slab both in X direction at jacking, erection and service condition.

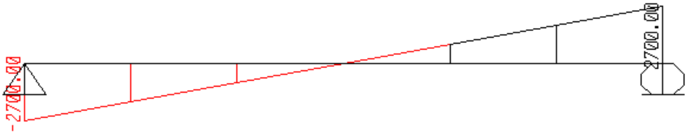


Figure 4.14 Shear Forces in Condition A (Y Direction)

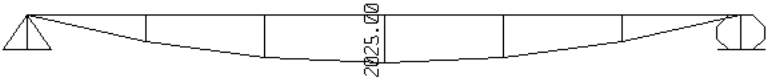


Figure 4.15Moment Forces in Condition A (Y Direction)

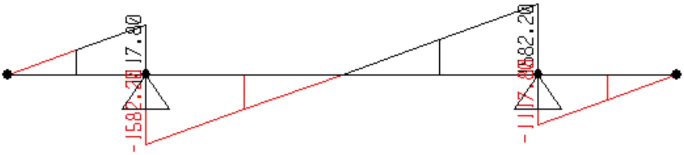


Figure 4.16 Shear Forces in Condition B (Y Direction)

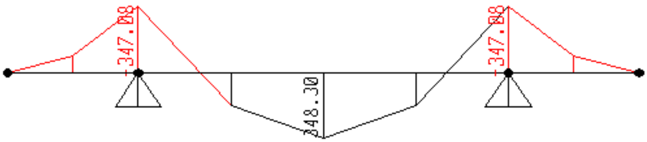


Figure 4.17Moment Forces in Condition B (Y Direction)

Figure 4.18Shear Forces in Condition C (Y Direction)

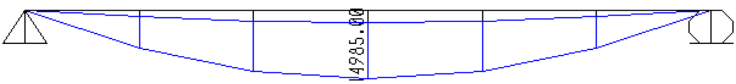


Figure 4.19Moment Forces in Condition C (X Direction)

Table 4.4 Element Forces in X Direction

XZ	Shear (kg)	Moment (kgm)	Moment (Nmm)
DL	5400	8100	81000000
D erection	1587.16	330.88	3308800
DL+LL	39960	59940	599400000

Table 4.5 Element Forces in Y Direction

YZ	Shear (kg)	Moment (kg/m2)	
		M+	M-
DL	2700	2025	-
D erection	1582.2	348.3	347.08
DL+LL	19980	14985	-

4.5 Permissible Stress andInitial Force (Fo)

4.5.1 Maximum Permissible Stresses in Concrete and Reinforcement

According to SNI 7833:2012 chap. 6.4, there are some permissible stresses in concrete and reinforcement. In this book, compression stress will be considered as *minus*, while tension stress will be considered as *plus*.

1. Transfer/jacking/erection condition:

Compression (σ_{c1}) = -0.6 fc’
= -0.6 x 50
= -30 MPa

Tension (σ_{t1}) = $0.5 \times \sqrt{fc'}$
= $0.5 \times \sqrt{50}$
= 3.536 MPa

2. Service:

Compression (σ_{c2}) = -0.45 fc’
= -0.45 x 50
= -22.5 MPa

Tension (σ_{t2}) = $0.25\sqrt{fci'}$
= $0.25\sqrt{50}$
= 1.768 MPa

4.5.2 Initial Forces (Fo)

Inititial force before loss prestress can be approximated (Lin and Burn). The using moment is from the critical moment with envelope combination.

$$\mathrm{Fo} = \frac{\mathrm{M}}{0.65\mathrm{h}} = \frac{599.4\mathrm{kNm}}{0.65 \times 0.25} = 3{,}688.62\mathrm{kN}$$

4.6 Loss of Prestress

The stresses of the distinctive feature of structural system may be tailored to the desired level to assure satisfactory performance. Hence, it is noted that the prestress force used in making the stress computation will not remain constant time. The actual materials and individual circumstances (time elapsed, exposure conditions, dimension, and size of member) must be considered as the time goes by which influence the amount of loss prestresss*(Lin, T.H, Third Edition)*.

There are two kinds of prestress losses as mentioned bellow:

- Short term or stressing losses – These are losses that occurs during and immediately after the post-tensioning operations and are caused by:
  1. Loss due to friction between the tendons and the ducts
  2. Elastic shortening
  3. Seating of anchors
  4. Loss due to steel relaxation
- Long term losses – These types of losses happen over time and also may be referred to as time dependant losses:
  1. Loss due to creep of concrete
  2. Loss due to shrinkage of concrete

4.6.1 Friction Loss

It is known that there is some friction in the jacking and anchorage system, so that the stress existing in the tendon is less than indicated by the pressure gage.

$$\Delta f_{pf} = e^{(-\mu\alpha - KL)}$$

where:

$$\alpha = \frac{8y}{x} = \frac{8 \times 40}{6000} = 0.05333$$

andwooble coefficient (K) and curvature coefficient (μ) are determined by Freyssinet:

K = 0.007

μ = 0.05

L = 6 m

Table 4.6 Friction loss tendon

Segment	L	KL	α	μα	KL+μα	-KL-μα	e^(-KL-μα)	%
AB	6	0.042	0.0667	0.00333	0.04533	-0.045333	0.9557	4.4321

4.6.2 Elastic Shortening of Concrete (ES)

As the prestress is transferred to the concrete, the member shortens and the prestressedsteel shortens with it. Hence, there is a loss of prestress in the steel.

Loss of prestress in steel is:

$$ES = \Delta fs = E_s\delta = \frac{E_s F_0}{A_c E_c} = \frac{nF_0}{A_c}$$

Table 4.7 Precast prestress specification

Precast Prestress Specification		
Fo	3688615	N
d strand	15	mm
n strand	25	
n tendon	5	
A concrete	750000	mm2
A anchorag	28000	mm2
E steel	200000	N/mm2
E concrete	33234.01872	N/mm2

From the data of precast prestress specification, the loss of prestress due to elastic shortening can be calculated:

Table 4.8 Elastic shortening for each tendon

Tendon	n	Fo (N)	Ac (mm2)	Δfs (N/mm2)	Kumulatif	Total (%)
1	6.01793	3,688,615	750000	29.5971	118.3884	6.6886
2	6.01793	3,688,615	750000	29.5971	88.7913	5.0165
3	6.01793	3,688,615	750000	29.5971	59.1942	3.3443
4	6.01793	3,688,615	750000	29.5971	29.5971	1.6722
5	6.01793	3,688,615	750000	29.5971	0	0

4.6.3 Loss Due to Anchorage Take Up

Losses occur due to slip of wires during anchoring or due to strain anchorage is of important in case of post-tensioned system. For any anchoring system, slip is roughly constant. In case of Freyssinet cones, the slip is 6mm for 5mm wires and 9mm for 7mm wires.

Considering the release of strain due to slip Δ_s, as uniform throughout the length L of the wire, the loss of prestress Δ_{fs}, is given by:

$$\Delta_{fs} = E_s = \frac{\Delta_s}{L}$$

But Freyssinet has had determined the loss of slip anchorage is 3%

4.6.4 Loss Due to Steel Relaxation

Test of prestressing steel with constant elongation maintained over a period of time have shown that the prestress force will decrease depends on both time duration and the ration (fpi/fpy). The loss of prestress is called relaxation.

The ACI-ASCE Committee uses the equation bellow to calculate the relaxation loss:

$$RE = (K_{re} - J\,(SH+CR+ES))\,C$$

But Freyssinet had determined the maximum elongation at 1,000 hours under 0.7 fpk for all strands is ≤ 2.5% (5 tendons), 0.5% for 1 tendon.

4.6.5 Loss due to Creep of Concrete

Creep is assumed to occur with the superimposed permanent dead load added to the member after it has been prestressed. Part of the initial compressive strain induced in the concrete immediately after transfer is reduced by the tensile strain resulting from the superimposed permanent dead load.

For unbonded tendons the average compressive stress is used to evaluate losses due to elastic shortening and creep of concrete losses. The losses in the unbounded tendon are related to the average member strain rather than strain at the point of maximum moment. Thus:

$$\mathrm{CR} = \mathrm{K_{cr}} \frac{\mathrm{E_s}}{\mathrm{E_c}} \mathrm{f_{cpa}}$$

K_{cr} = 1.6 for post-tensioned members
fcpa = 3.33 N/mm^2

$$\mathrm{CR} = 1.6 \frac{200000}{33234} 3.33 = 32\ \mathrm{N/mm^2} \text{ (for 5 tendons)}$$

$$\mathrm{CR} = 6.5\ \mathrm{N/mm^2} = 0.36\% \text{ (for 1 tendon)}$$

4.6.6 Loss due to Shrinkage of Concrete

Shrinkage of concrete is influenced by many factors which are most important: volume-to-surface ratio (V/S), relative humidity (RH), and time from end of moist curing to application of prestress. The factors can be seen bellow, as they influenced the product of the effective shrinkage, E_{sh}:

$$\mathrm{E_{sh}} = 8.2 \times 10^{-6} \left(1 - 0.06 \frac{\mathrm{V}}{\mathrm{S}}\right) (100 - \mathrm{RH})$$

Shrinkage loss will be influenced by only other, it's the coefficient K_{sh} which reflects the fact that the post-tensioned members benefit from the shrinkage which occurs prior to the post-tensioning.

$$SH = 8.2 \times 10^{-6} K_{sh} E_s \left(1 - 0.06\frac{V}{S}\right)(100 - RH)$$

Table 4.9 Values of K_{sh} for post-tensioned members

Time after end of moist curing to application of prestress, days	1	3	5	7	10	20	30	60
Ksh	0.92	0.85	0.8	0.77	0.73	0.64	0.58	0.45

K_{sh} = 0.60 (concrete 28 days)
E_s = 200000 N/mm^2
V = 4.5m^3
S = 0.75 m^2
RH = 70%

So, it is calculated as bellow:

$$SH = 8.2 \times 10^{-6} \times 0.6 \times 200000 \left(1 - 0.06\frac{4.5}{0.75}\right)(100 - 70)$$

SH = 18.9% (for 5 tendons)
= 3.78% (for 1 tendon)

Table 4.10 Total Loss for every tendon:

Tendon	ES (%)	CR (%)	SH (%)	RE (%)	FS (%)	FL (%)	Total Loss (%)
1	6.6886	0.3623	3.78	0.5	3	4.3684	18.6993
2	5.0165	0.3623	3.78	0.5	3	4.3684	17.0271
3	3.3443	0.3623	3.78	0.5	3	4.3684	15.3550
4	1.6722	0.3623	3.78	0.5	3	4.3684	13.6828
5	0	0.3623	3.78	0.5	3	4.3684	12.0107

4.7 Control Prestress

4.7.1 Control PrestressForceafter Loss (Fi & Fe)

Prestress Forces will be control in three conditions:

1. Transfer condition (right after jacking)

Elastic shortening and anchorage take-up loss will be occurred in this condition. Hence, Fo will be reduced by elastic shortening and slip anchorage loss.

2. Erection condition

   Fo value is same as Fo in jacking condition but with different moment as consequent of erection precast. Shock factor (1.2) impacts Fo that occurred.

3. Service condition

   All kind of load on slab work that makes some load combination, using moment in envelope combination. All losses include time dependent loss, use total loss of prestress to calculate Fo.

   Because of total loss is around 20%:

   = Fo x 120%

   = 3,688,800 N x 120%

   = 4,425,600 N

1. Transfer/jacking/initial condition:

   Fi = Fo x (1- (ES + FS+FL))

   = 4,425,600 x (1-(0.07+0.03+4.4))

   = 3,803,543N

   M = 8,100,000Nmm

   a. Top fiber stress:

   $f^t \geq fc1$

   $$f^t = \frac{\mathrm{Fi}}{\mathrm{A}} \pm \frac{\mathrm{M}\times\mathrm{yt}}{\mathrm{I}}$$

   $$f^t = -\frac{3{,}803{,}543}{7.5\times10^5} - \frac{8{,}100{,}000\times125}{1.17 \text{ x } 10^{10}}$$

   $$f^t = -5.071 - 0.864$$

   $$f^t = -5.935\ MPa \geq \mathrm{fc1} = -30 \text{ MPa (OK)}$$

   b. Bottom fiber stress:

   $f^b \leq ft1$

   $$f^b = \frac{\mathrm{Fi}}{\mathrm{A}} \pm \frac{\mathrm{M}\times\mathrm{yb}}{\mathrm{I}}$$

   $$f^b = -\frac{3{,}803{,}543}{7.5\times10^5} + \frac{8{,}100{,}000\times125}{1.17 \text{ x } 10^{10}}$$

   $$f^b = -5.071 + 0.864$$

$f^{b} = -4.207\ MPa \leq \text{ft1} = 3.536\text{MPa (OK)}$

2. Erection condition:

Fi = Fo x (1- (ES + FS))
= 4,425,600 x (1-(0.67+0.3))
= 3,803,543kN

M = 3,308,800Nmm

a. Top fiber stress:

$f^{t} \geq \text{fc1}$

$$f^{t} = \frac{\text{Fi}}{\text{A}} \pm \frac{\text{M}\times\text{yt}}{\text{I}}$$

$$f^{t} = -\frac{3{,}803{,}543}{7.5\times10^{5}} - \frac{3{,}308{,}800\times125}{1.17 \text{ x } 10^{10}}$$

$$f^{t} = -5.071 - 0.0353$$

$$f^{t} = -5.107\ MPa \leq \text{fc1} = -30 \text{ MPa (OK)}$$

b. Bottom fiber stress:

$f^{b} \leq \text{ft1}$

$$f^{b} = \frac{\text{Fi}}{\text{A}} \pm \frac{\text{M}\times\text{yb}}{\text{I}}$$

$$f^{b} = \frac{3{,}803{,}543}{7.5\times10^{5}} + \frac{3{,}308{,}800\times125}{1.17 \text{ x } 10^{10}}$$

$$f^{b} = -5.730 + 0.864$$

$$f^{b} = -5.036\ MPa \leq \text{ft1} = 3.536 \text{ MPa (OK)}$$

3. Service condition:

Fe = Fo x (1- (Total Loss))
= 4,425,600x (1-(0.187))
= 3,598,094kN

M = 599,400,000 Nmm

a. Top fiber stress:

$f^{t} \geq \text{fc2}$

$$f^{t} = \frac{\text{Fe}}{\text{A}} \pm \frac{\text{M}\times\text{yt}}{\text{I}}$$

$$f^{t} = \frac{3{,}598{,}094}{7.5\times10^{5}} - \frac{599{,}400{,}000\times125}{1.17 \text{ x } 10^{10}}$$

$$f^{t} = -4.797 - 6.3936$$

$$f^{t} = -11.191 MPa \geq \text{fc2} = -22.5 \text{ MPa (OK)}$$

b. Bottom fiber stress:

$f^b \leq \mathrm{ft2}$

$f^b = \frac{\mathrm{Fe}}{\mathrm{A}} \pm \frac{\mathrm{M \times yb}}{\mathrm{I}}$

$f^b = -\frac{3{,}598{,}094}{7.5 \times 10^5} + \frac{599{,}400{,}000 \times 125}{1.17 \text{ x } 10^{10}}$

$f^b = -4.797 + 6.3936$

$f^b = 1.596\ MPa \leq \mathrm{ft2} = 1.768 \text{ MPa (OK)}$

Top fiber

Table 4.11 Top fiber control

Condition	Tendon	Fo (N)	Losses	Fi or Fe	Fo/A (N/mm2)	My/I (N/mm2)	f top	Permissible fc	Permissible
Transfer	Top	4,425,600	0.141	3,803,543	-5.071	-0.864	-5.935	-30	ft>fc1
Erection	Top	4,425,600	0.141	3,803,543	-5.071	-0.0353	-5.107	-30	ft>fc1
Service	Top	4,425,600	0.187	3,598,094	-4.797	-6.3936	-11.191	-22.5	ft>fc2

Bottom fiber

Table 4.12 Bottom fiber control

Condition	Tendon	Fo (N)	Losses	Fi or Fe	Fo/A (N/mm2)	My/I (N/mm2)	f bottom	Permissible ft	Permissible
Transfer	Bottom	4,425,600	0.141	3,803,543	-5.071	0.864	-4.207	3.536	fb<ft1
Erection	Bottom	4,425,600	0.141	3,803,543	-5.071	0.0353	-5.036	4	fb<ft1
Service	Bottom	4,425,600	0.187	3,598,094	-4.797	6.3936	1.596	1.768	fb<ft2

4.8 Total Tendon Requirement

- Use the minimum Fo = 4,425,600N
- Total strand (n) $= \dfrac{F}{\%\text{jacking} \times fpu \times A}$

$$= \frac{4{,}425{,}600}{0.8 \times 1770 \times 176.715}$$

$$= 20.83 \text{ strands}$$

$$\approx 25 \text{ strands}$$

- Total tendon (1 tendon = 5 strands)
  n = 25/5= 5 tendons
- Distance between tendon
  = 300cm / 6 = 50cm

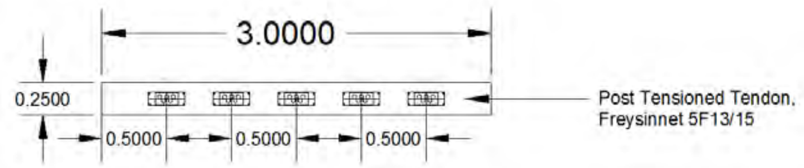


Figure 4.7Anchorage prestresstendon

4.9 Design Control

4.9.1 Punching Shear

- As consequences of **forklift**:

  Slab = 3m x 6m

  Forklift = MHE MFD (Diesel)

  = Wheelbase = 2.25m x 2.25m

  = Load capacity = 8,160kg

  Critical area = 3.375m x 3.375m

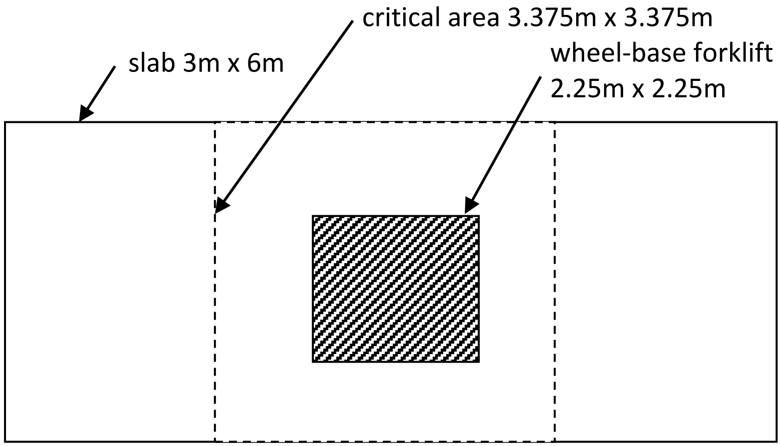


Figure 4.8 Punching shear area

Shear ultimate:

V_u = V x SF

= 8,160kg x 1.5

= 12,240 kg

Permissible shear:

(basedon SNI 2847:2013 chap. 11.11.2.2)

$$V_c = \left(\beta_p \lambda \sqrt{f'_c} + 0.3 f_{pc}\right) b_0 d + V_p$$

where :

$\beta \quad = \frac{Lx}{Ly} = \frac{2}{2} = 1$

$\lambda \quad$ = 1 (for normal weight concrete)

$b_o \quad$ = 4.s = 4 x 337.5 = 1350cm^2

d = 15cm – 2.cover = 15-2(2.5) = 10 cm

Vp = 39,960kg

f_{pc} = 47.97MPa

$$V_c = \left(1.\sqrt{5000} + (0.3 \times 47.97\right)1350\,.\,10 + 39.960$$

= 212,141 kg

Shear forces requirements

ϕVc> Vu

0.75 (291,528) >12,240

218,646 kg> 12,240kg (OK)

4.10 Mild-Steel Reinforcement

Moment in YZ direction will be resisted by mild-steel reinforcement while moment in XZ is resisted by prestresstendon.

4.10.1 Design Specification

Concrete strength, f'c	= 50 MPa
Yield strength, fy	= 420 MPa
Slab thickness, hf	= 250 mm
Decking concrete, d	= 25mm

(based on SNI 2847:2013 chap 7.72, decking concrete d = 25mm)

Reinf.diameter, ∅	= 12 mm
Lx	= 6000 mm
Ly	= 3000 mmm
dy	= hf – d-1/2D
	= 250 – 25-1/2.12

= 219 mm

4.10.2 Stress Occurred

Table 4.13 Element Forces in X Direction

YZ	Shear (kg)	Moment (kg/m2)	
		M+	M-
DL	2700	2025	-
D erection	1582.2	348.3	347.08
DL+LL	19980	14985	-

Mild-steel reinforcement will be design to resist moment. The top reinforcement will resist negative moment from erection, and the bottom reinforcement will resist positive moment from service load.

4.10.3 Reinforcement Needed Calculation

As_{ϕ} $= \frac{1}{4} \times \pi \times D^2 = \frac{1}{4} \times \pi \times 12^2 = 113.1\text{mm}^2$

(based on SNI 2857:2013 chap. 7.12.2.1)

$\rho_{shrinkage}$ = 0.0018 (for slab)

(based on SNI 2857:2013 chap. 7.12.2.1)

reduction factor for flexural reinforcement, ϕ = 0.9

4. 10.3.1 Reinforcement for Service (bottom)

Mu = 14,985 kgm

Mn $= \frac{\text{Mu}}{\phi} = \frac{14{,}985}{0.9} = 16{,}665\text{kgm}$

Rn $= \frac{\text{Mn}}{1\text{m}\times\text{dy}^2} = \frac{16{,}665}{1\text{m}\times 0.219^2} = 3.472 \times 10^5\ \text{kg/m}^2$

= 3.472 N/mm^2

ρ_{perlu} $= \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 50}{350}(1 - \sqrt{1 - \frac{2 \times 3{,}472}{0.85 \times 50}})$

$= 9.912 \times 10^{-7}$ (use ρ_{min})

As_{need} $= \rho_{shrinkage} \times 1m \times dy$

$= 0.0018 \times 1m \times 0.219m$

$= 3.942 \times 10^{-4}\ m^2$

$= 3942\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{394.2}{113.1} = 3.485$

use 4 reinforcements

Space of reinforcements:

n $= \frac{1m}{4} = 250mm$

useD12 - 250mm

4. 9.3.2 Reinforcement for Erection (top)

Mu $= 347.08$ kgm

Mn $= \frac{Mu}{\phi} = \frac{347.08}{0.9} = 385.644$ kgm

Rn $= \frac{Mn}{1m \times dy^2} = \frac{385.644}{1m \times 0.219^2} = 8.041 \times 10^3\ kg/m^2$

$= 0.08041\ N/mm^2$

ρ_{perlu} $= \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$$= \frac{0.85 \times 50}{350}\left(1 - \sqrt{1 - \frac{2 \times 0.08041}{0.85 \times 50}}\right)$$

$= 8.493 \times 10^{-5}$ (use ρ_{min})

As_{need} $= \rho_{shrinkage} \times 1m \times dy$
$= 0.0018 \times 1m \times 0.219m$
$= 3.942 \times 10^{-4}\ m^2$
$= 3942\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{394.2}{113.1} = 3.485$

use 4 reinforcements

Space of reinforcements:

n $= \frac{1m}{4} = 250mm$

useD12 - 250mm

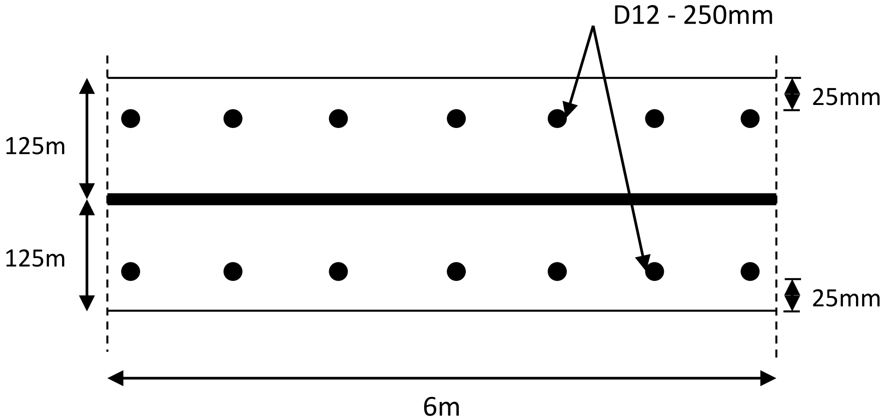


Figure 4.9 Cross section of X direction

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CHAPTER 5
MACHINE FOUNDATION

5.1 Soil Investigation Analysis

Soil investigation analysis was calculated based on data from Geotechnical Investigation Report.

With N correction:

1. Toward Groundwater (N') according to Terzaghi& Peck :

N' = 15+0.5(N-15), for N>15

N' = 1.25 for gravel or sandy gravel

2. Toward Soil Overburden Pressure (N_2):

$N_2 = \frac{4.N_1}{1+(0.4\,.\rho_0)}$ if $\rho_0 \leq 7.5\ ton/m^2$

$N_2 = \frac{4.N_1}{3.25+(1.4\times \rho_0)}$ if $\rho_0 \geq 7.5\ ton/m^2$

ρ_0 = vertical soil pressure at a depth which is reviewed. N_2 value is should be $\leq 2N_1$, if the correction is obtained that $N_2 > 2N_1$, use $N_2 = N_1$ ($\rho o = \gamma t \times h$)/m2 for silty clay

25 t/m2 for sandy silt

40 t/m2 for sand

qp = Tegangandiujungtiang

Ap = Section area pile

Qs = qs x As

$= \beta \times \left(\frac{Ns}{3} + 1\right) \times As$

Where:

β = Shaft coefficient intermediate soils for driven pile = 1

Ns = SPT average for planted pile, boundary $3 \leq N \leq 50$

As = Luasselimuttiangtertanam

qs = Teganganakibatgesertiang

Type of Pile:

Type	d	Ap
spunpile	0.3	0.070686
spunpile	0.4	0.125664
drivenpile	0.25	0.0625

Figure 5.1 Graphic of Allowable Bearing Capacity vs Depth

DEEP	NSPT	SPT correction	Soil Discription	Gs	γt (t/m3)	γ'	ρo	N2	N used
0.5	1	1	CLAY, greyish red spot while, soft, medium plasticity	2.51	1.6	0.6	0.8	3.0303	1
1	2	2		2.51	1.6	0.6	1.6	4.87805	2
1.5	3	3		2.51	1.6	0.6	2.4	6.12245	3
2	4	4		2.51	1.6	0.6	3.2	7.01754	4
2.5	3.75	3.75		2.51	1.6	0.6	4	5.76923	3.75
3	3.5	3.5		2.51	1.6	0.6	4.8	4.79452	3.5
3.5	3.25	3.25		2.51	1.6	0.6	5.6	4.01235	3.25
4	3	3	CLAY, brown spot white, soft	2.51	1.6	0.6	6.4	3.37079	3
4.5	5	5		2.51	1.6	0.6	7.2	5.15464	5
5	7	7	CLAY, yellowish brown, stiff, medium plasticity	2.51	1.6	0.6	8	6.66667	6.67
5.5	9	9		2.51	1.6	0.6	8.8	7.9646	7.96
6	11	11		2.51	1.6	0.6	9.6	9.09091	9.09
6.5	12	12		2.51	1.6	0.6	10.4	9.30233	9.30
7	13	13		2.51	1.6	0.6	11.2	9.48905	9.49
7.5	14	14	CLAY, grey spot yellow, very stiff. Medium plasticity	2.51	1.6	0.6	12	9.65517	9.66
8	16	15.5		2.51	1.6	0.6	12.8	10.4575	10.46
8.5	16.25	15.625		2.64	1.83	0.83	13.6	10.0932	10.09
9	16.5	15.75		2.64	1.83	0.83	14.4	9.76331	9.76
9.5	16.75	15.875		2.64	1.83	0.83	15.2	9.46328	9.46
10	17	16		2.64	1.83	0.83	16	9.18919	9.19
10.5	17.25	16.125	CLAY, grey, hard	2.64	1.83	0.83	16.8	8.93782	8.94
11	17.5	16.25		2.64	1.83	0.83	17.6	8.70647	8.71
11.5	17.75	16.375		2.64	1.83	0.83	18.4	8.49282	8.49
12	18	16.5		2.64	1.83	0.83	19.2	8.29493	8.29

5.2 Allowable Bearing Capacity

Lucciano De'Court method will be used for the clayey soil

$$Ql = Qp + Qs$$

Where:

Ql = Allowable bearing capacity of pile

Qp = Ultimate resistance at the end of pile

Qs = Ultimate resistance at the skin of pile

$$Qp = qp \times Ap$$
$$= \alpha \times Np \times K \times Ap$$

Where:

α = Base coefficient intermediate soil for driven pile = 1

Np = SPT average for 4B upper till 4B bellow pile (B is pile diameter)

K = Soil characteristic coefficient

12 t/m2 for clay

20 t/m2 for silty clay

25 t/m2 for sandy silt

40 t/m2 for sand

qp = Stress at the end of pile

Ap = Section area pile

$$Qs = qs \times As$$
$$= \beta \times \left(\frac{Ns}{3} + 1\right) \times As$$

Where:

β = Shaft coefficient intermediate soils for driven pile = 1

Ns = SPT average for planted pile, boundary $3 \leq N \leq 50$

As = Total area of pile

qs = shear stress of pile

Table 5.2 Q allowable of Pile (diameter 30cm)

D **0.3 m**

Deep (m)	NSPT	N used	Soil Discription	K	Np	qp	Qp (ton)	Ns	qs	As	Qs (ton)	QL (ton)	Qall (ton)
0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5	1	1	CLAY, greyish red spot while, soft, medium plasticity	12	1.5	18	1.272	1	1.333	0.471	0.628	1.901	0.634
1	2	2			2.5	30	2.121	2	1.667	0.942	1.571	3.691	1.230
1.5	3	3			3.188	38.25	2.704	3	2.000	1.414	2.827	5.531	1.844
2	4	4			3.25	39	2.757	3.5	2.167	1.885	4.084	6.841	2.280
2.5	3.75	3.75			3.5	42	2.969	3.625	2.208	2.356	5.203	8.172	2.724
3	3.5	3.5			3.625	43.5	3.075	3.563	2.188	2.827	6.185	9.260	3.087
3.5	3.25	3.25			3.7	44.4	3.138	3.406	2.135	3.299	7.044	10.182	3.394
4	3	3	CLAY, brown spot white, soft	12	3.7	44.4	3.138	3.203	2.068	3.770	7.795	10.934	3.645
4.5	5	5			4.479	53.75	3.799	4.102	2.367	4.241	10.040	13.839	4.613
5	7	6.67	CLAY, yellowish brown, stiff, medium plasticity	25	6.344	158.6109	11.212	5.384	2.795	4.712	13.170	24.381	8.127
5.5	9	7.96			7.605	190.1225	13.439	6.674	3.225	5.184	16.716	30.155	10.052
6	11	9.09			7.919	197.9731	13.994	7.883	3.628	5.655	20.513	34.507	11.502
6.5	12	9.30			9.1	227.5103	16.082	8.592	3.864	6.126	23.672	39.754	13.251
7	13	9.49			9.599	239.9749	16.963	9.041	4.014	6.597	26.479	43.442	14.481
7.5	14	9.66	CLAY, grey spot yellow, very stiff. Medium plasticity	40	9.799	391.9779	27.707	9.348	4.116	7.069	29.094	56.801	18.934
8	16	10.46			9.924	396.9491	28.059	9.903	4.301	7.540	32.428	60.487	20.162
8.5	16.25	10.09			9.886	395.4596	27.953	9.998	4.333	8.011	34.709	62.663	20.888
9	16.5	9.76			9.793	391.7317	27.690	9.881	4.294	8.482	36.419	64.109	21.370
9.5	16.75	9.46			9.489	379.5742	26.831	9.672	4.224	8.954	37.820	64.650	21.550
10	17	9.19			9.359	374.3549	26.462	9.431	4.144	9.425	39.052	65.513	21.838
10.5	17.25	8.94	CLAY, grey, hard	40	8.958	358.3166	25.328	9.184	4.061	9.896	40.192	65.520	21.840
11	17.5	8.71			8.724	348.9699	24.667	8.945	3.982	10.367	41.280	65.947	21.982
11.5	17.75	8.49			8.608	344.3205	24.339	8.719	3.906	10.838	42.339	66.678	22.226
12	18	8.29			8.498	339.923	24.028	8.507	3.836	11.310	43.380	67.408	22.469

Table 5.3 Q allowable of Pile (diameter 40cm)

D **0.4 m**

Deep (m)	NSPT	15.66265	Soil Discription	K	Np	qp	Qp (ton)	Ns	qs	As	Qs (ton)	QL (ton)	Qall (ton)
0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5	1	1	CLAY, greyish red spot while, soft, medium plasticity	12	1.5	18	2.262	1	1.333	0.628	0.838	3.100	1.033
1	2	2			2.292	27.5	3.456	2	1.667	1.257	2.094	5.550	1.850
1.5	3	3			2.464	29.571	3.716	3	2	1.885	3.770	7.486	2.495
2	4	4			2.929	35.143	4.416	3.5	2.167	2.513	5.445	9.862	3.287
2.5	3.75	3.75			3.214	38.571	4.847	3.625	2.208	3.142	6.938	11.785	3.928
3	3.5	3.5			3.643	43.714	5.493	3.563	2.188	3.770	8.247	13.740	4.580
3.5	3.25	3.25			4.167	50	6.283	3.406	2.135	4.398	9.392	15.675	5.225
4	3	3	CLAY, brown spot white, soft	12	4.733	56.796	7.137	3.203	2.068	5.027	10.393	17.531	5.844
4.5	5	5			5.496	65.952	8.288	4.102	2.367	5.655	13.386	21.674	7.225
5	7	6.67	CLAY, yellowish brown, stiff, medium plasticity	25	6.325	158.123	19.870	5.384	2.795	6.283	17.560	37.430	12.477
5.5	9	7.96			7.216	180.406	22.670	6.674	3.225	6.912	22.288	44.959	14.986
6	11	9.09			8.695	217.370	27.315	7.883	3.628	7.540	27.351	54.667	18.222
6.5	12	9.30			8.947	223.665	28.107	8.592	3.864	8.168	31.563	59.670	19.890
7	13	9.49			9.436	235.903	29.644	9.041	4.014	8.796	35.305	64.950	21.650
7.5	14	9.66	CLAY, grey spot yellow, very stiff. Medium plasticity	40	9.477	379.080	47.637	9.348	4.116	9.425	38.792	86.429	28.810
8	16	10.46			9.746	389.850	48.990	9.903	4.301	10.053	43.238	92.228	30.743
8.5	16.25	10.09			9.677	387.065	48.640	9.998	4.333	10.681	46.279	94.919	31.640
9	16.5	9.76			9.651	386.054	48.513	9.881	4.294	11.310	48.559	97.072	32.357
9.5	16.75	9.46			9.516	380.633	47.832	9.672	4.224	11.938	50.426	98.258	32.753
10	17	9.19			9.235	369.406	46.421	9.431	4.144	12.566	52.069	98.490	32.830
10.5	17.25	8.94	CLAY, grey, hard	40	8.978	359.130	45.130	9.184	4.061	13.195	53.589	98.719	32.906
11	17.5	8.71			8.847	353.8967	44.472	8.945	3.982	13.823	55.040	99.512	33.171
11.5	17.75	8.49			8.724	348.9699	43.853	8.719	3.906	14.451	56.452	100.305	33.435
12	18	8.29			8.608	344.320	43.269	8.507	3.836	15.080	57.841	101.109	33.703

Table 5.4 Q allowable of Pile (S = 25cm)

S **0.25 m**

Deep (m)	NSPT	5.919662	Soil Discription	K	Np	qp	Qp (ton)	Ns	qs	As	Qs (ton)	QL (ton)	Qall (ton)
0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5	1	1	CLAY, greyish red spot while, soft, medium plasticity	12	1.5	18	1.125	1	1.333	0.5	0.667	1.792	0.597
1	2	2			2.292	27.500	1.719	2	1.667	1	1.667	3.385	1.128
1.5	3	3			2.464	29.571	1.848	3	2	1.5	3	4.848	1.616
2	4	4			2.929	35.143	2.196	3.5	2.166667	2	4.333	6.530	2.177
2.5	3.75	3.75			3.214	38.571	2.411	3.625	2.208	2.5	5.521	7.932	2.644
3	3.5	3.5			3.643	43.714	2.732	3.563	2.188	3	6.563	9.295	3.098
3.5	3.25	3.25			4.167	50	3.125	3.406	2.135	3.5	7.474	10.599	3.533
4	3	3	CLAY, brown spot white, soft	12	4.733	56.796	3.550	3.203	2.068	4	8.271	11.821	3.940
4.5	5	5			5.496	65.952	4.122	4.102	2.367	4.5	10.652	14.774	4.925
5	7	6.666667	CLAY, yellowish brown, stiff, medium plasticity	25	6.325	158.123	9.883	5.384	2.795	5	13.974	23.856	7.952
5.5	9	7.964602			7.216	180.406	11.275	6.674	3.225	5.5	17.736	29.012	9.671
6	11	9.090909			8.695	217.370	13.586	7.883	3.628	6	21.765	35.351	11.784
6.5	12	9.302326			8.947	223.665	13.979	8.592	3.864	6.5	25.117	39.096	13.032
7	13	9.489051			9.436	235.903	14.744	9.041	4.014	7	28.095	42.839	14.280
7.5	14	9.655172	CLAY, grey spot yellow, very stiff. Medium plasticity	40	9.477	379.080	23.693	9.348	4.116	7.5	30.870	54.562	18.187
8	16	10.45752			9.746	389.850	24.366	9.903	4.301	8	34.407	58.773	19.591
8.5	16.25	10.09317			9.677	387.065	24.192	9.998	4.333	8.5	36.828	61.019	20.340
9	16.5	9.763314			9.651	386.054	24.128	9.881	4.294	9	38.642	62.770	20.923
9.5	16.75	9.463277			9.516	380.633	23.790	9.672	4.224	9.5	40.128	63.917	21.306
10	17	9.189189			9.235	369.406	23.088	9.431	4.144	10	41.435	64.523	21.508
10.5	17.25	8.937824	CLAY, grey, hard	40	8.978	359.130	22.446	9.184	4.061	10.5	42.645	65.090	21.697
11	17.5	8.706468			8.847	353.897	22.119	8.945	3.982	11	43.800	65.918	21.973
11.5	17.75	8.492823			8.724	348.970	21.811	8.719	3.906	11.5	44.923	66.734	22.245
12	18	8.294931			8.608	344.320	21.520	8.507	3.836	12	46.028	67.548	22.516

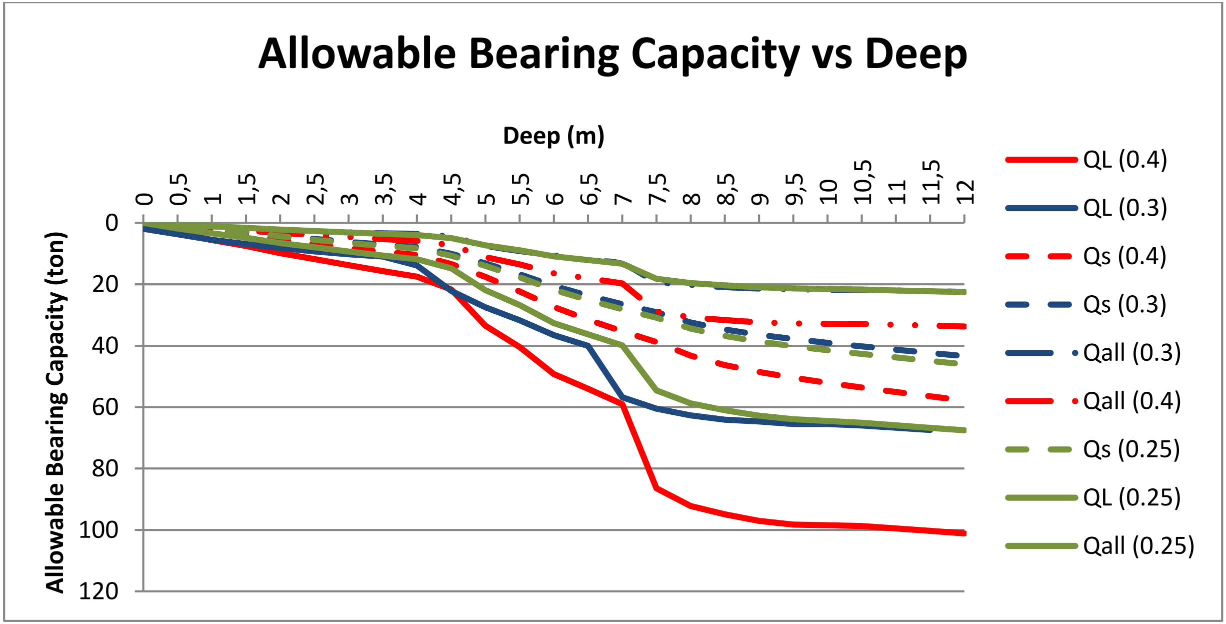


Figure 5.1 Graphic of Allowable Bearing Capacity vs Depth

5.3 Load and Load Combinations

5.3.1 Loading

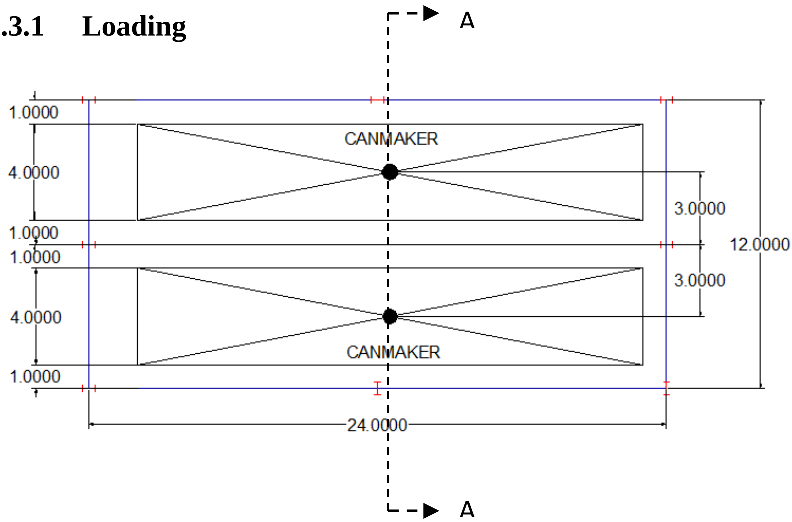


Figure 5.2 Plan Side Machine Foundation

Dynamic forces work in section A-A

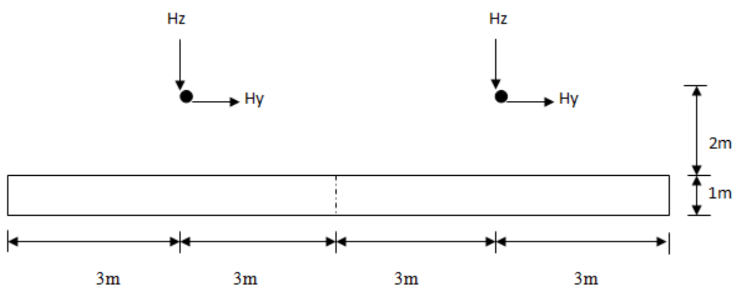


Figure 5.3 Cross Section A-A of Machine Foundation

a. Death Load
- Concrete selfweight
= 24kN/m^3 x 24m x 12m x 1m
= 6,912 kN

b. Live Load
- Live Load
= 25kN/m^2 x 12m x 6m
= 1,800kN

c. Machine Load 1 & 2

- Machine self weight (V)	= 110 kN
- Horizontal Force (Hy)	= 66 kN
- Vertical Force (Hz)	= 144 kN

5.3.2 Load Combination

According to there are various approach to analyze load combinations, the normal operations will be used in this section (based on ACI 351 3R-04, Foundation for Dynamic Equipment).

1. Dead Load
2. Dead load + thermal load + machine forces + live loads + wind + snow (thermal andsnow load are supposed to be zero, while wind load will be resisted by upper structure)
3. Dead load + thermal load + machine forces + seismic load + snow (thermal and snow load are supposed to be zero, seismic load will be resisted by upper structure)

5.3.3 Static Load Analysis

Table 5.5 Load Combination of Static Load

Loading	V	
	kN	ton
Dead Load (D)	6912	691.2
Live Load (L)	1800	180
Machine Force (F1)	110	11
Machine Force (F2)	110	11

Combination 1 = D

LOAD	FACTOR	FORCES (ton)	DISTANCE (m)	MOMENT (ton.m)
		V	Y	Mx
Dead Load	1	691.2		
Total		691.2		0

Combination 2 = D+L+F

LOAD	FACTOR	FORCES (ton)	DISTANCE (m)	MOMENT (ton.m)
		V	Y	Mx
Dead Load	1	691.2		
Live Load	1	180		
Machine Force (F1)	1	11	3	33
Machine Force (F2)	1	11	-3	-33
Total		893.2		0

Combination 3 = D+F

LOAD	FACTOR	FORCES (ton)	DISTANCE (m)	MOMENT (ton.m)
		V	Y	Mx
Dead Load	1	691.2		
Machine Force (F1)	1	11	3	33
Machine Force (F2)	1	11	-3	-33
Total		702.2		0

5.3.4 Dynamic Load Analysis

Loading	V		Hz		Hy	
	kN	ton	kN	ton	kN	ton
Dead Load (D)	6912	691.2				
Live Load (L)	1800	180				
Machine Force 1 (F1)	110	11	144	14.4	66	6.6
Machine Force 2 (F2)	110	11	144	14.4	66	6.6

Table 5.6 Load Combination of Dynamic Load (Both of Machine Work in the Same Direction)

Both of Machine Work (in the same direction)

Combination 1 = D

LOAD	FACTOR	FORCES (ton)			DISTANCE			MOMENT	
		V	Hz	Hy	X	Y	Z	Mx	My
Dead Load	1	691.2							
Total		691.2						0	0

Combination 2 = D+L+F

LOAD	FACTOR	FORCES (ton)			DISTANCE			MOMENT	
		V	Hz	Hy	X	Y	Z	Mx	My
Dead Load	1	691.2							
Live Load (L)	1	180							
Machine Force 1 (F1)	1	11	14.4	6.6	0	3	2	89.4	
Machine Force 2 (F2)	1	11	14.4	6.6	0	-3	2	-63	
Total		893.2	28.8	13.2				26.4	

Combination 3 = D+F

LOAD	FACTOR	FORCES (ton)			DISTANCE			MOMENT	
		V	Hz	Hy	X	Y	Z	Mx	My
Dead Load	1	691.2							
Machine Force 1 (F1)	1	11	14.4	6.6	0	3	2	89.4	0
Machine Force 2 (F2)	1	11	14.4	6.6	0	-3	2	-63	0
Total		713.2	28.8	13.2				26.4	0

Table 5.7 Load Combination of Dynamic Load (One of Machine Work)

One of Machine Works

Combination 1 = D

LOAD	FACTOR	FORCES (ton)			DISTANCE			MOMENT	
		V	Hz	Hy	X	Y	Z	Mx	My
Dead Load	1	691.2							
Total		691.2							

Combination 2 = D+L+F

LOAD	FACTOR	FORCES (ton)			DISTANCE			MOMENT	
		V	Hz	Hy	X	Y	Z	Mx	My
Dead Load	1	691.2							
Live Load (L)	1	180							
Machine Force 1 (F1)	1	11	14.4	6.6	0	3	2	89.4	0
Machine Force 2 (F2)	1	11	0	0	0	-3	2	-33	0
Total		893.2	28.8	13.2				56.4	0

Combination 3 = D+F

LOAD	FACTOR	FORCES (ton)			DISTANCE			MOMENT	
		V	Hz	Hy	X	Y	Z	Mx	My
Dead Load	1	691.2							
Machine Force 1 (F1)	1	11	14.4	6.6	0	3	2	89.4	0
Machine Force 2 (F2)	1	11	0	0	0	-3	2	-33	0
Total		713.2	28.8	13.2				26.4	0

5.4 Pile Analysis

a. Maximum load for every pile

$$P_{max} = \frac{V}{n} + \frac{M_x \times Y_{max}}{\sum Y_2} + \frac{M_y \times X_{max}}{\sum X_2}$$

Where:

P_{max} = Maximum load for one pile

Σ P = Total axial load occurred

Mx = Moment in X direction

My = Moment in Y direction

Xmax = Absistiangpancangterjauhterhadapgaris beratkelilingtiang = 9.6 m

Ymax = Ordinattiangpancangterjauhterhadapgaris beratkelilingtiang = 4 m

$\sum X^2$ = Jumlahkuadratabsistiangpancangterhadap garisberatkelompoktiang
= $(8x2.4^2) + (8x4.8^2) + (8x7.2^2) + (8x9.6^2)$
= $1382.4m^2$

$\sum Y^2$ = Jumlahkuadratordinattiangpancangterhadap garisberatkelompoktiang
= $(18x2^2) + (18x4^2)$
= $360m^2$

n = total of pile = 48

b. Efficiency number::

Pb = 29,974.219 ton

P1 = 33.703 ton

Equation for efficiency:

$$\eta = \sqrt{\frac{Pb^2}{Pb^2+nP1^2}}$$

$$= \sqrt{\frac{29{,}974.219^2}{29{,}974.219^2+(48\times 33.703)^2}}$$

= 0.9985

Table 5.8 Q allowable (Qgroup)(B = 10.4m)

Q group (B=10.4 m, L=21.4 m)

Deep (m)	NSPT	N used	Soil Discription	K	Np	qp	Ap	Qp	Ns	qs	As	Qs	QL	Qall
0	0	0	0	0	0	0	222.56	0	0	0	0	0	0	0
0.5	1	1	CLAY, greyish red spot while, soft, medium plasticity	12	6.695	80.334	222.56	17879.19	1	1.333	31.8	42.400	17921.594	5973.865
1	2	2		12	6.836	82.034	222.56	18257.4	2	1.667	63.6	106.000	18363.402	6121.134
1.5	3	3		12	7.233	86.794	222.56	19316.89	3	2.000	95.4	190.800	19507.695	6502.565
2	4	4		12	7.233	86.794	222.56	19316.89	3.5	2.167	127.2	275.600	19592.495	6530.832
2.5	3.75	3.75		12	7.233	86.794	222.56	19316.89	3.625	2.208	159	351.125	19668.020	6556.007
3	3.5	3.5		12	7.503	90.034	222.56	20037.99	3.563	2.188	190.8	417.375	20455.362	6818.454
3.5	3.25	3.25		12	8.049	96.585	222.56	21496.02	3.406	2.135	222.6	475.344	21971.366	7323.789
4	3	3	CLAY, brown spot white, soft	12	8.489	101.869	222.56	22671.9	3.203	2.068	254.4	526.025	23197.923	7732.641
4.5	5	5		12	8.489	101.869	222.56	22671.9	4.102	2.367	286.2	677.489	23349.387	7783.129
5	7	6.666667	CLAY, yellowish brown, stiff, medium plasticity	25	10.666	266.660	222.56	59347.85	5.384	2.795	318	888.716	60236.568	20078.856
5.5	9	7.964602		25	8.669	216.715	222.56	48232.17	6.674	3.225	349.8	1128.030	49360.204	16453.401
6	11	9.090909		25	8.922	223.043	222.56	49640.55	7.883	3.628	381.6	1384.271	51024.820	17008.273
6.5	12	9.302326		25	9.109	227.735	222.56	50684.74	8.592	3.864	413.4	1597.444	52282.188	17427.396
7	13	9.489051		25	9.109	227.735	222.56	50684.74	9.041	4.014	445.2	1786.850	52471.594	17490.531
7.5	14	9.655172	CLAY, grey spot yellow, very stiff. Medium plasticity	40	9.109	364.376	222.56	81095.59	9.348	4.116	477	1963.327	83058.918	27686.306
8	16	10.45752		40	9.183	367.326	222.56	81752.11	9.903	4.301	508.8	2188.305	83940.411	27980.137
8.5	16.25	10.09317		40	9.298	371.900	222.56	82770.13	9.998	4.333	540.6	2342.232	85112.364	28370.788
9	16.5	9.763314		40	9.375	375.007	222.56	83461.56	9.881	4.294	572.4	2457.625	85919.180	28639.727
9.5	16.75	9.463277		40	9.375	375.007	222.56	83461.56	9.672	4.224	604.2	2552.132	86013.687	28671.229
10	17	9.189189		40	9.375	375.007	222.56	83461.56	9.431	4.144	636	2635.281	86096.837	28698.946
10.5	17.25	8.937824	CLAY, grey, hard	40	9.396	375.839	222.56	83646.84	9.184	4.061	667.8	2712.203	86359.041	28786.347
11	17.5	8.706468		40	9.609	384.379	222.56	85547.44	8.945	3.982	699.6	2785.652	88333.087	29444.362
11.5	17.75	8.492823		40	9.599	383.976	222.56	85457.71	8.719	3.906	731.4	2857.111	88314.820	29438.273
12	18	8.294931		40	9.772	390.885	222.56	86995.27	8.507	3.836	763.2	2927.382	89922.657	29974.219

Static Load

Q allowable = QL pile x efficiency x 0.6 (reducing factor for static load)

Table 5.9 P max and Q allowable Comparing for Static Load

COMBO	FORCES (ton)	MOMENT		ΣP/n	(Mx x Ymax) / ΣY2	(My x Xmax) / ΣX2	Pmax (ton)	Q allowable (D-0.4m)
	V	Mx	My					
Combination 1	691.2	0	0	14.400	0	0	14.400	20.1924
Combination 2	893.2	0	0	18.608	0	0	18.608	20.1924
Combination 3	702.2	0	0	14.629	0	0	14.629	20.1924

Static+Dynamic Load

Q allowable = QL pile x efficiency x 0.8 (reducing factor for static+dynamic load)

Table 5.10 P max and Q allowable Comparing for Static+Dynamic Load

Both of Machine Work (in the same direction)

COMBO	FORCES (ton)			MOMENT		ΣP/n	(Mx x Ymax) / ΣY2	(My x Xmax) / ΣX2	Pmax (ton)	Q allowable (D-0.4m)
	V	Hz	Hy	Mx	My					
Combination 1	691.2	0	0	0	0	14.40	0	0	14.40	26.9232
Combination 2	893.2	28.8	13.2	26.4	0	18.608	0.236	0	18.84	26.9232
Combination 3	713.2	28.8	13.2	26.4	0	14.858	0	0	15.09	26.9232

One of Machine Works

COMBO	FORCES (ton)			MOMENT		ΣP/n	(Mx x Ymax) / ΣY2	(My x Xmax) / ΣX2	Pmax (ton)	Q allowable (D-0.4m)
	V	Hx	Hy	Mx	My					
Combination 1	691.2	0	0	0	0	14.40	0	0	14.40	26.9232
Combination 2	893.2	28.8	13.2	56.4	0	18.61	0.5036	0	19.1119	26.9232
Combination 3	713.2	28.8	13.2	26.4	0	14.86	0.2357	0	15.0940	26.9232

5.5 Control
5.5.1 Lateral Forces Analysis

Table 5.11 Brochure Pile of WIKA

CLASS A (Effective Prestress ≥ 4.0 N/mm ^{2})

Outer Diameter D (mm)	Wall Thickness (mm)	Length L (m)	PC Bar			Area of Concrete (cm^2)	Moment of Inertia Concrete (cm^4)	Calculated Bending Moment		*Allowable Axial Load (t)	Nominal Weight (kg/m)	Effective Prestress (N/mm^2)
			Diam (mm)	Num (pcs)	Area (cm^2)			Cracking (t-m)	Ultimate (t-m)			
300	60	6-12	7.1	6	2.40	452	34,608	2.1	3.5	85	118	4.9
350	60	6-12	7.1	6	2.40	547	59,925	2.8	4.1	104	142	4.1
400	65	6-12	7.1	8	3.20	684	99,577	4.2	6.3	129	178	4.4
450	70	6-12	7.1	10	4.00	836	155,956	6.0	8.9	158	217	4.5
500	80	6-12	7.1	12	4.80	1,056	241,199	8.1	11.8	200	274	4.3
600	90	6-12	9.0	12	7.68	1,442	483,427	14.6	22.7	270	375	4.9

CLASS B (Effective Prestress ≥ 5.0 N/mm ^{2})

Outer Diameter D (mm)	Wall Thickness (mm)	Length L (m)	PC Bar			Area of Concrete (cm^2)	Moment of Inertia Concrete (cm^4)	Calculated Bending Moment		*Allowable Axial Load (t)	Nominal Weight (kg/m)	Effective Prestress (N/mm^2)
			Diam (mm)	Num (pcs)	Area (cm^2)			Cracking (t-m)	Ultimate (t-m)			
250	55	6-12	7.1	6	2.40	337	17,289	1.5	3.0	62	88	6.3
300	60	6-12	7.1	7	2.80	452	34,608	2.3	4.1	84	118	5.6
350	70	6-15	7.1	9	3.60	616	64,115	3.5	6.2	115	160	5.3
400	80	6-15	9.0	8	5.12	804	109,378	5.4	10.1	149	209	5.8
450	80	6-15	9.0	8	5.12	930	166,570	6.8	11.4	174	242	5.1
500	90	6-15	9.0	10	6.40	1,159	255,324	9.4	15.8	217	301	5.1
600	100	6-15	9.0	14	8.96	1,571	510,509	16.0	26.5	293	408	5.2
700	110	6-36	9.0 / 10.7	20 / 14	12.80/ 12.60	2,039	918,012	25.7	43.5	379	530	5.6
800	120	6-36	10.7	18	16.20	2,564	1,527,870	37.8	63.9	475	666	5.7
900	130	6-36	10.7	20	18.00	3,145	2,397,074	50.1	79.9	587	818	5.2
1000	140	6-36	10.7	24	21.60	3,782	3,589,571	67.5	106.5	706	983	5.2

Modulus subgrade reaction of lateral forces:

$$k_s = 2 \times \frac{0.65}{B} \sqrt[12]{\frac{E_s B^4}{E_p I_p}} \times \frac{E_s}{1 - v^2}$$

where:

B =40cm

Es =50 kg/cm^2

Ep = 200,000 MPa

Ip $=\frac{1}{64}\pi D^4 - \frac{1}{64}\pi d^4$

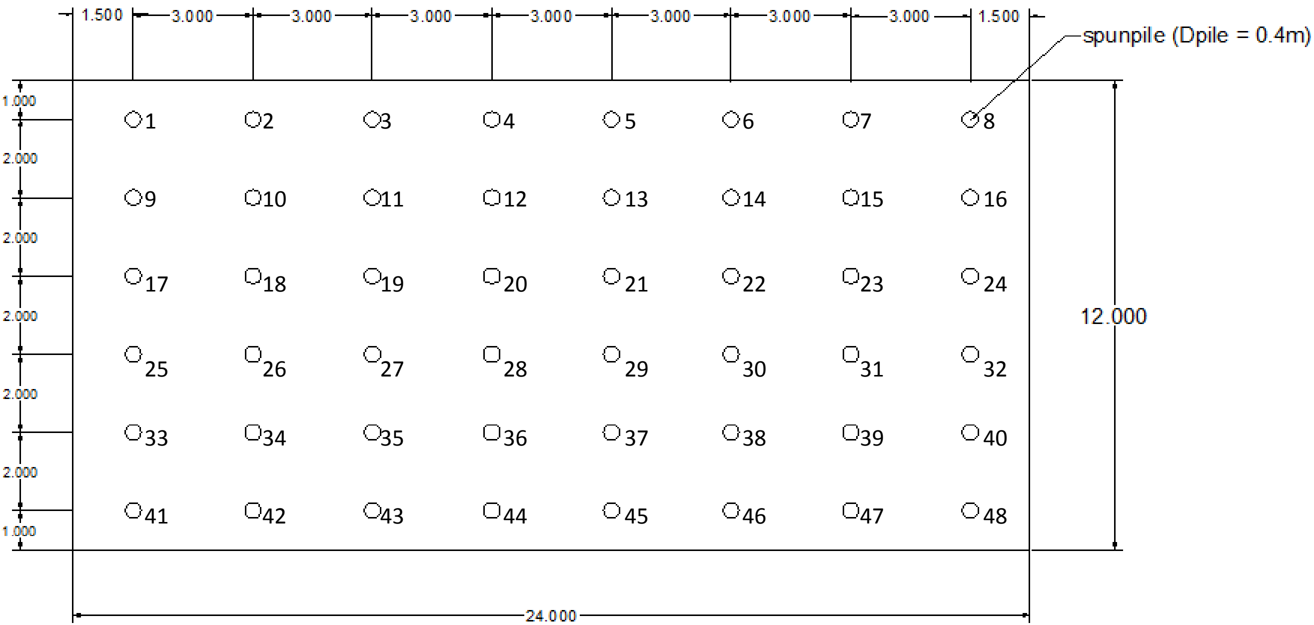
$= \frac{1}{64}\pi 40^4 - \frac{1}{64}\pi 27^4$

$= 125{,}091.15$

v = potion rasio = 0.4

$$k_s = 2 \times \frac{0.65}{40} \sqrt[12]{\frac{5x40^4}{200{,}000 \times 125{,}091.15}} \times \frac{50}{1 - 0.4^2}$$

$= 1.44 \mathrm{kg/cm^3}$


1.500
3.000
3.000
3.000
3.000
3.000
3.000
3.000
1.500
spunpile (Dpile = 0.4m)
1.000
2.000
2.000
2.000
2.000
2.000
1.000
12.000
24.000
1 2 3 4 5 6 7 8
9 10 11 12 13 14 15 16
17 18 19 20 21 22 23 24
25 26 27 28 29 30 31 32
33 34 35 36 37 38 39 40
41 42 43 44 45 46 47 48

Pile	Latitude Pile (%)	P 1 pile
1	77.5	26.120
2	77.5	26.120
3	77.5	26.120
4	77.5	26.120
5	77.5	26.120
6	77.5	26.120
7	77.5	26.120
8	77.5	26.120
9	77.5	26.120
10	55	18.537
11	55	18.537
12	55	18.537
13	55	18.537
14	55	18.537
15	55	18.537
16	55	18.537
17	77.5	26.120
18	55	18.537
19	55	18.537
20	55	18.537
21	55	18.537
22	55	18.537
23	55	18.537
24	55	18.537
25	77.5	26.120
26	55	18.537
27	55	18.537
28	55	18.537
29	55	18.537
30	55	18.537
31	55	18.537
32	55	18.537
33	77.5	26.120
34	55	18.537
35	55	18.537
36	55	18.537
37	55	18.537
38	55	18.537
39	55	18.537
40	55	18.537
41	77.5	26.120
42	77.5	26.120
43	77.5	26.120
44	77.5	26.120
45	77.5	26.120
46	77.5	26.120
47	77.5	26.120
48	77.5	26.120

Lateral forces efficiency/reduction

$n_h = ks \times \xi = 1.44 \text{ x } 0.64 = 0.9216cm^3$

where:

ks = 1.44

ξ= efficiency/reduction modulus of pile group

$= \frac{\text{Total cumulative efficiency}}{n \times Qall}$

$= \frac{1041.424}{48 \times 33.703} = 0.64$

= 64%

$$T = \sqrt[5]{\frac{E_p . I_p}{n_h}}$$

where:

E_p = 200,000 MPa
I_p =125,091.15cm^4
n_h =0.9216

$$T = \sqrt[5]{\frac{200{,}000 \times 125{,}091.15}{0.9216}} = 122.1\text{cm}$$

$$M = (A_m\text{-}0.93B_m) .Q_q . T$$

where:

A_m = Table 6-2 (PondasiBebanDinamis chap VI)
= 1 (right on surface)
B_m = Table 6-1 (PondasiBebanDinamis chap VI)
=1 (right on surface)
Q_q = 6ton + 6ton = 12 ton
T = 122.1cm=1.22cm
M = (A_m-0.93B_m) .Q_q . T
= (1-(0.93x0)) x 12 x 1.22
= 1.025 tm

Compare moment:
M_{pile} = 6.3 tm >$M_{lateral}$= 1.025 tm (OK)
The value of M lateral is really small because it’s just machine force without earthquake force.

5.5.2 Buckling check:

$$\frac{Ip}{A^2} > \frac{\sigma_{max}{}^2}{4. nh. d. Ep}$$

Ip $= 125{,}091.15\text{cm}^4$

A $= \frac{1}{4}\pi D^2 = \frac{1}{4}\pi 40^2 = 2{,}010{,}619\text{cm}^2$

$\sigma_{max}{}^2$ $= (\frac{P}{A})^2 = (\frac{129000}{\pi.20^2})^2 = 10538\text{kg.cm}$

n_h = 0.9216 kg/cm^3

d = 40cm

Ep = 200,000 MPa

$$\frac{125{,}091.15}{2{,}010{,}619^2} > \frac{10{,}538}{4 \times 0.9216 \times 40 \times 200{,}000}$$

0.0622 > 0.000357 (OK)

Because 0.0622 > 0.000357, the buckling will not happen

5.6 Pile Cap

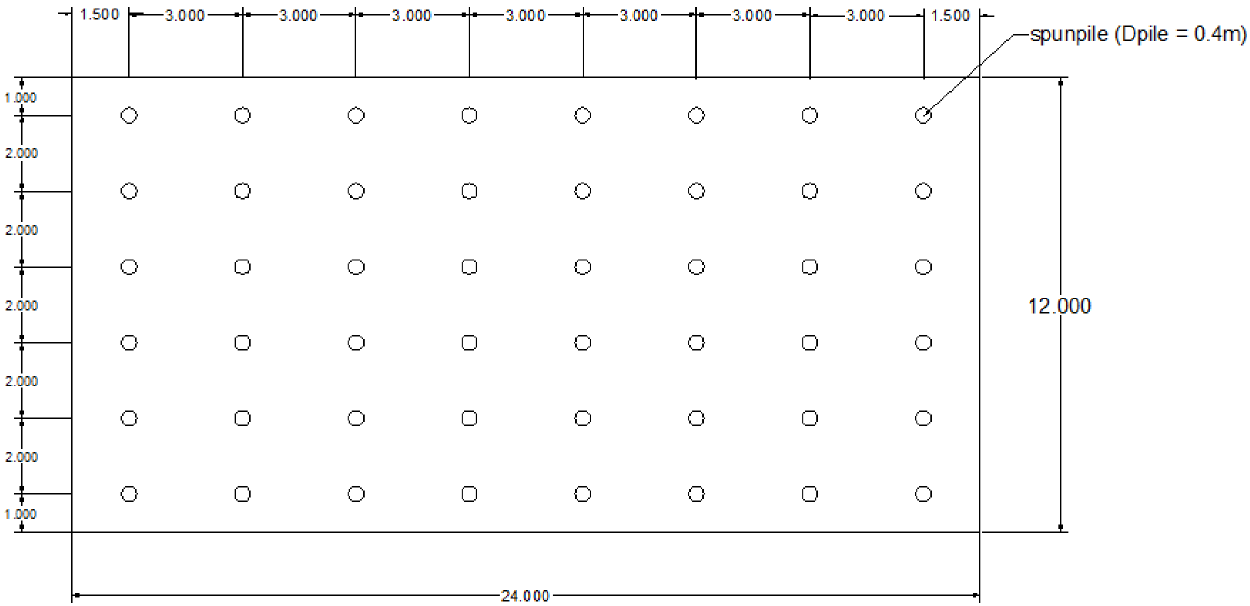


Figure 5.4 Piling location of machine foundation

5.6.1 Punching Shear Control

Punching shear of slab will be checked with thickness hf = 1m as consequence of pile loacation previous design. With the permissible stress:

Punching Shear (as a consequence of pile)

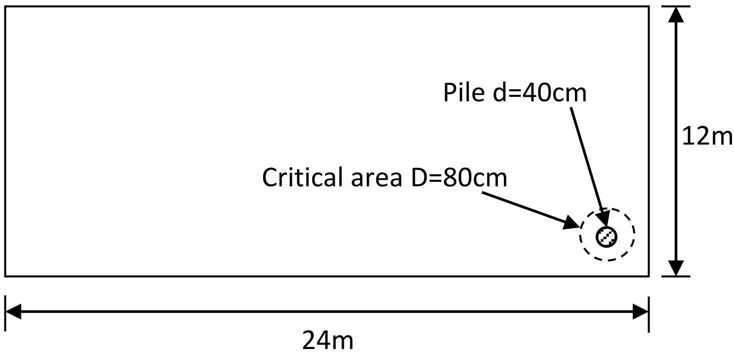


Figure 5.5 Punching shear

Shear ultimate:

V_u = Q_{all} x SF

= 33.703 ton x 1.5

= 50.55 ton

= 50,550 kg

Permissible shear:

(based on SNI 2847:2013 chap. 11.11.2.1)

$V_c = 0.17(1 + \frac{2}{\beta})\ \lambda\ (\sqrt{f'c})b_o d$ or

$V_c = 0.33\lambda\sqrt{f'_c}b_o\ d$

where,

$\beta = \frac{Lx}{Ly} = \frac{24}{12} = 2$

$\lambda = 1$ (for normal weight concrete)
$b_o = \pi D^2 = \pi.80 = 251.33 cm^2$
d = 1m = 100cm

$V_c \quad = 0.17(1 + \frac{2}{2})\sqrt{5000}\ 251.33 . 90$
$\quad = 543{,}814$ kg
$V_c \quad = 0.33\sqrt{5000}\ 251.33 . 90$
$\quad = 527{,}819$ kg

Shear forces requirements
$\phi Vc > Vu$
0.75 (527,819) > 50,550
395.864 kg > 50,550kg (OK)

5.6.2 Design Specification

Concrete strength, f'c	= 30 MPa
Yield strength, fy	= 350 MPa
Slab thickness, hf	= 1m
Decking concrete, d	= 50mm
(based on SNI 2847:2013 chap.7.7.3, d = 50 mm)	
Reinf.diameter, D	= 25 mm
Lx	= 12m
Ly	= 24m
dx	= hf – d-(1/2)D
	= 1000 – 50-(1/2).25
	= 938mm
dy	= hf – d –(3/2)D
	= 1000 – 50 – (3/2).25
	= 913 mm

5.6.3 Stress Occurred

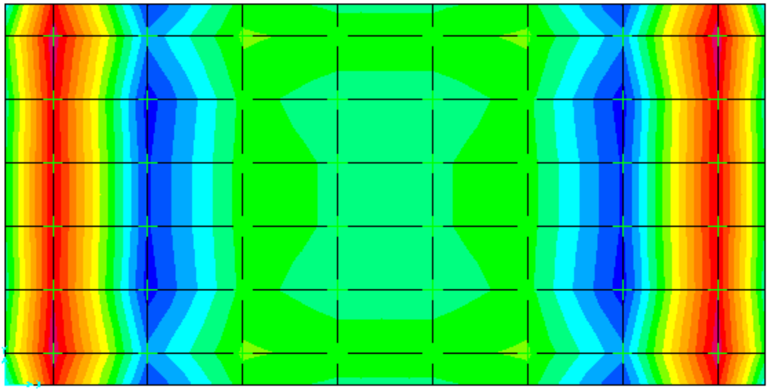


Figure 5.6 M_{11} for reinforcement

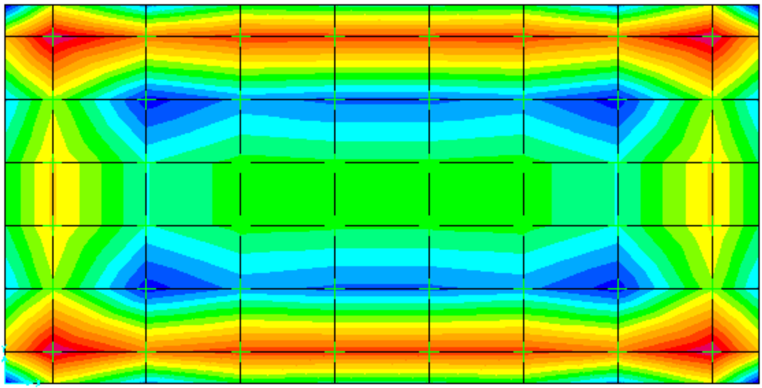


Figure 5.7 M_{11} for reainforcement

Table 5.11 Element forces of slab with envelope combination

Combination	Moment (kgm)		Shear (kg)	
	M11	M22	V13	V23
Envelope	-7648.71	-1573.07	-3155.61	-398
Envelope	-7498.45	-768.88	-3155.92	-398
Envelope	-3737.92	-768.76	-1542.14	-194.5
Envelope	-3664.49	-375.75	-1542.3	-194.5
Envelope	924.68	118.45	-3155.61	-398.19
Envelope	998.57	511.64	-3155.92	-398.19
Envelope	1892.12	242.38	-1542.14	-194.59
Envelope	2043.31	1046.94	-1542.3	-194.59

5.6.4 Reinforcement Needed Calculation

$As_D = \frac{1}{4} \times \pi \times D^2 = \frac{1}{4} \times \pi \times 25^2 = 490.9\text{mm}^2$

(based SNI 2847:2013 chap. 10.2.7.3)

(based on SNI 2857:2013 chap. 7.12.2.1)

$\rho_{shrinkage} = 0.0018$

(based on SNI 2857:2013 chap. 7.12.2.1)

reduction factor of reinforcement, ϕ = 0.9

5.6.4.1 Reinforcement for X direction (M11)

a. Positive Moment (Top)

Mu = 2,043.31kgm

$Mn = \frac{Mu}{\phi} = \frac{2{,}043.31}{0.9} = 2{,}279 \text{ kgm}$

$Rn = \frac{Mn}{1m \times dx^2} = \frac{2{,}279}{1m \times 0{,}938^2} = 2.583 \times 10^3 \text{ kg/m}^2$

$= 0.02583 \text{ N/mm}^2$

ρ_{perlu} $= \frac{0.85\times fc}{fy}(1-\sqrt{1-\frac{2Rn}{0.85\times fc}})$

$= \frac{0.85\times 30}{350}(1-\sqrt{1-\frac{2\times 0.02583}{0.85\times 30}})$

$= 7.384 \times 10^{-5}$ (use ρ_{min})

As_{need} $= \rho_{susut} \times 1m \times dx$

$= 0.0018 \times 1m \times 0.938m$

$= 1.688 \times 10^{-3}\ m^2$

$= 1688\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_D} = \frac{1688}{490.9} =$ 3.438 (use 4 reinf.)

s $= \frac{1m}{n} = 250mm$

useD25-250mm

b. Negative Moment(bottom)

Mu = 7,648.71 kgm

Mn $= \frac{Mu}{\phi} = \frac{7.648.71}{0.9} = 8{,}499$ kgm

Rn $= \frac{Mn}{1m\times dx^2} = \frac{8{,}499}{1m\times 0.9^2} = 1.049 \times 10^4\ kg/m^2$

$= 0.01049\ N/mm^2$

ρ_{perlu} $= \frac{0.85\times fc}{fy}(1-\sqrt{1-\frac{2Rn}{0.85\times fc}})$

$= \frac{0.85\times 30}{350}(1-\sqrt{1-\frac{2\times 0.01049}{0.85\times 30}})$

$= 3.066 \times 10^{-4}$ (use ρ_{min})

As_{need} $= \rho_{susut} \times 1m \times dx$

$= 0.0018 \times 1m \times 0.938m$

$= 1.688 \times 10^{-3}\ m^2$

$= 1688\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_D} = \frac{1688}{490.9} =$ 3.438 (use 4 reinf.)

s $= \frac{1m}{n} = 250mm$

useD25-250mm

5.6.4.2 Reinforcement for Y direction (M22)

a. *Positive Moment (Top)*

Mu $= 1{,}046.94kgm$

Mn $= \frac{Mu}{\phi} = \frac{1{,}046.94}{0.9} = 1{,}163kgm$

Rn $= \frac{Mn}{1m \times dy^2} = \frac{1{,}163}{1m \times 0.913^2} = 1.387 \times 10^3\ kg/m^2$

$= 0.001387/mm^2$

ρ_{perlu} $= \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 30}{350}(1 - \sqrt{1 - \frac{2 \times 0.00161}{0.85 \times 30}})$

$= 4.591 \times 10^{-5}$(use ρ_{min})

As_{need} $= \rho_{susut} \times 1m \times dx$

$= 0.0018 \times 1m \times 0.913m$

$= 1.642 \times 10^{-3}\ m^2$

$= 1642\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_D} = \frac{1642}{490.9} =$ 3.346 (use 4 reinf.)

s $= \frac{1m}{n} = 250mm$

useD25-250mm

b. Negative Moment(bottom)

Mu = 1,573.07kgm

Mn $= \frac{\mathrm{Mu}}{\phi} = \frac{1{,}573.07}{0.9} = 1748$ kgm

Rn $= \frac{\mathrm{Mn}}{1\mathrm{m}\times \mathrm{dy}^2} = \frac{1748}{1\mathrm{m}\times 0.85^2} = 2.158 \times 10^3$ kg/m^2

= 0.02158 N/mm^2

ρ_{perlu} $= \frac{0.85\times \mathrm{fc}}{\mathrm{fy}}\left(1 - \sqrt{1 - \frac{2Rn}{0.85\times fc}}\right)$

$= \frac{0.85\times 30}{350}\left(1 - \sqrt{1 - \frac{2\times 0.002158}{0.85\times 30}}\right)$

= 0.00004375 (use ρ_{min})

As_{need} = ρ_{susut} x 1m x dx

= 0.0018 x 1m x 0.913m

= 1.642 x 10^{-3} m^2

= 1642 mm^2

Total reinforcement:

n $= \frac{As_{need}}{As_D} = \frac{1642}{490.9} =$ 3.346 (use 4 reinf.)

s $= \frac{1\mathrm{m}}{\mathrm{n}} = 250\mathrm{mm}$

useD25-250mm

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CHAPTER 6
COLUMN FOUNDATION

6.1. Soil Investigation Analysis

Soil investigation analysis was calculated based on data from Geotechnical Investigation Report.

With N correction:

1. Toward Groundwater (N’) according to Terzaghi& Peck :

N’ = 15+0.5(N-15), for N>15

N’ = 1.25 for gravel or sandy gravel

2. Toward Soil Overburden Pressure (N_2):

$N_2 = \frac{4.N_1}{1+(0.4\ .\rho_0)}$ if $\rho_0 \leq 7.5\ ton/m^2$

$N_2 = \frac{4.N_1}{3.25+(1.4\times\ \rho_0\ )}$ if $\rho_0 \geq 7.5\ ton/m^2$

ρ_0 = vertical soil pressure at a depth which is reviewed. N_2 value is should be $\leq 2N_1$, if the correction is obtained that $N_2 > 2N_1$, use $N_2 = N_1$ ($\rho o = \gamma t$ x h)/m2 for silty clay

25 t/m2 for sandy silt

40 t/m2 for sand

qp = Tegangandiujungtiang

Ap = Section area pile

Qs = qs x As

$=\beta\ \times\ \left(\frac{Ns}{3}+1\right)\times$ As

Where:

β = Shaft coefficient intermediate soils for driven pile = 1

Ns = SPT average for planted pile, boundary 3 ≤ N ≤ 50

As = Luasselimuttiangtertanam

qs = Teganganakibatgesertiang

Table 6.1 Soil Investigation and N used of BH-13

DEEP	NSPT	N1	Soil Discription	Gs	γt (t/m3)	γ'	ρo	N2	N used
0.5	5	5	CLAY, yellowish grey red stiff	2.54	1.8	0.8	0.9	14.706	5
1	7	7		2.54	1.8	0.8	1.8	16.279	7
1.5	9	9		2.54	1.8	0.8	2.7	17.308	9
2	12	12		2.54	1.8	0.8	3.6	19.672	12
2.5	13	13		2.54	1.8	0.8	4.5	18.571	13
3	14	14	Silty SAND with gravel, greyish brown, medium, dense	2.54	1.8	0.8	5.4	17.722	14
3.5	15	15		2.54	1.8	0.8	6.3	17.045	15
4	16	15.5		2.54	1.8	0.8	7.2	16.495	15.5
4.5	16	15.5		2.54	1.8	0.8	8.1	15.094	15.094
5	16	15.5		2.54	1.8	0.8	9	13.913	13.91
5.5	17	16		2.54	1.8	0.8	9.9	13.710	13.71
6	17	16		2.54	1.8	0.8	10.8	12.782	12.78
6.5	19	17		2.53	1.81	0.81	11.7	13.380	13.38
7	20	17.5		2.53	1.81	0.81	12.6	13.245	13.25
7.5	22	18.5		2.53	1.81	0.81	13.5	13.750	13.75
8	24	19.5		2.53	1.81	0.81	14.4	14.201	14.20
8.5	20	17.5	Sandy CLAY, brownish yellow grey, very stiff	2.53	1.81	0.81	15.3	11.236	11.24
9	18	16.5		2.53	1.81	0.81	16.2	9.626	9.63
9.5	15	15		2.53	1.81	0.81	17.1	7.653	7.65
10	13	14		2.53	1.81	0.81	18	6.341	6.34
10.5	12	13.5		2.53	1.81	0.81	18.9	5.607	5.61
11	11	13		2.53	1.81	0.81	19.8	4.933	4.93
11.5	10	12.5		2.53	1.81	0.81	20.7	4.310	4.31
12	9	12		2.53	1.81	0.81	21.6	3.734	3.73
12.5	12	13.5	CLAY, grey, stiff, high plasticity	2.53	1.81	0.81	22.5	4.800	4.80
13	14	14.5		2.53	1.81	0.81	23.4	5.405	5.41
13.5	16	15.5		2.53	1.81	0.81	24.3	5.970	5.97
14	19	17		2.53	1.81	0.81	25.2	6.859	6.86

6.2. Allowable Bearing Capacity of Pile

LuccianoDe'Court method will be used for the clayey soil

$$Ql = Qp + Qs$$

where:

Qp $= qp \times Ap$

$= \alpha \times Np \times K \times Ap$

Qs $= qs \times As$

$= \beta \times \left(\frac{Ns}{3} + 1\right) \times As$

There are some diameters pile will be used for column foundation for interior and exterior column. Table 6.2 and Table 6.3shows the allowable bearing capacity of pile with diameter 20cm and 30cm. And there are graphics that illustrate comparison of shear, end bearing capacity and maximum force that can be resisted.

Diameter and end-bearing area of pile:

Type	D (m)	Ap
spunpile	0.2	0.031416
spunpile	0.3	0.070686

Table 6.2 Allowable Bearing Capacity of Pile D-25cm

DEEP	NSPT	N used	Soil Discription	K	Np	qp	Qp	Ns	qs	As	Qs	QL	Qall
0	0	0	CLAY, yellowish grey red stiff	0	0	0	0	0	0	0	0	0	0
0.5	5	5		12	4	48	1.508	5	2.667	0.314	0.838	2.346	0.782
1	7	7		12	8	96	3.016	6	3.000	0.628	1.885	4.901	1.634
1.5	9	9		12	10.25	123	3.864	7.5	3.500	0.942	3.299	7.163	2.388
2	12	12		25	12.000	300	9.425	9.75	4.250	1.257	5.341	14.765	4.922
2.5	13	13		25	12.6	315	9.896	11.375	4.792	1.571	7.527	17.423	5.808
3	14	14	Silty SAND with gravel, greyish brown, medium, dense	25	13.9	347.5	10.917	12.6875	5.229	1.885	9.857	20.774	6.925
3.5	15	15		25	14.375	359.375	11.290	13.84375	5.615	2.199	12.347	23.637	7.879
4	16	15.5		40	14.70148	588.0591	18.474	14.67188	5.891	2.513	14.805	33.279	11.093
4.5	16	15.09434		40	14.70148	588.0591	18.474	14.88311	5.961	2.827	16.854	35.329	11.776
5	16	13.91304		40	14.554	582.1706	18.289	14.39808	5.799	3.142	18.219	36.509	12.170
5.5	17	13.71		40	13.776	551.0344	17.311	14.05388	5.685	3.456	19.645	36.956	12.319
6	17	12.78		40	13.406	536.2399	16.846	13.41792	5.473	3.770	20.631	37.478	12.493
6.5	19	13.38		40	13.463	538.5333	16.919	13.3991	5.466	4.084	22.325	39.244	13.081
7	20	13.25		40	13.5	538.8676	16.929	13.32207	5.441	4.398	23.929	40.858	13.619
7.5	22	13.75		40	13.162	526.4996	16.540	13.53603	5.512	4.712	25.975	42.515	14.172
8	24	14.20		40	12.412	496.4627	15.597	13.86861	5.623	5.027	28.264	43.860	14.620
8.5	20	11.24	Sandy CLAY, brownish yellow grey, very stiff	40	12.203	488.1281	15.335	12.55228	5.184	5.341	27.687	43.022	14.341
9	18	9.63		40	9.811	392.4587	12.329	11.08898	4.696	5.655	26.557	38.887	12.962
9.5	15	7.65		25	8.093	202.3181	6.356	9.371018	4.124	5.969	24.614	30.970	10.323
10	13	6.34		25	6.832	170.802	5.366	7.856241	3.619	6.283	22.737	28.103	9.368
10.5	12	5.61		25	6.412	160.2948	5.036	6.731859	3.244	6.597	21.401	26.437	8.812
11	11	4.93		25	4.985	124.6323	3.915	5.832297	2.944	6.912	20.348	24.264	8.088
11.5	10	4.31		25	4.677	116.925	3.673	5.071321	2.690	7.226	19.440	23.114	7.705
12	9	3.73		25	4.444	111.1095	3.491	4.40288	2.468	7.540	18.605	22.096	7.365
12.5	12	4.80	CLAY, grey, stiff, high plasticity	25	4.844	121.1017	3.805	4.60144	2.534	7.854	19.901	23.705	7.902
13	14	5.405405		25	5.354	133.846	4.205	5.003423	2.668	8.168	21.791	25.996	8.665
13.5	16	5.970149		40	5.354	214.1536	6.728	5.486786	2.829	8.482	23.996	30.724	10.241
14	19	6.859206		40	5.354	214.1536	6.728	6.172996	3.058	8.796	26.897	33.624	11.208

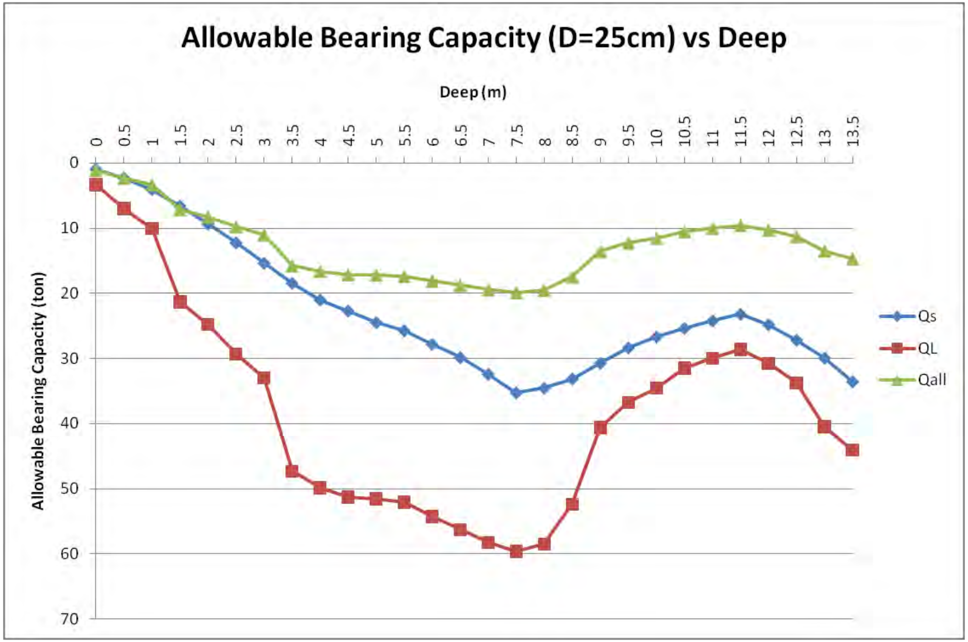
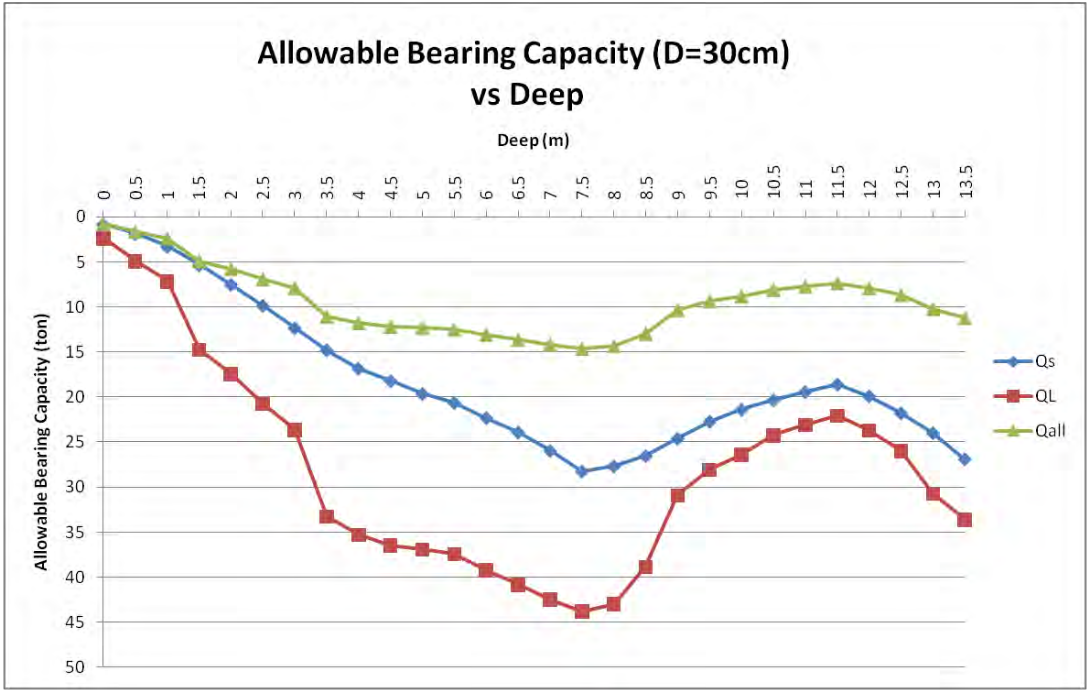


Figure 6.1 Graphic of Allowable Bearing Capacity Pile D-25cm

Table 6.3 Allowable Bearing Capacity of Pile D-30cm

DEEP	NSPT	N used	Soil Discription	K	Np	qp	Qp	Ns	qs	As	Qs	QL	Qall
0	0	0		0	0	0	0	0	0	0	0	0	0
0.5	5	5		12	4	48	3.393	5	2.667	0.314	0.838	4.231	1.410
1	7	7		12	8	96	6.786	7	3.333	0.628	2.094	8.880	2.960
1.5	9	9	CLAY, yellowish grey red stiff	12	10.25	123	8.694	9	4.000	0.942	3.770	12.464	4.155
2	12	12		25	12.000	300	21.206	12	5.000	1.257	6.283	27.489	9.163
2.5	13	13		25	12.6	315	22.266	13	5.333	1.571	8.378	30.644	10.215
3	14	14		25	13.9	347.5	24.563	14	5.667	1.885	10.681	35.245	11.748
3.5	15	15		25	14.375	359.375	25.403	15	6.000	2.199	13.195	38.597	12.866
4	16	15.5		40	14.70148	588.0591	41.567	15.5	6.167	2.513	15.499	57.066	19.022
4.5	16	15.09434		40	14.70148	588.0591	41.567	15.09434	6.031	2.827	17.054	58.621	19.540
5	16	13.91304	Silty SAND with gravel, greyish brown, medium, dense	40	14.554	582.1706	41.151	13.91304	5.638	3.142	17.711	58.863	19.621
5.5	17	13.71		40	13.776	551.0344	38.950	13.70968	5.570	3.456	19.248	58.198	19.399
6	17	12.78		40	13.406	536.2399	37.905	12.78195	5.261	3.770	19.832	57.737	19.246
6.5	19	13.38		40	13.463	538.5333	38.067	13.38028	5.460	4.084	22.299	60.366	20.122
7	20	13.25		40	13.5	538.8676	38.090	13.24503	5.415	4.398	23.816	61.907	20.636
7.5	22	13.75		40	13.162	526.4996	37.216	13.75	5.583	4.712	26.311	63.527	21.176
8	24	14.20		40	12.412	496.4627	35.093	14.20118	5.734	5.027	28.821	63.914	21.305
8.5	20	11.24		40	12.203	488.1281	34.504	11.23596	4.745	5.341	25.343	59.847	19.949
9	18	9.63		40	9.811	392.4587	27.741	9.625668	4.209	5.655	23.799	51.540	17.180
9.5	15	7.65		25	8.093	202.3181	14.301	7.653061	3.551	5.969	21.196	35.497	11.832
10	13	6.34	Sandy CLAY, brownish yellow grey, very stiff	25	6.832	170.802	12.073	6.341463	3.114	6.283	19.565	31.638	10.546
10.5	12	5.61		25	6.412	160.2948	11.331	5.607477	2.869	6.597	18.929	30.259	10.086
11	11	4.93		25	4.985	124.6323	8.810	4.932735	2.644	6.912	18.276	27.085	9.028
11.5	10	4.31		25	4.677	116.925	8.265	4.310345	2.437	7.226	17.607	25.872	8.624
12	9	3.73		25	4.444	111.1095	7.854	3.73444	2.245	7.540	16.925	24.779	8.260
12.5	12	4.80		25	4.282	107.0399	7.566	4.8	2.600	7.854	20.420	27.987	9.329
13	14	5.405405	CLAY, grey, stiff, high plasticity	25	5.354	133.846	9.461	5.405405	2.802	8.168	22.886	32.347	10.782
13.5	16	5.970149		40	5.354	214.1536	15.138	5.970149	2.990	8.482	25.362	40.500	13.500
14	19	6.859206		40	5.354	214.1536	15.138	6.859206	3.286	8.796	28.909	44.046	14.682

Figure 6.2 Graphic of Allowable Bearing Capacity Pile D-30cm



6.3. Stress Distribution of Column

Stress distribution should be analyzed to get part of the floor dead load of precast that will be resisted by pile cap.

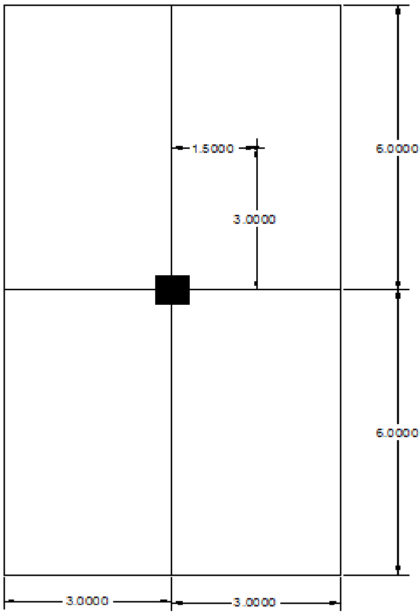


Figure 6.3 Estimation of Stress Distribution from Precast

From the picture, coordinate of stress precast was gotten:

x = 1.5 m; y = 3 m

z = 0.5 m (thickness of pile cap)

$m = \frac{x}{z} = \frac{1.5}{0.5} = 3$; $n = \frac{y}{z} = \frac{3}{0.5} = 6$

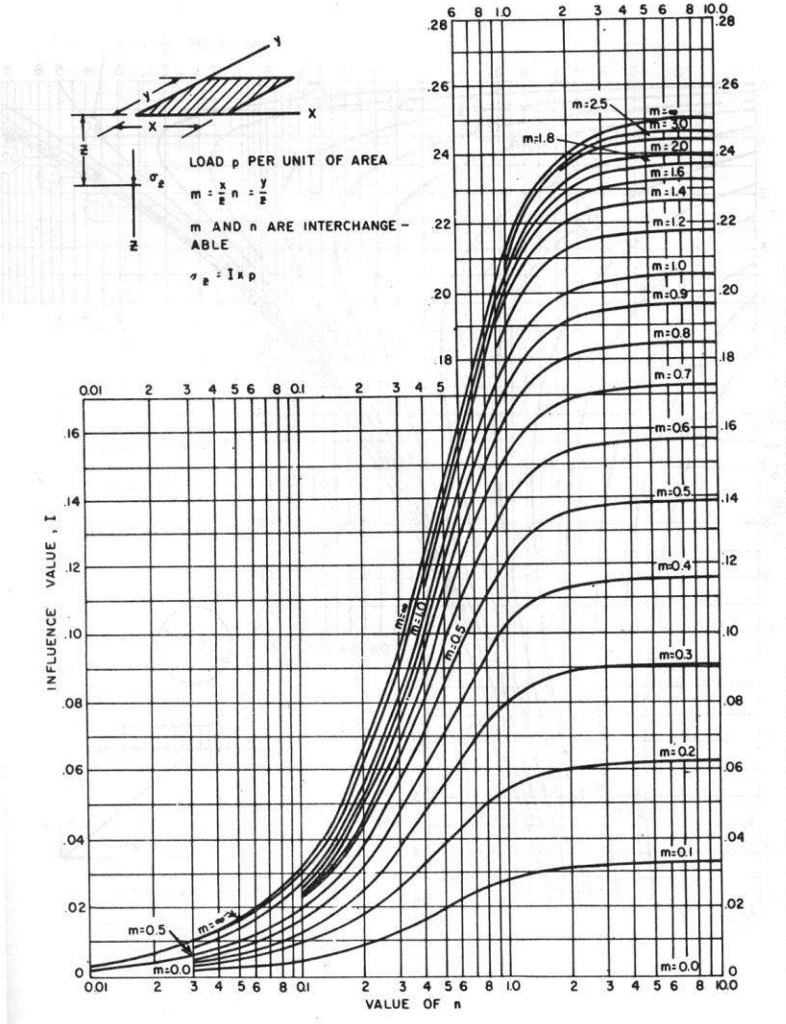


Figure 6.4 Graphic I factor for distributed area load

From the graphic, faktorpengaruh I is gotten = 0.155, because there are 4 precast, so I total = I x 4 = 0.155*4 = 0.62
Finally, the floor dead load could be calculated:

$$\mathrm{FD} = \mathrm{I}_{\mathrm{total}} \times \mathrm{V} \times \gamma_{\mathrm{concrete}}$$
$$\mathrm{FD} = 0.62 \times (6 \times 3 \times 0.25) \times 2400$$
$$\mathrm{FD} = 6696\ \mathrm{kN} = 6.696\ \mathrm{ton}$$

6.4. Load and Load Combination

Analyzing of load and load combination are differences by two types of column, interior and exterior column. The columns are steel structure with the internal forces that had been calculated by consultant. The foundation, include pile cap and pile, will be calculated without approximating the strength of steel, steel strength is supposed strong enough to resist forces without failed.

There are some load combinations according to SNI, but the factor won't be used to calculate the pile:

1. D
2. D+L+R
3. D+R+W
4. D+W+L+R
5. D+E+R
6. D+W
7. D+E

6.4.1. Interior Column

Theinterior column that will be analyzed is column in grid E-10 as shown in Table 6.4.

Table 6.4 Output Forces of Interior Column

Type		Interior Column	
X-Loc		96435	
Grid1 - Grid2		E-10	
Base Plate W x L (mm)		330 X 330	
Base Plate Thickness (mm)		20	
Anchor Rod Qty/Diam. (mm)		4 - 24.0	
Column Base Elev.		-425	
Load Type	Desc.	Hx	Vy
D	Frm	0.55	65.05
FD	Frm	-0.13	-0.22
CG	Frm	1.12	73.18
W1>	Frm	-4.54	-43.22
<W1	Frm	6.15	-22.15
W2>	Frm	-0.83	-37.98
<W2	Frm	-0.83	-37.98
CU	Frm	-	-
R	Frm	-0.18	53
L	Frm	-0.18	-0.31
WP	Frm	-	-
WB1>	Brc	-	-0.09
<WB1	Brc	-	-
WB2>	Brc	-	-0.14
<WB2	Brc	-	-
E>	Frm	-25.8	-37.64
EG+	Frm	-	23.78
<E	Frm	25.73	37.52
EG-	Frm	-	-23.78
EB>	Brc	-0.16	-0.1
<EB	Brc	0.09	-0.14

Not all loads will be used from Table 6.4. Some loading (with yellow line) will be needed for calculating, because not all forces will happen at the same time.

Table 6.5 The Used Loads for Design Foundation of Interior Column

Loading	V		Hx	
	kN	ton	kN	ton
Dead Load (D)	65.05	6.505	7.37	0.737
Floor Dead Load (FD)	66.96	6.696		
Live Load (L)	0.31	0.031	0.18	0.018
Rain Load (R)	53	5.3	0.18	0.018
Wind Load (W)	0.14	0.014		
Earthquake Load (E)	37.64	3.764	25.8	2.58

Table 6.6(a) Load Combination 1

Combination 1 = D

LOAD	FACTOR	FORCES (ton)		DISTANCE	MOMENT
		V	Hx	Y	Mx
Dead Load	1	6.505	0.737	0.5	0.3685
Floor Dead Load (FD)	1	6.696			
Total		13.201			0.3685

Table 6.6(b) Load Combination 2

Combination 2 = D+L+R

LOAD	FACTOR	FORCES (ton)		DISTANCE	MOMENT
		V	Hx	Y	Mx
Dead Load	1	6.505	0.737	0.5	0.3685
Floor Dead Load (FD)	1	6.696			
Live Load	1	0.031			
Rain Load	1	5.3	0.018	0.5	0.009
Total		18.532	0.755		0.009

Table 6.6(c) Load Combination 3

Combination 3 = D+R+W

LOAD	FACTOR	FORCES (ton)		DISTANCE	MOMENT
		V	Hx	Y	Mx
Dead Load	1	6.505	0.737	0.5	0.3685
Floor Dead Load (FD)	1	6.696			
Rain Load	1	5.3	0.018	0.5	0.009
Wind Load	1	0.014			
Total		18.515	0.755		0.3775

Table 6.6(d) Load Combination 4

Combination 4 = D+W+L+R

LOAD	FACTOR	FORCES (ton)		DISTANCE	MOMENT
		V	Hx	Y	Mx
Dead Load	1	6.505	0.737	0.5	0.3685
Floor Dead Load	1	6.696			
Wind Load	1	0.014			
Live Load	1	0.031			
Rain Load	1	5.3	0.018	0.5	0.009
Total		18.546	0.755		0.3775

Table 6.6(e) Load Combination 5

Combination 5 = D+E+L

LOAD	FACTOR	FORCES (ton)		DISTANCE	MOMENT
		V	Hx	Y	Mx
Dead Load	1	6.505	0.737	0.5	0.3685
Floor Dead Load (FD)	1	6.696			
Earthquake	1	3.764	2.58	0.5	1.29
Live Load	1	0.031			
Total		16.996	3.317		1.6585

Table 6.6(f) Load Combination 6

Combination 6 = D+W

LOAD	FACTOR	FORCES (ton)		DISTANCE	MOMENT
		V	Hx	Y	Mx
Dead Load	1	6.505	0.737	0.5	0.3685
Floor Dead Load (FD)	1	6.696			
Wind Load	1	0.014			
Total		13.215	0.737		0.3685

Table 6.6(g) Load Combination 7

Combination 7 = D+E

LOAD	FACTOR	FORCES (ton)		DISTANCE	MOMENT
		V	Hx	Y	Mx
Dead Load	1	6.505	0.737	0.5	0.3685
Floor Dead Load (FD)	1	6.696			
Earthquake	1	3.764	2.58	0.5	1.29
Total		16.965	3.317		1.6585

6.4.2. Exterior Column

The exterior column that will be analyzed is column in grid E-10 as shown in Table 6.7.

Table 6.7Output Forces of Exterior Column

Type		Exterior Column		
X-Loc		0		
Grid1 - Grid2		L-2		
Base Plate W x L (mm)		280 X 431		
Base Plate Thickness (mm)		20		
Anchor Rod Qty/Diam. (mm)		4 - 24.0		
Column Base Elev.		-425		
Load Type	Desc.	Hx	Hz	Vy
D	Frm	5.82	-	22.03
FD	Frm	-	-	-
CG	Frm	26.65	-	80.3
W1>	Frm	-10.5	-	-9.61
<W1	Frm	3.08	-	-5.65
W2>	Frm	-0.89	-	-8.68
<W2	Frm	-0.89	-	-8.68
CU	Frm	-	-	-
R	Frm	4.22	-	12.49
L	Frm	-	-	-
WP	Frm	-	-	-
WB1>	Brc	-0.2	-	18.41
<WB1	Brc	0.3	8.64	-18.05
WB2>	Brc	-0.32	-	29.09
<WB2	Brc	0.51	14.41	-30.14
E>	Frm	-38.24	-	-26.44
EG+	Frm	5.19	-	16.16
<E	Frm	38.11	-	26.42
EG-	Frm	-5.19	-	-16.16
EB>	Brc	-1.79	-0.13	164.2
<EB	Brc	2.72	77.31	-161.08

It’s same as the previous loading. Some loading (with yellow line) will be needed for calculating, because not all forces will happen at the same time as shown in Table 6.8. The load combination will be shown in Table 6.9(a) till Table 6.9(g).

Table 6.8 The Used Loads for Design Foundation of Exterior Column

Loading	V		Hx		Hz	
	kN	ton	kN	ton	kN	ton
Dead Load (D)	22.03	2.203	5.82	0.582		
Floor Dead Load (FD)	66.96	6.696				
Live Load (L)	0	0				
Rain Load (R)	12.49	1.249	12.49	1.249		
Wind Load (W)	30.14	3.014	0.51	0.051	14.41	1.441
Earthquake Load (E)	161.08	16.108	2.72	0.272	77.31	7.731

Table 6.9(a) Load Combination 1

Combination 1 = D

LOAD	FACTOR	FORCES (ton)			DISTANCE	MOMENT
		V	Hx	Hz	Y	Mx
Dead Load	1	2.203	0.582		0.5	0.291
Floor Dead Load	1	6.696				
Total		8.899				0.291

Table 6.9(b) Load Combination 2

Combination 2 = D+L+R

LOAD	FACTOR	FORCES (ton)			DISTANCE	MOMENT
		V	Hx	Hz	Y	Mx
Dead Load	1	2.203	0.582		0.5	0.291
Floor Dead Load	1	6.696				
Live Load	1	0				
Rain Load	1	1.249	1.249		0.5	0.6245
Total		10.148	1.831			0.6245

Table 6.9(c) Load Combination 3

Combination 3 = D+R+W

LOAD	FACTOR	FORCES (ton)			DISTANCE	MOMENT
		V	Hx	Hz	Y	Mx
Dead Load	1	2.203	0.582		0.5	0.291
Floor Dead Load	1	6.696				
Rain Load	1	1.249	1.249		0.5	0.6245
Wind Load	1	3.014	0.051	1.441	0.5	0.746
Total		13.162	1.882	1.441		1.6615

Table 6.9(d) Load Combination 4

Combination 4 = D+W+L+R

LOAD	FACTOR	FORCES (ton)			DISTANCE	MOMENT
		V	Hx	Hz	Y	Mx
Dead Load	1	2.203	0.582		0.5	0.291
Floor Dead Load	1	6.696				
Wind Load	1	3.014	0.051	1.441	0.5	0.746
Live Load	1	0				
Rain Load	1	1.249	1.249		0.5	0.6245
Total		13.162	1.882	1.441		1.6615

Table 6.9(e) Load Combination 5

Combination 5 = D+E+L

LOAD	FACTOR	FORCES (ton)			DISTANCE	MOMENT
		V	Hx	Hz	Y	Mx
Dead Load	1	2.203	0.582		0.5	0.291
Floor Dead Load	1	6.696				
Earthquake	1	16.108	0.272	1.441	0.5	0.8565
Live Load	1	0				
Total		25.007	0.854	1.441		1.1475

Table 6.9(f) Load Combination 6

Combination 6 = D+W

LOAD	FACTOR	FORCES (ton)			DISTANCE	MOMENT
		V	Hx	Hz	Y	Mx
Dead Load	1	2.203	0.582		0.5	0.291
Floor Dead Load	1	6.696				
Wind Load	1	3.014	0.051	1.441	0.5	0.746
Total		11.913	0.633	1.441		1.037

Table 6.9(g) Load Combination 7

Combination 7 = D+E

LOAD	FACTOR	FORCES (ton)			DISTANCE	MOMENT
		V	Hx	Hz	Y	Mx
Dead Load	1	2.203	0.582		0.5	0.291
Floor Dead Load	1	6.696				
Earthquake	1	16.108	0.272	7.731	0.5	4.0015
Total		25.007	0.854	7.731		4.2925

6.5 Pile Analysis

Pile analysis differences by interior and exterior column. The design of both pile are different based on the loading as seen in Figure 6.5 and Figure 6.6. The comparison between the real load and the design are checked in Table 6.10 and Table 6.11.

6.5.1 Interior Column

For the interior column, pile with diameter, d = 25cm will be used.

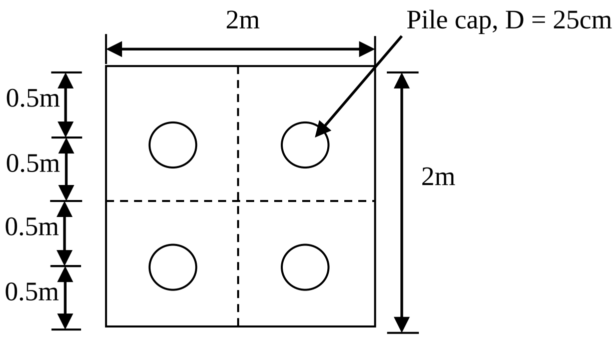


Figure 6.5 Pile Cap for Interior Column

pile cap	= 2m x 2m
t pile cap	= 50 cm
type pile	= spun pile
d pile	= 25cm

a. Load of one pile:

$$P_{max} = \frac{V}{n} + \frac{M_x \times Y_{max}}{\Sigma Y_2} + \frac{M_y \times X_{max}}{\Sigma X_2}$$

Where:

P_{max}	= Maximum load for one pile
Σ P	= Total axial load occurred
Mx	= Moment in X direction
My	= Moment in Y direction
Xmax	= 0.5 m
Ymax	= 0.5 m

$\sum X^2$ = $4 \times 0.5^2 = 1\ m^2$
$\sum Y^2$ = $4 \times 0.5^2 = 1\ m^2$
n = total of pile = 4

b. Efficiency:

Efficiency of pile considered as the previous chapter based on the good soil condition.

$\eta = 0.9$

Q allowable =
Q allowable pile x ηx 0.6 (static factor for static pile)

Checking:
Q allowable > P max

Table 6.10 Checking of P max and Q allowable for Interior Column

COMBO	FORCES (ton)			MOMENT		ΣP/n	(Mx x Ymax) / ΣY2	(My x Xmax) / ΣX2	Pmax (ton)	Q allowable (D-25cm)
	V	Hz	Hy	Mx	My					
Combination 1	8.899	0	0	0.291	0	2.22	0	0	2.370	7.944
Combination 2	18.532	0.755	0	0.009	0	4.63	0.005	0	4.638	7.944
Combination 3	18.515	0.755	0	0.378	0	4.63	0.189	0	4.818	7.944
Combination 4	18.546	0.755	0	0.378	0	4.64	0.189	0	4.825	7.944
Combination 5	16.996	3.317	0	1.659	0	4.25	0.829	0	5.078	7.944
Combination 6	13.215	0.737	0	0.369	0	3.30	0.184	0	3.488	7.944
Combination 7	16.965	3.317	0	1.659	0	4.24	0.829	0	5.071	7.944

c. Punching Shear

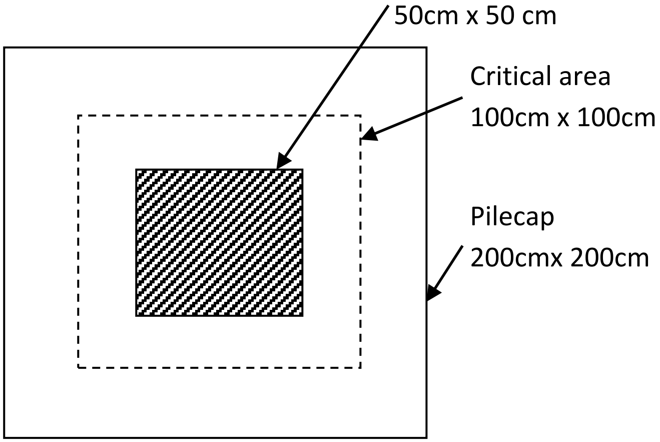
1. Two way slab punching shear

- As consequences of **column**:

  Steel column = WF 330x330

  Pedestal column = 500x500

  Pile cap = 2m x 2m



Shear ultimate:

V_u = V x SF

= 18.546 ton x 1.5

= 27.819 ton

= 27,819 kg

Permissible shear:

(based on SNI 2847:2013 chap. 11.11.2.1)

$V_c = 0.17\left(1 + \frac{2}{\beta}\right) \lambda (\sqrt{f'c}) b_o d$

or

$V_c = 0.33\lambda\sqrt{f'_c} b_o\ d$

where:

$\beta = \frac{Lx}{Ly} = \frac{2}{2} = 1$

λ = 1 (for normal weight concrete)

b_o= 4.s = 4 x 100 = 400cm^2

d = 80cm

V_c = 0.17(1 + $\frac{2}{1}$) ($\sqrt{3000}$400 . 80

= 893,883 kg

V_c = $0.33\sqrt{3000}\ 400\ .80$

= 583,653 kg

Shear forces requirements

ϕVc> Vu

0.75 (583,653) > 50,550

8,795,948kg > 50,550kg (OK

- As consequences of **pile**:

Shear ultimate:

V_u = Q_{all} x SF

= 11.208 ton x 1.5

= 16.812 ton

= 16,812 kg

Permissible shear:

(based on SNI 2847:2013 chap. 11.11.2.1)

V_c = 0.17(1 + $\frac{2}{\beta}$) λ ($\sqrt{f'c}$)b_od

or

V_c = $0.33\lambda\sqrt{f'_c}b_o\ d$

where,

$\beta = \frac{Lx}{Ly} = \frac{2}{2} = 1$

λ = 1 (for normal weight concrete)

$b_o = \pi D = \pi . 50 = 157.08cm^2$

d = 80cm

V_c $= 0.17(1 + \frac{2}{1}) (\sqrt{3000})\ 157.08 \ .\ 80$

$= 351{,}027$ kg

V_c $= 0.33\sqrt{3000}\ 157.08 \ .\ 80$

$= 229.200$ kg

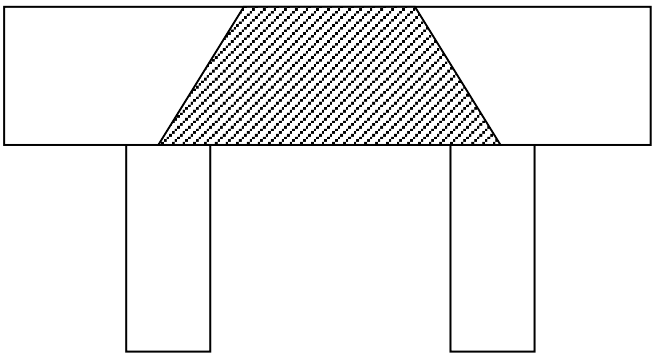
Shear forces requirements

ϕVc> Vu

0.75 (229.200) >16,812 kg

171,900kg >16,812 kg (OK)

2. One way slab punching shear:



Pile location is out of critical area of punching shear, so it should be check: (based on SNI 2847:2013 chap 11.11.2.1)

$$V_c = 0.083\left(\frac{\alpha_s.d}{bo}+2\right)\lambda\sqrt{f'c}\,.bo\,.d$$

where

α_s = 40 (for interior column)

bo = 50+100+2(83.82) = 317.63

$$V_c = 0.083\left(\frac{40.80}{317.63}+2\right)\sqrt{3000}\,.317.63\,.80$$

= 1,394.837 kg

Shear forces requirements

ϕVc> Vu

0.75 (1,394.837) > 16,812 kg

1,046,127 kg > 16,812 kg (OK)

6.5.2 Exterior Column

For the exterior column, pile with diameter, d = 30cm will be used.

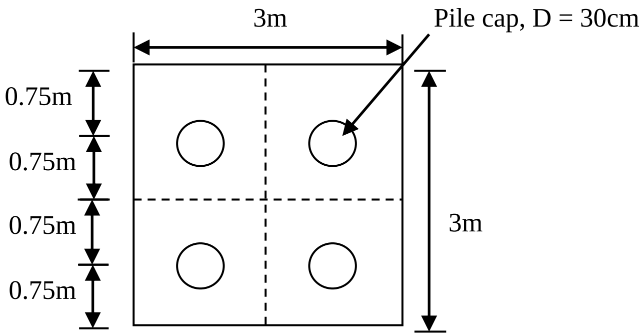


Figure 6.6 Pile Cap for Exterior Column

pile cap	= 3m x 3m
t pile cap	= 50 cm
tupe pile	= spun pile
d pile	= 30cm

a. Load of one pile:

$$P_{max} = \frac{V}{n} + \frac{M_x \times Y_{max}}{\sum Y_2} + \frac{M_y \times X_{max}}{\sum X_2}$$

Where:

P_{max}	= Maximum load for one pile
Σ P	= Total axial load occurred
Mx	= Moment in X direction
My	= Moment in Y direction
Xmax	= 0.75 m
Ymax	= 0.75 m
$\sum X^2$	= 4 x 0.75^2 = 2.25 m^2
$\sum Y^2$	= 4 x 0.75^2 = 2.25 m^2
n	= total of pile = 4

b. Efficiency:

Efficiency of pile considered as the previous chapter based on the good soil condition.

$\eta \quad = 0.9$

Q allowable =

Q allowable pile x ηx 0.6 (static factor for static pile)

Checking:

Q allowable > P max

Table 6.11 Checking of P max and Q allowable for Exterior Column

COMBO	FORCES (ton)			MOMENT		ΣP/n	(Mx x Ymax) / ΣY2	(My x Xmax) / ΣX2	Pmax (ton)	Q allowable (D-30cm)
	V	Hz	Hz	Mx	My					
Combination 1	8.899	0	0	0.291	0	2.2248	0	0	2.3218	9.2292
Combination 2	10.148	1.831	0	0.625	0	2.5370	0.208	0	2.7452	9.2292
Combination 3	13.162	1.882	1.441	1.662	0	3.2905	0.554	0	3.8443	9.2292
Combination 4	13.162	1.882	1.441	1.662	0	3.2905	0.554	0	3.8443	9.2292
Combination 5	25.007	0.854	1.441	1.148	0	6.2518	0.383	0	6.6343	9.2292
Combination 6	11.913	0.633	1.441	1.037	0	2.9783	0.346	0	3.3239	9.2292
Combination 7	25.007	0.854	7.731	4.293	0	6.2518	1.431	0	7.6826	9.2292

6.6 Control

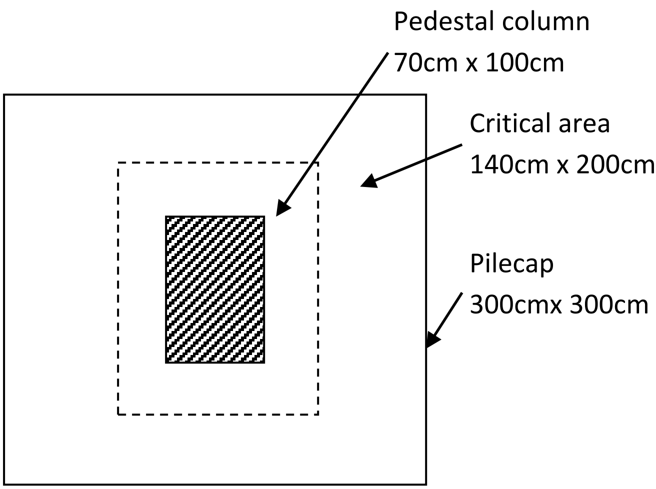
Punching Shear

1. Two way slab punching shear
   - As consequences of **column**:

     Steel column = WF 280x431

     Pedestal column = 70cmx100cm

     Pile cap = 3m x 3m



Shear ultimate:

V_u = V x SF

= 25.007 ton x 1.5

= 37.511 ton

= 37,511 kg

Permissible shear:

(based on SNI 2847:2013 chap. 11.11.2.1)

$V_c = 0.17\left(1 + \frac{2}{\beta}\right) \lambda \left(\sqrt{f'c}\right) b_o d$

or

$V_c = 0.33\lambda\sqrt{f'_c}b_o\ d$

where,

$\beta = \frac{Lx}{Ly} = \frac{3}{3} = 1$

$\lambda = 1$ (for normal weight concrete)

$b_o = 2(s1+s2) = 2(200+140) = 680cm^2$

$d = 80cm$

$V_c = 0.17(1 + \frac{2}{1}) (\sqrt{3000} .680 . 80$

$= 1,519,601$ kg

$V_c = 0.33\sqrt{3000}\ 680.\ 80$

$= 992,210kg$

Shear forces requirements

$\phi Vc > Vu$

0.75 (992,210) > 37,511

744,158 kg> 37,511kg (OK)

- As consequences of **pile**:

Shear ultimate:

$V_u = Q_{all}$ x SF

= 14.682 ton x 1.5

= 22.203 ton

= 22,203 kg

Permissible shear:

(based on SNI 2847:2013 chap. 11.11.2.1)

$V_c \quad = 0.17(\,1 + \frac{2}{\beta}\,)\,\lambda\,(\sqrt{f'c})b_o d$

or

$V_c \quad = 0.33\lambda\sqrt{f'_c}b_o\, d$

where,

$\beta = \frac{Lx}{Ly} = \frac{3}{3} = 1$

$\lambda = 1$ (for normal weight concrete)

$b_o = \pi D = \pi.\,60 = 188.5cm^2$

$d = 80cm$

$V_c \quad = 0.17(\,1 + \frac{2}{1}\,)\,(\sqrt{3000})\; 188.5\; .\; 80$

$\quad\quad = 421.33$ kg

$V_c \quad = 0.33\sqrt{3000}\;.188.5\;.\,80$

$\quad\quad = 275{,}040$ kg

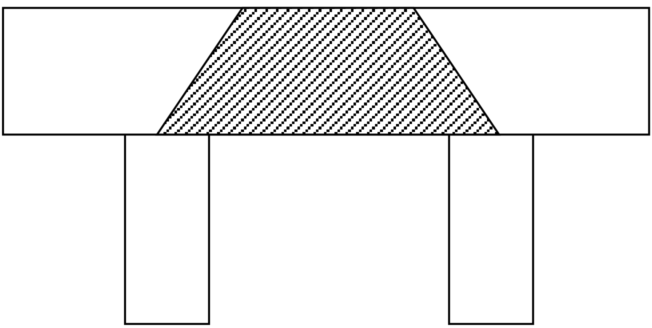
Shear forces requirements

$\phi Vc > Vu$

0.75 (275,040) >22.023kg

206.280kg >22.023kg (OK)

2. One way slab punching shear:



Pile location is out of critical area of punching shear, so it should be check: (based on SNI 2847:2013 chap 11.11.2.1)

V_c $= 0.083 \left(\frac{\alpha_s.d}{bo} + 2\right) \lambda\sqrt{f'c} \,.\, bo \,.\, d$

where

α_s = 20 (for corner column)

bo = 70+140+2(87.32) = 384.64

V_c $= 0.083 \left(\frac{20.80}{384.64} + 2\right) \sqrt{3000}$ .384.64 .80

= 861,678 kg

Shear forces requirements

ϕVc> Vu

0.75 (861,678) > 22.023kg

646,258 kg > 22.023kg

6.7 Pile-Cap Reinforcement

Pile cap will be reinforced in two direction, x and y. The critical moment

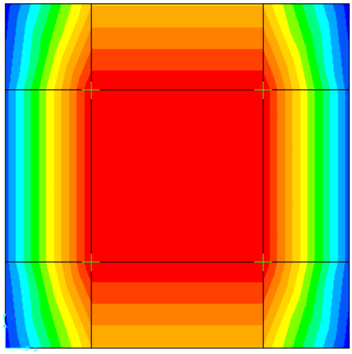
5.5.2 Interior Column Pilecap

Concrete strength, f'c	= 30 MPa
Yield strength, fy	= 420 MPa
Slab thickness, hf	= 80cm
Decking concrete, d	= 50mm

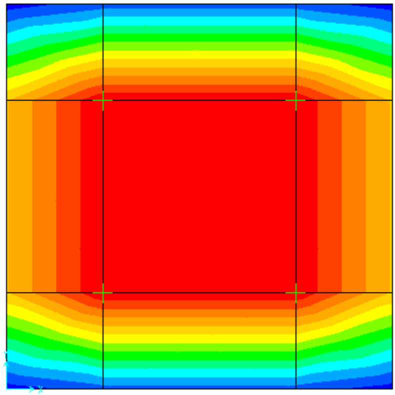
(based on SNI 2847:2013 chap.7.7.3, d = ± 20 mm)

Reinf.diameter, D	= 20 mm
Lx	= 2 m
Ly	= 2 m
dx	= hf – d-(1/2D)
	= 800 –50-(1/2.20)
	= 725 mm
dy	= hf – d-(3/2D)
	= 800 – 50 –(3/2.20)
	= 660 mm

5.5.3Stress Occurred



M_{11} for reinforcement Direction



M_{22} for reinforcement Direction

Figure 6.7 Stress Occurred in Interior Column

Table 6.12 Element Forces in Pile Cap

Combination	Moment (kgm)		Shear (kg)	
	M11	M22	V13	V23
Envelope	-1558.31	-1558.31	2710.78	2710.78
Envelope	-1170.52	-317.87	2699.66	2710.78
Envelope	-317.87	-1170.52	2710.78	2699.66
Envelope	64.35	64.35	2699.66	2699.66

4.9.3 Reinforcement Needed Calculation

Because of the symmetric design, reinforcement in x and y direction will have the same reinforcement needed. So it will be differences by negative and positive moment.

$As_{\phi} = \frac{1}{4} \times \pi \times D^2 = \frac{1}{4} \times \pi \times 20^2 = 0.03142 \text{mm}^2$

(based on SNI 2857:2013 chap. 7.12.2.1)
$\rho_{shrinkage}$ = 0.0018 (for slab)

(based on SNI 2857:2013 chap. 7.12.2.1)
reduction factor of reinforcement, ϕ = 0.9

4. 9.3.1 Reinforcement for X Direction

a) Positive Moment

Mu = 64.35kgm

$Mn = \frac{Mu}{\phi} = \frac{64.35}{0.9} = 71.5 \text{ kgm}$

$Rn = \frac{Mn}{1m \times dx^2} = \frac{71.5}{1m \times 0.7^2} = 145.918 \text{ kg/m}^2$

$= 0.00145 \text{ N/mm}^2$

ρ_{perlu} $= \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 50}{350}(1 - \sqrt{1 - \frac{2 \times 0.00145}{0.85 \times 30}})$

$= 4.169 \times 10^{-5}$ (use ρ_{min})

As_{need} $= \rho_{shrinkage} \times 1m \times dx$

$= 0.0018 \times 1m \times 0.7m$

$= 1.26 \times 10^{-3}\ m^2$

$= 1260\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{1260}{314.2} = 4.011$ (use 5 reinf.)

s $= \frac{1m}{n} = 200mm$

useD20-200mm

b) Negative Moment

Mu $= 1{,}558.31$ kgm

Mn $= \frac{Mu}{\phi} = \frac{= 1{,}558.31}{0.9} = 1.731 \times 10^3$ kgm

Rn $= \frac{Mn}{1m \times dx^2} = \frac{1.731 \times 10^3}{1m \times 0.7^2} = 3.534 \times 10^3\ kg/m^2$

$= 0.03534\ N/mm^2$

ρ_{perlu} $= \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 50}{350}(1 - \sqrt{1 - \frac{2 \times 0.03534}{0.85 \times 30}})$

$= 1.016 \times 10^{-3}$ (use $\rho_{shrinkage}$)

As_{need} $= \rho_{shrinkage}$ x 1m x dx
= 0.0018 x 1m x 0.7m
$= 1.26 \times 10^{-3}\ m^2$
$= 1260\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{1260}{314.2} = 4.011$ (use 5 reinf.)

s $= \frac{1m}{n} = 200mm$

useD20-200mm

4. 9.3.2 Reinforcement for Y direction

a) Positive Moment

Mu = 64.35 kgm

Mn $= \frac{Mu}{\phi} = \frac{64.35}{0.9} = 71.5$ kgm

Rn $= \frac{Mn}{1m \times dy^2} = \frac{71.5}{1m \times 0.66^2} = 164.141\ kg/m^2$
$= 0.0164\ N/mm^2$

ρ_{perlu} $= \frac{0.85 \times fc}{fy}\left(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}}\right)$

$= \frac{0.85 \times 50}{350}\left(1 - \sqrt{1 - \frac{2 \times 0.0164}{0.85 \times 30}}\right)$

$= 4.664 \times 10^{-5}$ (use ρ_{min})

As_{need} $= \rho_{shrinkaget}$ x 1m x dy
= 0.0018 x 1m x 0.66m
$= 1.12 \times 10^{-3}\ m^2$
$= 1120\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{1120}{314.2} = 3.782$ (use 4 reinf.)

s $= \frac{1m}{n} = 250mm$

useD20 - 250mm

b) Negative Moment

Mu = 1,558.31 kgm

Mn $= \frac{Mu}{\phi} = \frac{= 1,558.31}{0.9} = 1.731 \times 10^3$ kgm

Rn $= \frac{Mn}{1m \times dy^2} = \frac{1.731 \times 10^3}{1m \times 0.66^2} = 3.975 \times 10^3 kg/m^2$

$= 0.03975\ N/mm^2$

$\rho_{perlu} = \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 50}{350}(1 - \sqrt{1 - \frac{2 \times 0.03975}{0.85 \times 30}})$

$= 1.142 \times 10^{-3}$ (use ρ_{min})

As_{need} $= \rho_{shrinkage} \times 1m \times dy$

$= 0.0018 \times 1m \times 0.66m$

$= 1.12 \times 10^{-3}\ m^2$

$= 1120\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{1120}{314.2} = 3.782$ (use 4 reinf.)

s $= \frac{1m}{n} = 250mm$

useD20 - 250mm

5.5.2 Exterior Column Pilecap

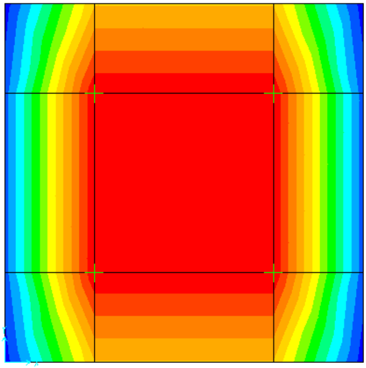
Concrete strength, f'c = 30 MPa
Yield strength, fy = 420 MPa
Slab thickness, hf = 80cm
Decking concrete, d = 50mm
(based on SNI 2847:2013 chap.7.7.3, d = ± 20 mm)

Reinf.diameter, D = 20 mm
Lx = 3 m
Ly = 3 m
dx = hf – 2d

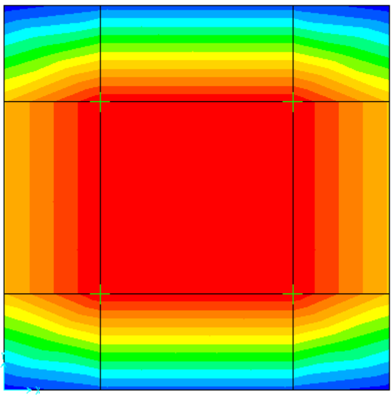
$= 800 - 2.50$
$= 800mm$

dy $= hf – 2d –2D$
$= 250 – 2.50 – 2.20$
$= 660mm$

5.5.3 Stress Occurred



M_{11} for reinforcement Direction



M_{22} for reinforcement Direction

Figure 6.8Stress Occurred in Exterior Column

Table 6.13 Element Forces in Exterior Pile Cap

Combination	Moment (kgm)		Shear (kg)	
	M11	M22	V13	V23
Envelope	-3544.55	-3544.55	4149.67	4149.67
Envelope	-2661.5	-688.6	4131.71	4149.67
Envelope	-688.6	-2661.5	4149.67	4131.71
Envelope	180.98	180.98	4131.71	4131.71

4.9.3 Reinforcement Needed Calculation

Because of the symmetric design, reinforcement in x and y direction will have the same reinforcement needed. So it will be differences by negative and positive moment.

$As_{\phi} = \frac{1}{4} \times \pi \times D^2 = \frac{1}{4} \times \pi \times 20^2 = 0.03142 mm^2$

(based on SNI 2857:2013 chap. 7.12.2.1)

$\rho_{shrinkage} = 0.0018$

(based on SNI 2857:2013 chap. 7.12.2.1)

reduction factor of reinforcement, ϕ = 0.9

4. 9.3.1 Reinforcement for X Direction

a) Positive Moment

$Mu = 180.98 kgm$

$Mn = \frac{Mu}{\phi} = \frac{190.98}{0.9} = 201.089 \text{ kgm}$

$Rn = \frac{Mn}{1m \times dx^2} = 201.089 = 410.385 \text{ kg/m}^2$

$= 0.041 \text{ N/mm}^2$

$\rho_{perlu} = \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 30}{350}(1 - \sqrt{1 - \frac{2 \times 0.041}{0.85 \times 30}})$

= 1.174 x 10^{-4} (use $\rho_{shrinkage}$)

As_{need} = $\rho_{shrinkage}$ x 1m x dx

= 0.0018 x 1m x 0.7m

= 1.26 x 10^{-3} m^2

= 1260 mm^2

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{1260}{314.2} = 4.011$ (use 5reinf.)

s $= \frac{1m}{n} = 200mm$

useD20-200mm

b) Negative Moment

Mu = 3,544.55kgm

Mn $= \frac{Mu}{\phi} = \frac{=3{,}544.55}{0.9} = 3.938 \times 10^3$ kgm

Rn $= \frac{Mn}{1m \times dx^2} = \frac{3.938 \times 10^3}{1m \times 0.7^2} = 8.038 \times 10^3$ kg/m^2

= 0.08038 N/mm^2

ρ_{perlu} $= \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 30}{350}(1 - \sqrt{1 - \frac{2 \times 0.08038}{0.85 \times 30}})$

= 2.989 x 10^{-3} (use ρ_{min})

As_{need} = $\rho_{shrinkage}$ x 1m x dx

= 0.0018 x 1m x 0.7m

= 1.26 x 10^{-3} m^2

= 1260 mm^2

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{1260}{314.2} = 4.011$ (use 5reinf.)

s $= \frac{1\text{m}}{\text{n}} = 200\text{mm}$

useD20-200mm

4. 9.3.2 Reinforcement for Y direction

a) Positive Moment

Mu = 180.98 kgm

Mn $= \frac{\text{Mu}}{\phi} = \frac{190.98}{0.9} = 201.089$ kgm

Rn $= \frac{\text{Mn}}{1\text{m}\times\text{dy}^2} = 201.089 = 461.637\ \text{kg/m}^2$

= 0.0461 N/mm^2

ρ_{perlu} $= \frac{0.85\times\text{fc}}{\text{fy}}(1 - \sqrt{1 - \frac{2Rn}{0.85\times fc}})$

$= \frac{0.85\times30}{350}(1 - \sqrt{1 - \frac{2\times0.0461}{0.85\times30}})$

= 1.313 x 10^{-4} (use $\rho_{shrinakge}$)

As_{need} = $\rho_{shrinkage}$ x 1m x dy

= 0.0018 x 1m x 0.66m

= 1.2 x 10^3 m^2

= 1200 mm^2

Total reinforcement:

n $= \frac{\text{As}_{\text{need}}}{\text{As}_{\phi}} = \frac{1200}{314.2} = 3.782$ (use 4reinf.)

s $= \frac{1\text{m}}{\text{n}} = 250\text{mm}$

useD20-250mm

b) Negative Moment

Mu = 3,544.55 kgm

Mn $= \frac{\text{Mu}}{\phi} = \frac{=3{,}544.55}{0.9} = 3.938\times10^3$ kgm

Rn $= \frac{\text{Mn}}{1\text{m}\times\text{dy}^2} = \frac{3.938\times10^3}{1\text{m}\times0.66^2} = 9.041\times10^3\ \text{kg/m}^2$

= 0.09041 N/mm^2

ρ_{perlu} $= \frac{0.85 \times fc}{fy}(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc}})$

$= \frac{0.85 \times 30}{350}(1 - \sqrt{1 - \frac{2 \times 0.09041}{0.85 \times 30}})$

$= 2.989 \times 10^{-3}$ (use $\rho_{shrinkage}$)

As_{need} $= \rho_{shrinkage} \times 1m \times dy$

$= 0.0018 \times 1m \times 0.66m$

$= 1.2 \times 10^{3}\ m^2$

$= 1200\ mm^2$

Total reinforcement:

n $= \frac{As_{need}}{As_{\phi}} = \frac{1200}{314.2} = 3.782$ (use 4 reinf.)

s $= \frac{1m}{n} = 250mm$

use D20-250mm

CHAPTER 7
CONCLUSION

7.1. Conclusion

Conclusion from the analysis and calculating of this final project are:

1. Precast prestress slab on ground
   - Dimension : 3m x 6m
   - Thickness : 25 cm
   - Reinforcement : tendon, mild-steel reinf.
   - Tendon : Freyssinet, Type F, 5 strands
   - Mild-reinforcement : D12-250mm
2. Machine foundation
   - Dimension : 12m x 24m
   - Thickness : 1m
   - Reinforcement : D25-250mm
3. Interior column pilecap
   - Dimension : 2m x 2m
   - Thickness : 80cm
   - Reinforcement : D20-200mm
4. Exterior column pilecap
   - Dimension : 3m x 3m
   - Thickness : 80cm
   - Reinforcement : D20-200mm

7.2. Suggestion

Furthermore learning of upper structure will be needed to analyze the exact real condition, hence, the calculating of foundation could be more detailed.

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WRITER'S BIODATA

The writer was born in Jakarta, December 17^{th} 1993. She is the second daughter with an older brother and a twin. She studied in Mardi Yuana Elementary and Junior High School then continued in SMAN 1 Serang in 2009 and graduated in 2011. She attended University in Surabaya, Institut Teknologi Sepuluh Nopember, and took Bachelor Degree Program in Civil Engineering.

Writer was also active in some organizations and community in campus and out of the campus. For example, a social organization, Civillage, that build a library in a remote area as material and human resources engineer, took a role in Christian community in university, and joint some volunteering works in International Church. She had also joined some internship in WIKA Building in 2013 and PT. Teamworx Indonesia as structural engineer in 2014.

The writer is really interested in structural engineering and property business. She hope with this final project she can reach her dream to get a Bachelor Degree title and continue her study in Master Degree.