CERUCUK
Volume 6 No. 5 2022 (381-395)

UPPER AND LOWER STRUCTURE DESIGN OF GRAND GLOBAL HOTEL BANJARMASIN

Rafa'na Rahman, Rusdiansyah, Nursiah Chairunnisa
Department of Civil Engineering, Faculty of Engineering, Lambung Mangkurat University
Author: rafarahmann@gmail.com

ABSTRACT

As one of the cities with continuing development and improvement, that expansion in various sectors also impacts the population growth. It is increasing population numbers led to high public demand for the availability of land and space for new settlements up to the office. The high demand for land and space are not well followed by the availability of land itself so that high-rise buildings could be the perfect solution to utilize the functionality and capacity of a building so that it becomes more efficient. Grand Global Hotel Banjarmasin contributes to the development in the tourism sector as it becomes the housing option for residents.

The design of Grand Global Hotel Banjarmasin consists of designing the upper and lower structure. Structure design refers to the SNI 2847-2013, for the load refers to the SNI 1727-2013, and for earthquake loads calculation refer to SNI 1726-2012. With the 30 Mpa for FC’ and 400 Mpa for its FY. Banjarmasin classified in B category in seismic design with SE site category classification. The structural analysis assisted by the SAP2000 program.

Based on the results, the design obtained two types of the beam. A 400 x 500 mm dimensions of B1 – B2, and 300 x 400 mm dimensions of BA1 – BA2 with each diameter as 19 mm for main reinforcement and 10 mm for stirrup reinforcement. There are three types of columns that obtained which are 700 x 700 mm, 600 x 600 mm, 450 x 450 mm. A 120 mm thickness of floor slab. A 200 mm thickness for walls and plate basement. It is using the spun pile foundation with 500 mm diameter and 40 m length. There are four types of pile cap, which are six poles (PC-1), nine poles (PC-2), 12 poles (PC-3), 14 poles (PC-4).

Keywords: High-rise buildings, concrete, reinforcement, soil bearing capacity

1. INTRODUCTION

They were increasing population numbers led to high public demand for the availability of land and space for new settlements and offices. This high demand is not well followed by the availability of land and space in Banjarmasin itself so that high-rise building is the right solution to overcome these problems (Poulus, 2017). Story building is a building that has more than one floor or story to increase the functionality and capacity of the building, so it becomes more efficient. Banjarmasin is now taking shape with the construction of many new high rise building in the business center as in the area of KM. 2 and at the center of government as on Jl. Mangkurat, Jl. MT Haryono and surrounding areas. In high-rise building, wind load and earthquake load will provide a wobble with increasing height of the structure. Such expenses will be forwarded to the structure that is the foundation bottom. The foundations of the building must be able to ensure the stability of the building to vertical loads (such as the live load of building functions, the dead load of the beam, column and slab) and horizontal loads (such as wind load and earthquake load) and should not be a decline in the foundation of the local or uneven exceeding of the limit specific (Gunawan, 1983).

Based on the Banjarmasin geographical conditions that made of mostly soft soil will certainly provide challenges to the construction to be made. Problems are often found in multi-story buildings on soft soil is on the low bearing capacity, a substantial drop, and uneven, causing the occurrence of cracks on the building until the gap between the floor of the building.

To overcome the problems caused by the soil condition, the use of a foundation that can withstand vertical and horizontal loads. Thus, this scheme will be designed using the building foundation on soft soil to get an otherwise decent and safe place to bear the burdens of existing thereon.

2. THEORETICAL STUDY

Loads

The load design is based on SNI 1727-2013. Loads generally classified into two different categories, which are static and dynamic.

Static Forces

1. Dead loads
   Based on Article 3.1, the dead load is the weight of all building construction materials
2. Live Loads
   Based on Article 4.1, the live load is the load caused by the user and occupants of buildings or other structures that are not included in the construction load and environmental load

Dynamic style

1. Wind Loads
   In high-rise buildings, wind loads need to be calculated in designing the structural system of the building as it can lead to wobble. Wind loads analysis refers to Article 27.2
2. Quake loads
   In a building, the heavier the weight of a building, the greater the force that occurs. According to Indarto, et al. (2013), seismic loads analysis procedures based on SNI 1726-2012 in buildings has calculation stages as follows:
   a. Building structure risk category (I-IV)
   b. Determining the seismic priority category, I_e.
   c. Determining the mapped seismic acceleration parameter (S_s, S_1).
   d. Determining the site classification ($SA – SF$)
   e. Determining the site coefficient
   f. Determining the response spectrum design
   g. Determining the seismic design category (A-D)
   h. Determining the structure of the system and the parameters system (R, Cd, Ω0)
   i. Determining the fundamental period approach (Ta)
   j. Calculate the seismic response coefficient (Cs)
   k. Calculating the effective seismic weight (W)
   l. Calculating the base shear force (V)

Loads Combination

The factored loads combination for strength design methods are as follows:

1. 1.4D
2. 1.2D + 1.6L + 0.5 (Lf or S or R)
3. 1.2D +1.6 (Lf or S or R) + (L or 0.5W)
4. 1.2D + 1.0W + L + 0.5 (Lf or S or R)
5. 1.2D + 1.0E + L + 0.2s
6. 0.9D + 1.0W
7. 0.9D + 1.0E

Selection of the Foundation Types

Selection of the type of foundation should consider the state of the soil, the amount of load to be received by the foundation, environmental restrictions, and the cost - time of implementation. The key to designing the foundation for high-rise buildings is to be ensured that the system has the capacity and stability of the foundation are eligible to resist all loads and load combinations. The foundation system must have adequate capacity to withstand vertical and lateral loads, and resistance to rotation is sufficient to withstand the moment and torsion.

Design Method

The design procedure of this building is referring to SNI 2847-2013

Design Beam

The cross-section design that is subjected to bending must meet the requirements of SNI 2847-2013 Article 22.5.1 as follows:

$\emptyset M_n \geq M_u$

M_u is the ultimate moment on the cross-section of the review, and Mn is the nominal moment.

The design of beam shear strength must meet the requirements of SNI 03-2847-2013 Article 11.1.1, as follows:

$\emptyset V_n \geq V_u$

Vu is the factored shear force on the cross-section of the review and Vn is the nominal shear strength.

Design column

Column cross-section design must meet the requirements of SNI 2847-2013 Article 22.5.1 and Article 22.5.2, as follows:

$\emptyset M_n \geq M_u$

$\emptyset P_n \geq P_u$

Plate

The minimum slab thickness (t_p), must be following the regulation on SNI 2847-2013 Article 9.5.3.3 as follows:

1. With the $\alpha fm \leq 0.2$,
   tp min = 125 mm (without drop panels)
   tp min = 100 mm (with drop panels)

2. For 0.2 <αfm ≤ 2.0,
   tp min = 125 mm
   $h = \frac{\ell_n\left(0.8+\frac{fy}{1400}\right)}{36+5\beta\left(\alpha_{fm}-0.2\right)}$
3. To αfm> 2.0,
   tp min = 90 mm
   $h = \frac{\ell_n\left(0.8+\frac{fy}{1400}\right)}{36+9\beta}$

Basement

Based on the procedures for calculation of concrete structures for buildings SNI 03 1728 2002 Article 16.5.3. The wall basement thickness should not be less than 190 mm.

Foundation

Pile bearing capacity of the vertical load is calculated based on SPT data with Meyerhof method against lateral load with Broms method.

Vertical Bearing Pile Capability Based on SPT

On the grained ground (granular soil) used method of Meyerhof (1956),

$Q_u = Q_b + Q_f = 40N_{cor}\left({}^{l}/_{d}\right)A_b + 2\overline{N}_{cor}A_s$

with,

QB = $40N_{cor}\left({}^{l}/_{d}\right) \leq 400N_{cor}$

Ncor = CN No. Uh C_d cs Cb

Based on SNI 8460-2017 Article 9.2.3. The minimum safety factor for vertical ultimate bearing capacity is 2.5 for the deep foundation. Murthy (2007) recommended safety factor value is more than one or equal to 4.

Lateral Bearing Pile Capacity

Lateral forces that occur on the pile depends on the rigidity of the pole type, kind of soil, planting the pole tip into the cover plate pole head. For pile wedged end (fixed-head), lateral load (Pu) is equal to the carrying capacity of the ultimate lateral pile (Broms, 1964a):

Pu =$\frac{2M_y}{(1.5D+0.5f)}$

The lateral ultimate bearing capacity for long fixed-head pile can be determined from P_{ult}/c_uD^2 and M_{yield}/c_uD^3 relation graphs. (Murthy, 2007)

Pile Deflection

Based on Broms (1964) method, the pile is classified as a fixed-head long pile if βL > 1.5. Then the deflection of pile ends in cohesive soils at ground level is as follows,

yo = with, $\frac{H\beta}{kD}\beta = \sqrt[4]{\frac{kD}{4EI}}$

(Pamungkas & Harianti, 2013)

The safety factor for pile deflection based on SNI 8460-2017 Article 9.7.3.1 is 12 mm for planned earthquake and 25 mm for a strong earthquake.

3. DESIGN METHOD

Flowchart

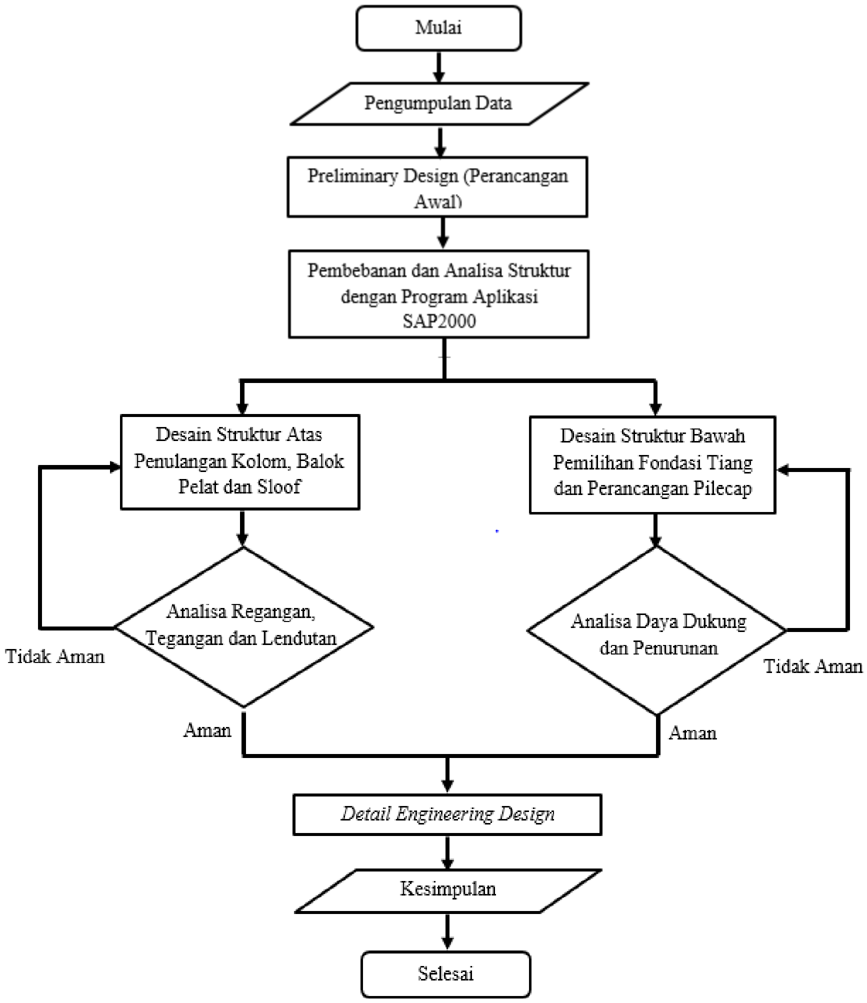


Figure 1 diagrams the flow

4. DISCUSSION

Data Planning

- Building Function = Hotel
- Quality concrete (fc ') = 30 MPa
- Quality steel (fy) = 400 MPa
- Modulus elasticity of concrete (Ec) = 4700√ (fc ') = 25742.960 MPa
- Modulus elasticity of steel (Es) = 200000 Mpa

Preliminary Design

Beam

Based on SNI 2847-2013 Section 9.5.2.2. Design for continuous beam one end and a second end of the constant can be seen in Table 1. And Table 2.

Table 1 Preliminary Design

Kondisi	Balok	
	h	b
Satu ujung menerus	$\frac{1}{18.5}L \times (0.4 + \frac{fy}{700})$	$\frac{2}{3}h$
Dua ujung menerus	$\frac{1}{21}L \times (0.4 + \frac{fy}{700})$	
Kantilever	$\frac{1}{8}L \times (0.4 + \frac{fy}{700})$	

Table 2 Preliminary Design Beams

Kode	Panjang (mm)	Satu Ujung Menerus		Kedua Ujung Menerus		Direncanakan		Dimensi (mm)
		h (mm)	b (mm)	h (mm)	b (mm)	h (mm)	b (mm)	
B1	7000	367,57	245,05	323,81	215,87	500	400	400/500
B2	6400	336,06	224,04	296,05	197,37			
BA1	7000	367,57	245,05	323,81	215,87	400	300	300/400
BA2	6400	336,06	224,04	296,05	197,37			

Column

The minimum thickness of the column that is as wide as the beam rested thereon. So the size of the columns used in this design is 450x450 mm2 column, 600x600 mm2, and 700x700 mm2.

Plate

Example calculation of the ground floor slab preliminary design:

1. Calculating the β value
   Sample calculation A panel β
   $$\beta = \frac{L_n}{L_s} = \frac{3650}{3150} = 1.159$$
   β average ground floor = 1.311
2. Calculating the minimum plate thickness
   Obtained αfm = 31.277> 2 (greater than 2), the provisions of the minimum plate thickness on SNI 03-2847-2013 Article 9.5.3.3 should not be less than 90 mm and shall not be less than the following formula equation:
   $$h = = \frac{\ell n\left(0,8+\frac{fy}{1400}\right)}{36+9\beta}\ \frac{6050\left(0,8+\frac{400}{1400}\right)}{36+9(4,481)} = 86.056 \text{ mm}$$
   thus, take the ground floor slab thickness amounting to 120 mm.
   Preliminary plate design can be seen in Table 3.

Table 3 Preliminary design plate

Lantai	Tebal Pelat (mm)
Dasar	120
2 s.d 8	120

Loads

The loads on the input to the SAP program consists of dead loads, live, wind, and earthquake.

1. Dead load
   Results dead load calculations based on SNI 1727-2013 and PPURG 1989 can be viewed at **Error! Reference source not found.**
2. Live Loads
   Results of live load calculation based on SNI 1727-2013 can be seen in **Error! Reference source not found.** and **Error! Reference source not found.**

3. Wind loads
   Wind load calculations based on ISO 1727-2013 Article 26 and Article 27 RESULTS:
   Wind pressure (p)
   p = Q (GCP) - qi (GCpi) (N / m2)
   a. The inflation pressure on the side wall of wind comes
      P_{FINAL} for the wind walls come amounted to 0.770 kN / m2
   b. Wind pressure on the windward side wall away
      P_{FINAL} for wind wall go equal to -0.012 kN / m2.
4. Quake loads
   a. Risk categories: risk category II
   b. Factors primacy of the earthquake (Ie) : 1.0
   c. Short periods of acceleration parameter (Ss) : 0.10 g
   d. Parameter acceleration period 1 second (S1) : 0.05 g
   e. Classification of sites based on standard penetration resistance average field: SE
   f. Fa sites coefficient: 2.5 and coefficient Fv site: 3.5
   g. Under the provisions of Article 6.4 ISO 1726-2012, obtained graphic design response spectrum, see Figure 2.

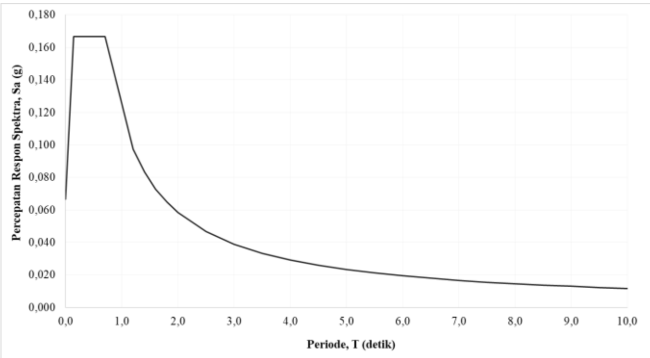


Figure 2 design response spectrum

   h. Seismic design categories:
      g = 9.81 m / s2; Ie = 1; R = 3
      RS-X (X direction in response spectra)
      U1 $= 100\% \times ()\frac{g \times Ie}{R} = 100\% ()\times \frac{9.81 \times 1}{3} = 3, 27$
      U2 $= 30\% \times ()\frac{g \times Ie}{R} = 30\% ()\times \frac{9.81 \times 1}{3} = 0, 98$
      RS-Y (Y direction in response spectra)
      U1 $= 30\% \times ()\frac{g \times Ie}{R} = 30\% ()\times \frac{9.81 \times 1}{3} = 0, 98$
      U2 $= 100\% \times ()\frac{g \times Ie}{R} = 100\% ()\times \frac{9.81 \times 1}{3} = 3, 27$

Structural analysis

Steps calculation of the structure with the SAP program are as follows:

1. Modeling the structure with defining section properties and enter values loading such as dead loads, live loads, wind loads, and earthquake, then define the combination of loading and conducted Run Analysis.

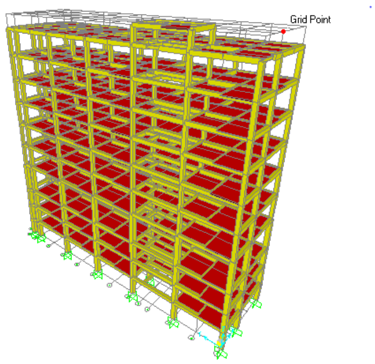


Figure 3 Modeling 3D on a computer program

2. Modeling Control Structure

Modeling control structures include control over:

a. Participation mass ratio

Based on SNI 1727-2013 Article 7.9.1, wide mass participation combined structure of at least 90% of the actual mass in each direction horizontally. Used 12 varieties vibrating patterns and mass participation to a wide mass participation has to comply with Sum UX amounted to 93.22%, and Sum UY amounted to 93.19% on the 12th modes.

b. Base Shear Force

Base shear force results from the response spectrum analysis with SAP program can be seen in Table 4.

Table 4 1 The shear force response spectrum analysis

TABLE: Base Reactions	
V_{tx} (ton)	V_{ty} (ton)
88,52	94,40

Base shear force equivalent static analysis is as follows,

Ct = 0, 0466; x = 0.9

hn = 33.5 m

Ta = *Ct* hnx

= 0.0466 × 33,50,9

= 1.10 sec

Calculation of the seismic response coefficient sees Table 5.

Table 52 Limitation of the seismic response coefficient

$C_{s\,min} = 0.044\ S_{DS}\ I_e$	$C_s = S_{DS} / (R/I_e)$	$C_{s\,maks} = S_{D1} / (T(R/I_e))$	C_s yang digunakan
0,0073	0,0278	0,0177	0,0177

From computer programs calculation, found effective seismic weight, as shown in Table 6.

Table 6 Weight seismic effective

TABLE: Base Reactions	
Load Case/Combo	FZ (ton)
Dead	4299,04
Live	1767,91
W = D + 50% L	5182,99

V = Cs W = 0.0177 × 582.99

= 91.8771 tons

Control of Vt> 0,85V

Vtx = 88, 52 tons > 0.85V = 78.18 tons ...**(OK)**

vTY = 94, 40 tons > 0.85V = 78.18 tons ...**(OK)**

c. Deviation between floors
Story drift control based on ISO 1726-2012 Article 7.12.1 sees Table 7.

Table 7 Control deviation between floors

Tinggi (h_{sx})	Lantai	Simpangan antar lantai tingkat desain (Δ)		simpangan antar lantai tingkat izin	Kontrol
m		arah-X	arah-Y	(Δa) = 0,025 h_{sx}	Δ ≤ Δa
3	Lantai Dasar	0,0000	0,0000	0,0750	OK
3	Lantai Mizanie	0,0017	0,0012	0,0750	OK
4,5	Lantai 2	0,0054	0,0038	0,1125	OK
3,5	Lantai 3	0,0097	0,0068	0,0875	OK
3,5	Lantai 4	0,0178	0,0121	0,0875	OK
3,5	Lantai 5	0,0249	0,0168	0,0875	OK
3,5	Lantai 6	0,0312	0,0207	0,0875	OK
3,5	Lantai 7	0,0369	0,0240	0,0875	OK
3,5	Lantai 8	0,0420	0,0267	0,0875	OK
3,5	Lantai dak	0,0461	0,0287	0,0875	OK
2	M/C ROOM TOP	0,0452	0,0252	0,0500	OK

Reinforcement Design of Bending Beam

The forces used to design reinforcement beam bending obtained from a computer program. Based on the manual calculation of longitudinal reinforcement, a transverse beam is obtained as shown in Table 8.

Table 8 Results of the calculations of reinforcement beams

Kode	Dimensi	Tulangan Tumpuan		Tulangan Lapangan		Tulangan Geser	
	cm	Tarik	Tekan	Tarik	Tekan	Perlu	Minimum
B1	40/50	5D-19	4D-19	4D-19	5D-19	D10-150	D10-200
B2	40/50	5D-19	4D-19	4D-19	5D-19	D10-150	D10-200
BA1	30/40	4D-19	3D-19	3D-19	4D-19	D10-100	D10-150
BA2	30/40	4D-19	3D-19	3D-19	4D-19	D10-100	D10-150

Bending Reinforcement Design Column

Column longitudinal reinforcement calculated by a computer program while the transverse reinforcement calculated manually to obtain the results as shown in Table 9.

table 9 3 The result of the calculation of column reinforcement

TIPE	Dimensi (mm)		Tulangan Longitudinal	Tulangan Transversal	
	b	h		Perlu	Minimum
C1	700	700	20D20	6D10-100	2D10-100
C2	600	600	20D20	6D10-100	6D10-100
C3	600	600	20D20	6D10-100	6D10-100
C4	450	450	20D20	5D10-100	5D10-100

Reinforcement Design and Plate Bending Basement Walls, Floor Plates

Planned basement walls and plates are having a thickness of 200 mm. The plate design moment coefficient calculated manually by the method of moments (PBI-71). The needs of each floor slab reinforcement can be seen in Table 10.

Table 10. Results of the reinforcing plate

Dinding	Tulangan yang Dipakai	
	Tumpuan	Lapangan
Basement	D13 - 250	D13 - 300
Lantai	**Tulangan yang Dipakai**	
	Tumpuan	**Lapangan**
Basement	D13 - 200	D13 - 300
Dasar	D13 - 300	D13 - 450
Mizanie	D13 - 400	D13 - 450
2	D13 - 400	D13 - 450
3-6'	D13 - 400	D13 - 450
7	D13 - 400	D13 - 450
8	D13 - 400	D13 - 450
Dak	D13 - 400	D13 - 450
M/C Room Top	D13 - 400	D13 - 450

Soil Interpretation

Based on the investigation of the soil in the profile obtained, soil sampling and soil layer characteristics are varied but dominated by a layer of soft soil. Bedrock was found at a depth of about 40 m. Selection of foundation using piles in and do pole to a depth of 40 m from ground level.

Design Foundations

Pile Vertical Capability Based on SPT data

Carrying capacity is based on SPT data with methods Meyerhoff (1956):

- L = 40m; D = 0.5 m
- N_{cor} = N60 = C_N No. Uh C_d cs Cb
  = 24.70 (Bowles, 1996)
  $\overline{N}_{cor}$= 24.70
- Prisoners ultimate end,
  $Q_b = 40N_{cor}\left({}^{l}/_{d}\right)A_b \leq 400N_{cor}A_b$
  = 3881,41 ≥ 1940.65 kN
  Taken Qb = 1940.65 kN
- Ultimate frictional resistance,
  Q_f = = 2 × 25.51 × 1.571 = 80.16 kN$2\overline{N}_{cor}A_s$
- Ultimate bearing capacity,
  $Q_u = Q_b + Q_f = 2020{,}81$ kN
  Qizin = Qu / SF (SF = 3)
  = 673.603 kN

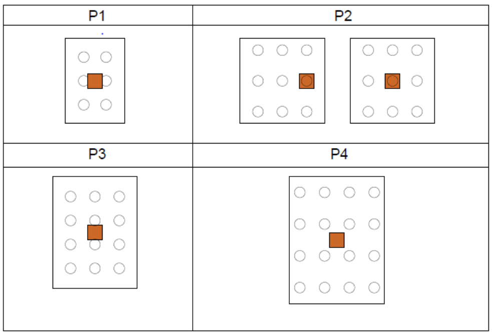
The number of piles needed

The required number of poles obtained from the division between the axial forces that occur can be seen in Table 11 the importance of the diversity plan is based on the number of poles pile cap can be seen in Table 12

Table 11 Number of poles

Joint	Pu	Jumlah Tiang		Tipe Pile Cap
	kN	Perlu	Terpasang	
1	2286,72	4	6	PC-1
51	2303,75	4		
141	2685,76	4		
191	2773,09	5		
11	3227,33	5	9	PC-2
21	3410,97	6		
121	3650,71	6		
71	3748,14	6		
31	3950,02	6		
41	3993,01	6		
171	4557,26	7	12	PC-3
181	4573,58	7		
161	4586,96	7		
151	4661,42	7		
81	5922,08	9	14	PC-4
91	6095	10		
101	6633,06	10		
111	6664,94	10		

Table 12 Forms plan pile cap



Efficiency Column Group

The calculation of the efficiency of pile groups based on a formula Converse-Labarre of the Uniform Building Code AASHTO:

Eg = 1 - θ $\frac{(n-1)m+(m-1)n}{90mn}$, With θ = tan-1 (D / s)

The calculation of the efficiency of the pole can be seen in Table 13.

Table 13 Efficiency pile group

TIPE	Jumlah Tiang	n	m	Jarak Digunakan		θ	Eg	Q_{izin} (kN)	Pu (kN)	Kontrol Pu < Q_{izin}
				Diambil	s (m)					
PC-1	6	2	3	2,5D	1,25	21,80	0,72	2899,41	2773,09	OK
PC-2	9	3	3	2,5D	1,25	21,80	0,68	4104,36	3993,01	OK
PC-3	12	3	4	2,5D	1,25	21,80	0,66	5309,31	4661,42	OK
PC-4	14	3	4	3D	1,5	18,43	0,71	6693,92	6664,94	OK

Maximum load Pole on Pole Group

Axial loads and moments that will work are distributed to the pile cap.

$$Pmaks = \frac{Pu}{n\ tiang} \pm \frac{My\ .\ Xmaks}{\sum X^2} \pm \frac{Mx\ .\ Ymaks}{\sum Y^2}$$

Calculations can be seen in Table 14.

Table 14 Control the maximum load on the pole group.

TIPE	Jumlah Tiang	Pu_{maks} (kN)	Mx_{maks} (kNm)	My_{maks} (kNm)	s (m)	x_{maks} (m)	y_{maks} (m)	ΣX^2	ΣY^2	P_{maks} (kN)	Kontrol $P_{maks} < Q_{izin}$
PC-1	6	2773,09	-66,87	-98,85	1,25	1,25	1,25	0,3	0,8	-119,29	OK
PC-2	9	3993,01	-102,10	-97,46	1,25	1,3	1,9	0,8	0,8	42,71	OK
PC-3	12	4661,42	-70,46	-104,73	1,25	1,3	1,9	0,8	1,6	136,33	OK
PC-4	14	6664,94	-130,23	-100,21	1,5	2	3	1,1	2,3	102,01	OK

Lateral Supports Power Pole

Based on data in the field and after the test laboratiories, the land is dominated by clay (cohesive). Carrying capacity of the lateral pole with Broms method (1964) for cohesive soil with the tip wedged in the pile cap is as follows:

- *L* = 40 m; *D*= 0.5 m
- *Cu* = 229.09 kN / m2
- T = $\sqrt[5]{EI/n_h}$ = 2.93

  L/ T = 13.64 ≥ 4 (pole length)
- *My = Mcrack* = 102.97 KNM

  Hu = 9*Cu* D (L - 3D / 2) = 40462.46 kN

  On Hu the moment that occurs on the pole:

  Mmax = Hu (L / 2 + 3D / 2) = 152424.59 KNM> My = 102.97 KNM (long pole)
- *My*/ CuD3 = 3.60, fromFigure 4 Graph relationship My / CuD3 with Hu / CuD2 obtained,

  Hu/ CuD2 = 4.2

  Hu Cu = 24 D2 = 240.54 kN
- *Hizin* = 40.090 kN (SF = 1.5 x 4 = 6)

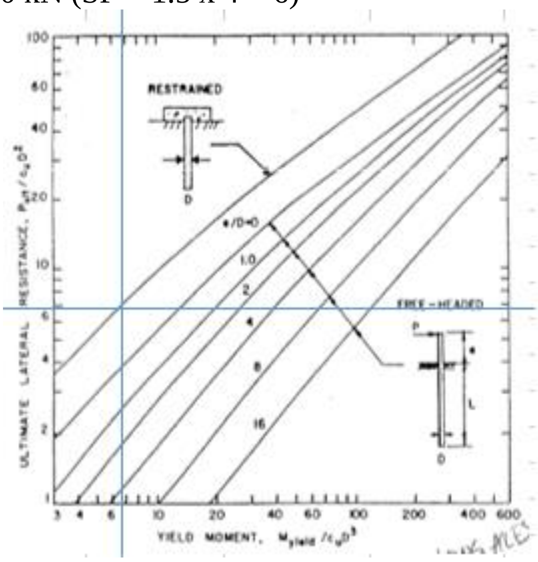


Figure 4 Graph relationship My / CuD3 with Hu / CuD2

Deflection Control Column

Vertical mast deflection control is calculated based on the method Broms (1964) as follows:

k = 50 MN / m3; = 0.51$\beta = \sqrt[4]{\frac{kD}{4EI}}$

βL = 20.73> 1.5 (Long Piles)
Thus, for the deflection of a long pole at ground level (yo),

$$yo = \frac{H_{izin} \cdot \beta}{kD} = 0.00083124 \text{ m} = 0.83124 \text{ mm} < 12 \text{ mm (OK)}$$

Settlement

Soil conditions that found at the end of the pile is dominated by sandy soil, so the decline was a decrease in immediately (elastic). For further calculations are presented in Table 15.

Table 15 Calculation of Decline

Joint	fz	L	B	Q	e	v	Is	Si	Batas Ijin	Cek Batas Izin	Beda Penurunan	Cek Beda Penurunan	Batas Izin Beda	Cek Batas Izin Beda
	kN													
1	228,67	1,05	2,25	96,79	50000	0,3	1	0,00396	0,1538	Aman	-0,00003	-0,00001	0,00333	Aman
51	230,38	1,05	2,25	97,51	50000	0,3	1	0,00399	0,1538	Aman	-0,00066	-0,00017	0,00333	Aman
141	268,58	1,05	2,25	113,68	50000	0,3	1	0,00466	0,1538	Aman	-0,00015	-0,00004	0,00333	Aman
191	277,31	1,05	2,25	117,38	50000	0,3	1	0,00481	0,1538	Aman	0,00264	0,00066	0,00333	Aman
11	322,73	2,3	2,9	48,39	50000	0,3	0,85	0,00217	0,1548	Aman	-0,00012	-0,00003	0,00333	Aman
21	341,10	2,3	2,9	51,14	50000	0,3	0,85	0,00229	0,1548	Aman	-0,00059	-0,00015	0,00333	Aman
121	365,07	2,3	2,9	54,73	50000	0,3	1	0,00289	0,1548	Aman	-0,00008	-0,00002	0,00333	Aman
71	374,81	2,3	2,9	56,19	50000	0,3	1	0,00297	0,1548	Aman	0,00031	0,00008	0,00333	Aman
31	395,00	2,3	2,9	59,22	50000	0,3	0,85	0,00266	0,1548	Aman	-0,00003	-0,00001	0,00333	Aman
41	399,30	2,3	2,9	59,87	50000	0,3	0,85	0,00269	0,1548	Aman	-0,00092	-0,00023	0,00333	Aman
171	455,73	2,3	3,39	58,45	50000	0,3	1	0,00361	0,1557	Aman	0,00063	0,00016	0,00333	Aman
181	457,36	2,8	3,89	41,99	50000	0,3	1	0,00297	0,1565	Aman	-0,00001	0,00000	0,00333	Aman
161	458,70	2,8	3,89	42,11	50000	0,3	1	0,00298	0,1565	Aman	-0,00005	-0,00001	0,00333	Aman
151	466,14	2,8	3,89	42,80	50000	0,3	1	0,00303	0,1565	Aman	-0,00004	-0,00001	0,00333	Aman
81	592,21	3,51	4,5	37,49	50000	0,3	1	0,00307	0,1575	Aman	-0,00009	-0,00002	0,00333	Aman
91	609,50	3,51	4,5	38,59	50000	0,3	1	0,00316	0,1575	Aman	-0,00028	-0,00007	0,00333	Aman
101	663,31	3,51	4,5	41,99	50000	0,3	1	0,00344	0,1575	Aman	-0,00002	0,00000	0,00333	Aman
111	666,49	3,51	4,5	42,20	50000	0,3	1	0,00346	0,1575	Aman	0,00346	0,00086	0,00333	Aman

Dimensional Planning Pile Cap

*Pile cap*must be designed to be able to bear a sliding one-way and two-way. Planning dimensions pile cap, and pile cap against the sliding control can be seen in Table 16 - Table 18.

table 16 4 Dimensions pile cap

TIPE	Jumlah Tiang	s (m)	x (m)	Dimensi Pile Cap (m)		
				P	L	t
PC-1	6	1,25	0,75	2,8	4,0	1,0
PC-2	9	1,25	0,75	4,0	4,0	1,0
PC-3	12	1,25	0,75	4,0	5,3	1,0
PC-4	14	1,5	0,75	4,5	6,0	1,0

table 17 5 Control slide in one direction

TIPE	Tipe Kolom	Dimensi		øt (mm)	Pu (Ton)	Kontrol Geser Satu Arah						
		bk	hk			A (m2)	σ (t/m2)	d (m)	G' (m)	Vu (ton)	φVc (ton)	φVc > Vu
PC-1	C1	0,70	0,70	16	277,3089	11	25,21	0,917	0,108	7,4873	172,652	OK
PC-2	C1	0,70	0,70	16	278,3089	16	24,96	0,917	0,733	73,172	251,131	OK
	C1	0,70	0,70	16	399,3012	16	24,96	0,917	0,733	73,172	251,131	OK
PC-3	C1	0,70	0,70	16	466,1417	21	22,2	0,917	0,733	65,082	251,131	OK
PC-4	C1	0,70	0,70	16	666,4937	27	24,68	0,917	0,983	109,19	282,522	OK

table 18 6 Sliding two-way control

TIPE	Tipe Kolom	Dimensi		øt (mm)	Pu (Ton)	Kontrol Geser Dua arah												
		bk	hk			B'	βc	bo	αs	Vu (ton)	Vc1 (ton)	Vc2 (ton)	Vc3 (ton)	Vc (ton)	φVc (ton)	φVc > Vu	Mux (tonm)	Muy (tonm)
PC-1	C1	0,70	0,70	16	277,3089	1,617	1,000	6,47	40,00	124,73	1656,80	2076,68	1082,88	1082,88	812,16	OK	41,60	41,60
PC-2	C2	0,70	0,70	16	278,3089	1,617	1,000	6,47	30,00	334,05	1656,80	2076,68	1082,88	1082,88	812,16	OK	39,93	67,66
	C1	0,70	0,70	16	399,3012	1,617	1,000	6,47	40,00	334,05	1656,80	2076,68	1082,88	1082,88	812,16	OK	39,93	67,66
PC-3	C1	0,70	0,70	16	466,1417	1,617	1,000	6,47	40,00	297,12	1656,80	2076,68	1082,88	1082,88	812,16	OK	34,96	59,24
PC-4	C1	0,70	0,70	16	666,4937	1,617	1,000	6,47	40,00	435,33	1656,80	2076,68	1082,88	1082,88	812,16	OK	90,45	126,16

Reinforcement Design Pile Cap

Design moment happens to pile cap due to axial load, and reinforcement obtained as follows can be seen in Table 19.

Table 19 Results of the calculations of the reinforcement pile cap

TIPE	Tulangan yang Dipakai			
	Arah x		Arah y	
	Bawah	Atas	Bawah	Atas
PC-1	D16 - 50	D16 - 150	D16 - 100	D16 - 200
PC-2	D16 - 50	D16 - 150	D16 - 50	D16 - 150
PC-3	D16 - 50	D16 - 150	D16 - 100	D16 - 200
PC-4	D16 - 50	D16 - 100	D16 - 50	D16 - 100

5. CONCLUSION

The design of the Grand Global Building consists of the upper and lower structure. Structure calculation consists of a preliminary design, loading calculation, analysis structure by SAP2000, beam reinforcement, column reinforcement, plate reinforcement. Calculation of lower structure consisting of soil bearing capacity, calculation of the vertical and lateral pile, settlement, the planning dimension pile cap, and pile cap reinforcement. Based on the upper structure and the lower structure calculation, the conclusion as follows:

1. It is obtained two types of beam design. A 400 mm x 500 mm for B1 – B2 and 300 mm x 400 mm for BA1 – BA2 with a 19 mm diameter for main reinforcement and 10 mm for stirrup reinforcement.
2. It is obtained four types of column design. A 700 x 700 mm for C1, 600 x 600 mm for C2 and C3 and 450 x 450 mm for C4. Longitudinal and transversal reinforcement in each column used D20 and D10 reinforcement.
3. It is obtained a 120 mm thickness for its plate. Using 13 mm diameter for main reinforcement.
4. It is obtained a 200 mm thickness for the wall and plate basement. Using 13 mm diameter for main reinforcement.
5. Using the spun pile foundation with 500 mm diameter and 40 m length.
6. It is obtained the ultimate bearing capacity is 2020.81 kN with a safety factor of 3 and it is obtained $Q_{allowed}$ amounted to 673.603 kN. And $H_{allowed}$ amounted to 40.090 kN.
7. The settlement was safe enough for the building, with the dense soil condition the settlement that is happened is immediate settlement.
8. There are 4 types of pile cap, a 2.8 x 4.0 x 1.0 m with 6 poles as PC-1, 4.0 x 4.0 x 1.0 m with 9 poles as PC-2, 4.0 x 5.3 x 1.0 m with 12 poles as PC-3, 4.5 x 6.0 x 1.0 m with 14 poles as PC-4. Each pile cap used 16 mm diameter reinforcement for x and y direction.

BIBLIOGRAPHY

1. Asroni, Ali. 2010b. Column Foundations & T Concrete Beams. Graha Science: Yogyakarta.
2. Afwan, True Faizal & Jane Dela Dacrea. 2008. High-Level Building Planning System Flat Plate Structure - Core Wall Building System. UNDIP. Semarang.
3. Bowles, Joseph E. 1997. Foundation Analysis and Design 5th Edition. Mc-Graw-Hill Book Co.: Singapore.
4. Broms, B. 1964a. "Lateral Resistance of Piles in Cohesive Soils", JSMFD, ACE, Vol 90, SM2.

5. BSN. 2012. ISO 1726: 2012 Earthquake Resilience Planning Procedures for Building and Non-Structural Building. National Standardization Agency.
6. BSN. 2013. ISO 1727: 2013 Minimum Design Loads Buildings and Other Structures. National Standardization Agency.
7. BSN. 2013. ISO 2847: 2013 Requirements for Structural Concrete Building. National Standardization Agency.
8. BSN. 2017. Geotechnical Perancangaan ISO 8460-2017 requirements. National Standardization Agency.
9. Public Works Department. 1989. Imposition Planning Guide for Home and Building. Jakarta: Foundation Board Publisher PU.
10. Higher Education - Education Ministry. 1997. Engineering Foundations I: Construction Soil Retaining. Jakarta: Gunadarma.
11. Gunawan, Ir. Rudy. 1983. Introduction to Engineering Foundations. Kanisus: Yogyakarta.
12. Gunel, Mehmet H, & Hüseyin Emre Ilgin. 2014 Tall Buildings: Structural Systems and aerodynamic form. Routledge: New York.
13. Hallenbrand, Erik, and Wilhelm Jakobsson. 2016. Structural Design of High-Rise Buildings. Lund University: Sweden.
14. Hardiyatmo, Hary Christady. 2006. Mechanical Foundations 2. Beta Offset: Yogyakarta.
15. Indarto, Hima, et al. 2013. Applications Earthquake SNI 1726-2012 for Dummies. Civil Engineering UNNES: Semarang.
16. Idham, Noor Cholis. 2013. Designing Low-Rise Building. Graha Science: Yogyakarta.
17. Ngunaidi, Ali K et al. 2016. Alternative Pole Foundations for Building Planning High on Soft Soil in New City Development Project Summarecon-Bandung. ITS Engineering Journal. Surabaya
18. Poulos, Harry G., 2017. Tall Building Foundation Design. Taylor & Francis: Boca Raton.
19. Prasetyo, Yunita Anggreni. 2019. Design North Building 21 Floor Duta Mall Banjarmasin. Essay. University Mangkurat.
20. Sardjono, HS 1988. Foundations Piles II. Sinar Wijaya: Surabaya.
21. Schodek, Daniel L., & Martin Bechthold. 2014. Structures: Seventh Edition. Pearson Education: United States.
22. Setiawan Agus. 2016. Design of Concrete Structures Based on ISO 2847: 2013. Erland: Jakarta.
23. Surjandari, Nikan Silmi. 2009. Analysis decline Raft Foundation in Soft Soil. Gema Technique: Semarang.
24. Tjitradi, Darmansyah. 2018. Concrete Structures II. Faculty of Engineering. University Mangkurat: Banjarmasin