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A Review on Structure Design of the Hospital Building in Malang which used Special-Moment Resisting Frame System

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Abstract

A building is a physical product of a construction project where people do all kinds of activities inside. A building Design has to meet the requirements set in order that it will not put the people who use the building in danger. A lot of methods are used to design a building. One of them is Special Moment-Resisting Frame system. The result of the review that has been conducted shows that the design of the structure of the hospital building in Malang which used Special-Moment Resisting Frame system gives a significant description on the steps should be taken, and the result obtained from the plan gives an additional description on the design of a building structure.

Keywords: *Design, building, system, frame*

1. Introduction

Concrete is still the primary materials used by the construction workers in constructing buildings, bridges, roads, and other construction works.

When designing a building frame using reinforced concrete, affecting factors such as the loads that act, the strength of the soil bearing capacity, the dimension of the reinforced concrete used, and the earthquake factor applied in the region where the building will be built must be taken into account.

A hospital building is a specific building in its architecture, structure, mechanics-electrics and plumbing, as well as environmental governance. Architecturally, it has to meet the requirements of Medical Architecture; structurally, according to SNI 1726:2012 about the Procedures of earthquake resistance designs for building and non-building structures, a hospital building is categorized into Level-IV structure risk category. Because of its importance factor, the building has to perfectly function both in mechanics-electrics and in plumbing in any condition. In the same Regulation, it is also stated that the ratio coefficient mapped of C_{RS} spectral response period of short period of 0.2 second in the region of Malang is 1.00-1.05g, and the ratio coefficient mapped of C_{RS} spectral response period of long period of 1 second is 0.95-1.00g. The soil analysis conducted by Laboratory of soil and rock mechanics of ITS Surabaya shows that the site of the propective activity location goes into the classificasion of SD site (medium soil) so that the building, structurally, has to be designed specifically.

This design will, hopefully, answer the problems in designing reinforced concrete structures that meet the requirements of Special Moment Resisting Frame system as written in SNI 1726:2012 about the Procedures of earthquake resistance designs for buildings and non-building structures and in SNI 2847:2013 about the requirements of structural concrete for buildings.

2. The Objectives To Achieve

The review carried out on the building structure plan using Special Moment Resisting Frame system is to find out how far steps can be taken and results can be obtained from the plan using Special Moment Resisting Frame system.

3. Articles Related To Building Designs

Many building structure designs have been carried out by using different methods or frame systems in their calculation process.

Manurung (2008) designed the building of RSUD Namlea in Buru Islands using SAP2000 software, in which the structure model was made into 3 dimensions, and the earthquake loading was executed using equivalent static method. The design norm used is SKSNI 1726: 2002. The loading combination used consists of LC1: 1.20DL +1.60LL, LC2: 1.20DL+ 0.50LL+ 1.00ELx+ 0.30ELy, LC3: 1.20DL+ 0.50LL+ 1.00ELy+ 0.30ELx. In the design, the output portal is structure element dimension, longitudinal reinforcement and shear reinforcement.

Prihatmoko (2013) designed Rusunawa Pringwulung Sleman Yogyakarta by using Special Moment Resisting Frame (SMRF) and Intermediate Moment Resisting Frame (IMRF) systems with the help of SAP2000 software, in which the structure model was made into 3 dimensions, and the earthquake loading was executed using equivalent static method. The design norm used is SKSNI 1726: 2002. The loading combination used consists of LC1: 1.40DL, LC2: 1.20DL+ 1.60LL, LC3: 1.20DL+ 1.00LL+ 1.00Ex+ 0.30Ey+ 0.80Wy+ 1.30Wy, LC4: 1.20DL+ 1.00LL– 1.00Ex– 0.30Ey– 0.80Wx – 1.30Wy, LC5: 1.20DL+ 1.00 LL + 0.30Ex+ 1.00Ey+ 1.30Wx+ 0.80Wy, LC6: 1.20DL+ 1.00LL– 0.30Ex– 1.00Ey – 1.30Wx– 0.80Wy, LC7: 0.90DL+ 1.00Ex+ 0.30Ey+ 0.80Wx+ 1.30Wy, LC8: 0.90 DL – 1.00Ex– 0.30Ey– 0.80Wx– 1.30Wy, LC9: 0.90DL+ 0.30Ex+ 1.00Ey+ 1.30Wx+ 0.80Wy, LC10: 0.90DL– 0.30Ex– 1.00Ey– 1.30Wx– 0.80Wy. In the design, the output portal is structure element dimension, longitudinal reinforcement and shear reinforcement.

Kurniawan and Sudrajat (2013) designed the patient wards building of Emanuel Hospital in Kabupaten Banjarnegara using STAADPro software, in which the structure model was made into 2 dimensions, and the earthquake loading was executed using equivalent static method. The design norm used is SKSNI T-15-1991-03.The loading combination used consists of LC1: 1.20DL +1.60LL, LC2: 1.05DL+ 1.05LL+ 1.05WL, LC3: 1.05DL+ 1.05LL- 1.05WL, LC4:1.40DL, LC5: 1.20DL+ 1.00LL+ 1.00EL, LC6: 0.90DL+ 1.00EL. In the design, the output portal is structure element dimension, longitudinal reinforcement and shear reinforcement.

Sumirin (2013) designed the building of Belefina Hospital in Semarang using SANSPro software, in which the structure model was made into 3 dimensions, and the eartquake loading was executed using equivalent static method. The design norm used is SKSNI 1726: 2002. The loading combination used consists of LC1:1.40DL, LC2: 1.20DL+ 1.60LL+ 1.00LR, LC3: 1.20DL+ 1.00LL+ 1.60LR LC4: 1.20DL+ 1.00WL+ 1.00LL+ 0.50LR, LC5: 1.20DL+ 1.00LL+ 1.00EL, LC6: 0.90DL+ 1.00WL, LC7: 0.90DL+ 1.00EL. In the design, the output portal is structure element dimension, longitudinal reinforcement, shear reinforcement, and confinement reinforcement.

4. Design Review

The review conducted is a review on the structure design of Malang hospital building which was designed using SAP2000 software, in which the structure model is made into 3 dimensions, and the earthquake loading is done using the spectral response method. The design norm used is SKSNI 1726: 2012. The loading combination used consisted of LC1: 1.20DL +1.60LL, LC2: 1.20DL+ 0.50LL+ 1.00RS1+ 0.30RS2, LC3: 1.20DL+ 0.50LL+ 1.00RS2+ 0.30RS1. The structure system used is Special Moment Resisting Frame (SMRF) structure. In the design, the output portal is structure element dimension, longitudinal reinforcement, shear reinforcement, and confinement reinforcement. The specific difference from the previous review is the application of the Earthquake SNI of 2012 and Concrete SNI of 2013, as well as the application of Spectral Response Graphic in accordance with the regulation in Earthquake SNI of 2012.

5. The Process Of The Building Design

In designing this bulding by using Special Moment Resisting Frame system, some steps have been done, i.e:

1) Estimating the dimension of the building structure based on the data from the architectural drawings;
2) Making 3-dimension building structure model based on the data from the architectural drawings with the help of SAP2000 software;
3) Giving load on the 3-dimension building structure model based on the data from the architectural drawings, as stipulated in SKBI-1.3.5.3-1987 about the Guidance to loading designs for houses and buildings.
4) Making loading combination of SNI 1726:2012 about the procedures of earthquake resistance designs for building and non-building structures;
5) Inputting calculation references based on SNI 1726:2012 about the procedures of earthquake resistance designs for building and non-building structures and SNI 2847:2013 about the requirements of structural concrete for buildings;
6) Controlling the structural dimension of the structure model and controlling the tremor time limits of the structure model;
7) Displaying the plate moment, the deep force acting on the beams and columns, and the support reactions at the foundation point;
8) Designing the whole structure elements by paying attention to the restrictions stated in SNI 1726:2012 about the procedures of the earthquake resistance designs for building and non-building structures and in SNI 2847:2013 about the requirements of structural concrete for buildings;

6. Discussions

The portal of the structure of the care unit hospital building in Malang was a concrete portal which is modeled as 3-dimension (3D) frame element in SAP 2000 by referring to the manual of the standards of Eartquake SNI of 2012 and Concrete SNI of 2013. The type of analysis used is Dynamic Analysis Procedure, which is selected by considering that a hospital building goes into KRB IV building risk category where the building location goes into KDS D seismic design category.

Dynamic analysis used in this structure design is Spectrum Response Dynamic Analysis in which the structure is modeled as lumped mass model to reduce the number of the structure freedom degree to accelerate the structure analysis process.

The loading combination used in the design is the fixed loading U = 1.2 DL + 1.6 LL and the temporary loading U = 1.2 DL + 0.5 LL ± Ex ± 0.3 Ey, U = 1.2 DL + 0.5 LL ± 0,3 Ex ± Ey. The live load factor was reduced to 0.5 because the rooms used had live loads less than 500 Kg/m².

The data of the structure design used for the analysis are:

1. The portal structure type of a 5-storey-and- 1-management-room concrete building structure, the building function as the care- unit hospital building (KRB IV), the location of Malang city (KDS D), the building design which is designed based on SMRF (Special Moment Resisting Frame) structure, the concrete characteristic compressive strength used i.e. f'c = 25 Mpa, the yield tension of the reinforced steel which is designed i.e. fy = 400 Mpa for the main reinforcement and fys = 240 Mpa for shear reinforcement.
2. The land site is medium soil (SD), the site coefficient is Fa= 1.1 and Fv = 1.5, Cu = 1.5, Ct = 0.0466, x = 0.9.
3. With KDS D, SDs= 0.526 and SD1 = 0.222, PGA= 0.43, FPGA = 1.
4. With SMRF, the response modification coefficient is R =8, the system excessive strength factor is Ω = 3 and the deflection expansion factor is C_d= 5.5.
5. Allowability of Drift between each floor is Δ = 0,01 hsx
6. For lift and ramp is ap= 1 and Rp = 2.5.

Table 1. Site Classification according to Earthquake *SNI* of 2012

Kelas Situs	V (m/dtk)	N	S (kPa)
SA (batuan keras)	>1500	N/A	N/A
SB (batuan)	750 – 1500	N/A	N/A
SC (tanah keras,sangat padat dan batuan lunak)	350 – 750	> 50	≥ 100
SD (tanah sedang)	175 – 350	15 – 50	50 – 100

The seismic waves travel through the bed rock under the earth surface. From the depth of this bed rock, the seismic waves then travel to the earth surface while at the same time they amplify depending on the type of the soil layer above the bed rock. The type of the soil layer is determined from the average value of the shear strength of the subgrade as follows:

$$S = c + \gamma . h . tg\phi \tag{1}$$

$$\bar{S}_u = \frac{\sum_{i=1}^{m} t_i}{\sum_{i=1}^{m} \frac{t_i}{S_{ui}}} \tag{2}$$

S_{ui} = Shear Strength of soil of i-layer
m = Number of soil layers above the bed rock
t_i = Thickness of i-layer soil

Table 2. Calculation of niralir shear strength of subgrade

No	H	□□ sat	C	□	S = c + □ * tg □	h/s
	Cm	Kg/cm3	Kg/ cm2	Derajat	Kg/ cm2	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	500	0.001843	0.0	37.80	0.714	700.12
2	500	0.001836	0.0	38.61	0.733	682.53
3	500	0.001854	0.0	38.34	0.732	682.75
4	500	0.001903	0.0	39.40	0.781	640.06
5	500	0.001984	0.0	40.28	0.840	595.08
6	500	0.001999	0.0	41.30	0.878	569.76
Tot.	3000					3,870.3

Su = 3000/3870 = 0.775 Kg/ cm^2 = 77.5 KPa

The data shows that the soil N-SPT of the building location ranges from 15-50, therefore according to the Earthquake SNI of 2012 the land site goes to the category of medium land (SD site). The Earthquake Response Factor is obtained from the earthquake response spectrum diagram in www.puskim.go.id as pointed out in the Earthquake SNI of 2012, based on the building location of Malang city dan the SD rock site.

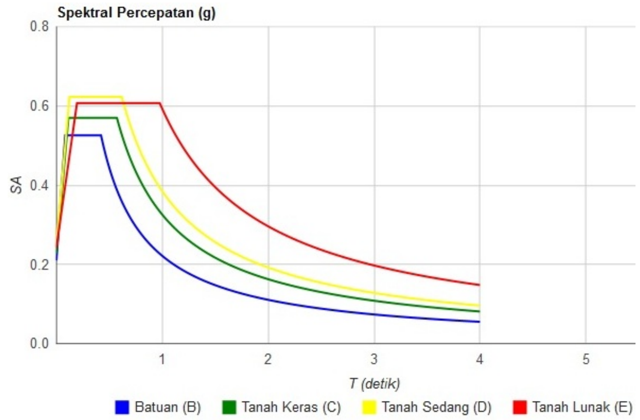

Figure1. Earthquake Response Spectrum of Malang city

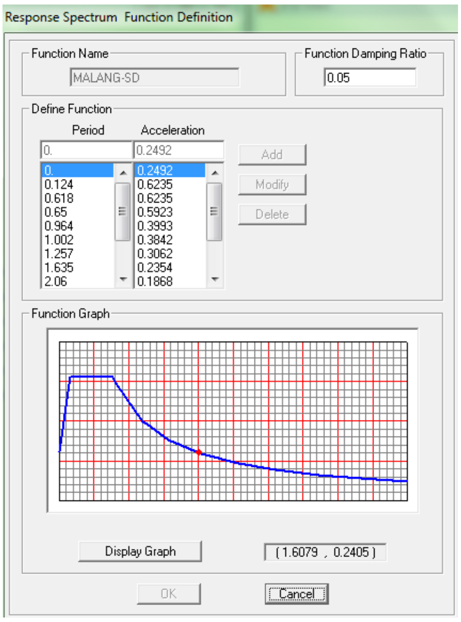

Figure 2. RS Plotting in SAP2000

The calculation of the building weight is gained by adding up the dead load and the reduced live load acting on each floor of the building. According to the Earthquake SNI of 2012 the live load reduction factor to calculate the weight of the building structure is 50%, so that the structure weight used is:

$$Wt = 100\%\ DL + 50\%\ LL \qquad (3)$$

The calculation of the structure weight is done by placing a pinned support on each floor in the SAP2000 modelling so that the data of the weight on each floor and the location of the mass center point of the floor can be determined.

Based on the weight of each floor of the building structure, the mass value of each floor is determined by dividing the value of the structure weight and the value of the gravity acceleration.

Table 3. Building weight table

Floor	Weight (Ton)	Gravity (m/det^2)	Mass (Ton.det^2/m)
2nd Floor	3028	9.81	308.66
3rd Floor	3008	9.81	306.63
4th Floor	2961	9.81	301.83
5th Floor	2913	9.81	296.94
6th Floor	2192	9.81	223.45
Roof	481	9.81	49.03

The following is the data input in the SAP2000 program in conducting Spectrum Response Dynamic Analysis for the building structure of the care-unit building of RSPN :

a) Defining the Spectrum Response based on the KRB and the KDS,
b) Earthquake load is defined as RS1 in the direction of x-axis and RS2 in the direction of y-axis, with scale factor the same as the gravity = 9.8 m/dt2,
c) Conducting joint constraint on the joints on each floor by defining it as Diapraghm Constraint,
d) Placing the mass at the weight point of each floor in the global direction of x- and y-axis. The calculation of weigth and mass of the structure is done separately using SAP2000 program by defining the loading combination of 100% Dead Load and 50% Live Load,
e) Defining the loading combination as follows:

$$COMB1 : 1.2DL+1.6LL \qquad (4)$$
$$COMB2 : 1.2DL+0.5LL+0.1875RS1+0.0563RS2 \qquad (5)$$
$$COMB2 : 1.2DL+0.5LL+0.0563RS1+0.1875RS2 \qquad (6)$$

Numbers 0.1875 and 0.0563 are the value of Ie/ R of 100% and 30% in accordance with the distribution of the earthquakes on each axis.

Modal analysis is conducted to find out structure dynamic character as well as the natural tremor periods. The calculation of modal analysis in SAP2000 results in the structure tremor periods.

The performance of the building serviceability limit is determined by the drift between each floor levels resulting from the planned earthquake effects, that is to limit excessive crackings on concrete buildings, as well as to prevent non-structure damage and inhabitants's discomfort. To calculate the requirements of serviceability limit performance, in whatever condition, the drift between each floor levels which is calculated from the drift of the building is not allowed to exceed $\Delta_a = 0.01\ h_{sx.}$

The analysis of the drift between each floor levels shows that the performance of the building structure meets the provision required.

Table 4. Drift in x-direction

Lantai ke-	hsx (mm)	Syarat □ max (mm)	□□ □mm □	Selisih □□□□ mm □	Cd	Ie	□ (mm)	Ket.
Lantai 2	5000	50	0,50	-0,37	5,5	1,5	-1,35	Ok
Lantai 3	4500	45	0,13	-0,05	5,5	1,5	-0,17	Ok
Lantai 4	4500	45	0,08	0,41	5,5	1,5	1,52	Ok
Lantai 5	4500	45	0,50	-0,43	5,5	1,5	-1,57	Ok
Lantai 6	4500	45	0,07	-0,04	5,5	1,5	-0,13	Ok
Atap	4500	45	0,03	0,00	5,5	1,5	0,00	Ok

6.1 Analysis of Portal Beam

Imran (2010) states that *SRPMK* (SMRF) elastic structure components has to meet detailing of requirements as follows:

a) The net span of the structure components is not allowed to be less than four times its effective height;
b) The ratio of the width to the the height of the structure component is not allowed to be less than 0.3;
c) The cross section width has to be $\geq$ 250 mm and $\leq$ the column width plus the space on each side of the column which is not more than three-fourth the flexible structure components;
d) Each reinforcement , both top and bottom, has to be greater than the equation of 7.8 and less than 0.025 (2.5%) and at least there has to be 2 reinforcements that is continuously fitted on both top and bottom reinforcements;

$$Asmin = \frac{0{,}25 * bw * d * \sqrt{f'c}}{fy} \quad (7)$$

$$Asmin = \frac{1{,}4 * bw * d}{fy} \quad (8)$$

e) The beam positive elastic strength on column front must be more than or the same with half its negative elastic strength;
f) The throughput joint has to be fitted with closed cross bars along the joint with 10-cm distance;
g) The throughput joint is not allowed to be placed in the beam-column connection (*HBK*) area and in the area which is two times the beam height on the column front;
h) The first cross bar has to be fitted not more than 50 mm from the support front;
i) The space of the cross bar is not allowed to be greater than d/4.8 times the elastic reinforcement diameter, 24 times the cross bar diameter, 300 mm

From the portal analysis result the deep forces in the beam elements are figured out, which then are used to do the design process especially reinforcement designs, in which the beam reinforcement configuration used is based on the design in SAP2000 using ACI design code by adjusting the reduction factor to the Concrete *SNI* of 2013. The reinforment design of the beam elements is based on the value of the maximum deep forces from the existing combinations.

The total torsional horizontal reinforcement as much as that of 3D19 reinforcement has been accommodated bythe beam longitudinal reinforcement in the strain area as musch as that of 4 D19, so that additional longitudinal reinforcement from the available configuration is not needed. It shows result similarity with that of the manual analysis.

6.2. Portal Column Analysis

Just like that of portal beam analysis, the columns of the structure of the Care-unit building of RSPN is designed based on the maximum forces from input combinations in SAP2000.In this review portal column manual analysis is also done to verify the reinforcement configuration output of the design in SAP2000 to show the appropriateness in using the SAP2000 design. The portal column used in the

manual analysis is selected from As C-7 portal, i.e. the 1582 column element.
The calculation of beam and column rigidity is applied to the whole elements on the same level with 1582 column.

In order to obtain a structure design which fits the principle of Capacity Design i.e. *Strong Column Weak Beam*, an analysis to the column based on the provision of Special Moment Resisting Frame system (SMRF/*SRPMK*) of 03-2012 *SNI* regulation (article 23.10) is conducted.

After conducting conventional calculation of the structure column, comparison is then done with the column design result using finite element method in SAP2000 program.

The comparison result of the value on the P-M interaction diagram on both colomn elements shows that design output in SAP2000 has the limitation of column capacity value which is close to the result of manual analysis, so that the analysis result in SAP2000 is relatively feasible.

6.3. Foundation Plan

The foundation of the building Structure of Care-unit building of RSPN is designed using PC square pile 25x25, 9 m long ex. A pile put into the ground using hydraulic-jack-in system. The consideration of choosing the type of foundation is based on the result of the soil test done in Rock and Soil Laboratory of *ITS*-Surabaya. The result of Cone Penetration Test in 3 (three) points shows that the hard soil layer is located in the depth of 6.50 meter with conus resistance (qc) of ±250 Kg/cm^2 with total friction of ±900 Kg/cm. The machine borring at BM-1 point shows that the hard soil is in the depth of -30,00 meter with the value of N-SPT > 60.

Table. 5. PC square pile Specifications

Type	:	TA 25x25
Concrete quality	:	K 500
Main Reinforcement	:	4 dia 16
Fy of main reinforcement	:	13 500 Kg/ cm2 (JIS G3536)
Cross bar reinforcement	:	Ø10
Fy of cross bar reinforcement	:	2 400 Kg/ cm2
P of allowability	:	45.00 Ton

6.3.1. Pile Allowability Bearing Capacity

The calculation of Pa for one pile will be reviewed from three soil bearing capacity calculation formula. According to Begemann, identical with Guy Sangrelat's opinion in Pamungkas and Harianti (2013)

$$Pa = \frac{qc*A}{3} + \frac{TF*O}{5} \qquad (9)$$

qc = Value of element resistance (kg/cm2)
A = Width of pile cross bar (cm2) = 625 cm2
TF = Total shear resistance (kg/cm')
3 & 5 = Safety Factor
O = 100 cm

$$\text{Pa} = \frac{250*625}{3} + \frac{900*100}{5}$$
$$\text{Pa} = 70\ 083\ \text{kg} = 70\ \text{Ton}$$

Meyerhoff (1956) in Pamungkas and Harianti (2013) suggested the formula to determine the bearing capacity of pile foundation based on the following data:

$$Pa = \frac{qc*Ap}{3} + \frac{\Sigma lifi*Ast}{5} \qquad (10)$$

qc = 40 N for sand, N is the value of N-SPT
A = 625 cm2 = 0.0625 m2
Li = 6.5 m
Fi = Shear force on the pile segment sleeve
Ast = Width of sleeve

$$\text{Pa} = \frac{40*37.5*0{,}0625}{3} + \frac{6.5*10*6.5}{5}$$
Pa = 115 Ton

According to the element requirements of *SRMPK*, each element is only allowed to bear axial load of 0.1 Ag*f'c. So, the cross bar of 0.25 x 0.25 multiplied by 0.1 multiplied by K500 equals 31 250 Kg or 31.25 Tons.

6.3.2.Total Number of Pile Analysis

To determine the need of piles at each column point, readings of support reaction divided by bearing capacity of one pile multiplied by pile group efficiency is done. Calculation output result of structure analysis (SAP 2000) is the value of reaction force on the support of each column based on dead loads (1.2DL + 1.6LL). The axial load existing is based on 70 x 70 cm column capacity,that is from loading combination resulted from the P-M diagram. The value is: P is 101.34 tons and M is 2.27 ton-meter. The bottom structure has to be design stronger than the top structure so that if there is failure on the top structure, the bottom structure does not fail as well. Therefore the load acts on the bottom structure is increased to a bigger number than that of the real load acting. This concept suits the principle of capacity design, so that the bottom structure is stronger than the top one. The planned load acting on the bottom structure becomes 1.05 times.

6.3.3. Subsidence Calculation

Subsidence Calculation is based on Poulus's and Davis's opinions (1980) in Pamungkas and Harianti (2013). Subsidence consists of sudden subsidence (S_i) and Consolidation Subsidence (S_c).

$$Stotal = Si + Sc \tag{11}$$

$$Si = \mu 1 * \mu 0 * \frac{qB}{Eu} \tag{12}$$

Si = Sudden Subsidence

μ1 = Correction Factor based on Janbu's, Bjerrum's, and Kjaernsli's opinions is 0.7

μ0 = 0,7

Q = 402 Ton/ (3.05*3.8) =34.7 T/ m^2

Eu = 400 Cu = 400

$$Si = 0{,}7 * 0{,}7 * \frac{402}{400}$$

Si = 0.49 mm

Sc = μd * σz * Sz * H

Sc = 0.19 * 0,44* 6 = 0,501 cm = 5.01 mm

Stotal = Si + Sc = 5.5 mm

6.3.4. Pile Cap Calculation

Pile cap functions to bond the piles into one unity and transfer the column load to the pile. Pile cap design is done by assuming that the pile cap is very rigid, the pile top end hangs on the pile cap so there is no elastic moment resulted from the pile cap to the pile, the pile is a short and elastic column so that tension distribution and deformation shape a plane. In this calculation, the pile cap with the biggest point load i.e. P20 with Pu = 402 Tons and Mu = 5.5 Ton-m is reviewed. The the followings are the technical data of the pile:

Σ biggest number of piles = 20 piles

Pile cap 25 x25 cm

Pile Cap dimension 3,05 m x 3.80 m

Required Pile space determination according to *PTUL* Department in Sarjono (1991):

Table 6. Pile space requirement from axis to axis

Pile space requirement from axis-axis	Space requirement from pile axis to pile cap edge
2.5 D < S < 4 D	S_1 > 1.25 D
2.5 * 25 < S < 4*25	S_1 > 1.25 *25
62.5 < S < 100	S_1 > 31.25
Determined S =75 cm	Determined S_1 = 40 cm

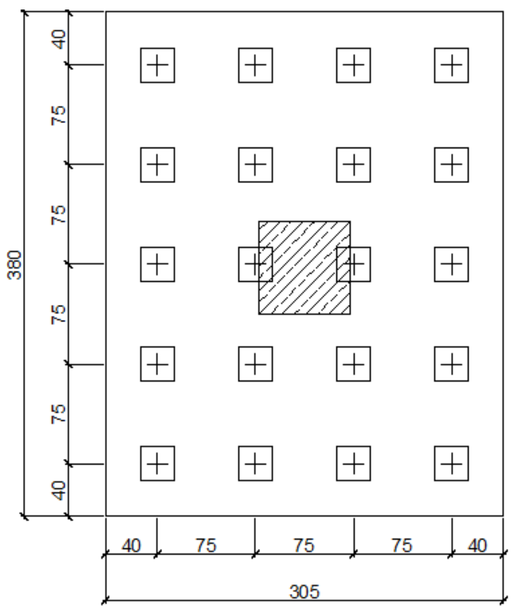

Figure 3. P20 Lay out

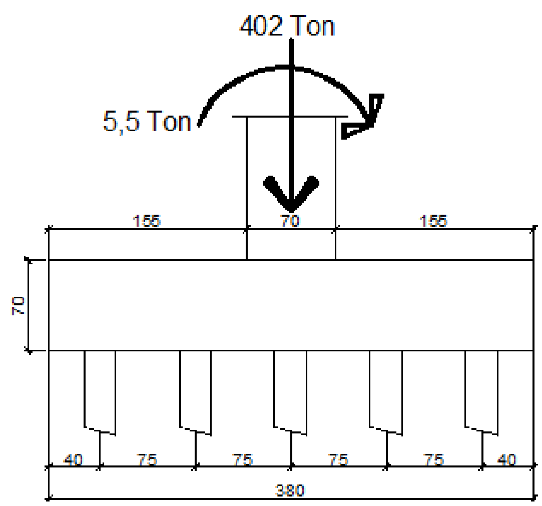

Figure 4. Pile cap thickness determination

The principle of SRPMK states that the negative design moment is not allowed to be less than half its positive moment, so the compressive reinforcement design is determined to be half its positive reinforcement D19-300 (13 rods are fitted) As 3 685 mm^2

From the calculation , the pile cap design is as described in figure 5

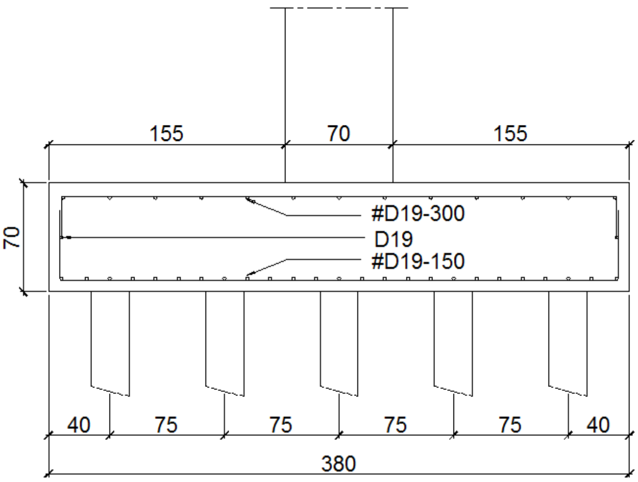

Figure 5. Pile cap design

6.4. Tie Beam

Pamungkas and Harianti (2013) state that the main function of tie beam is to tie the foundations so that if there is subsidence on the foundation, the subsidence can be restrained or happen at the same time; in brief it is expected that differential settlement can be minimized. Because of its function, the loading is determined by the function of subsidence between foundations and 10% of column axial force acting at the same time with the moment. In this plan D8-D9 joint is reviewed because this joint has the biggest foundation point load.

Based on tha calculation , the design of tie beam with 300x500 dimension and 7 D19 top reinforcement, 7 D19 bottom reinforcement, and Ø13-100 stirrup reinforcement.

7. Conclusion

a. The structure is reinforced concrete portal with concrete grade of f'c 25 MPa, thread reinforced steel grade of fy 400 MPa and plain reinforced steel of fy 240 Mpa. The floor plate is planned to be 12 cm thick with double reinforcement of # Ø10-150, the roof plate is planned to be 12 cm thick with double reinforcement of # Ø10-100, the stair plate is planned to be 15 cm thick with double reinforcement of # D16-150, the lift pit plate is planned to be 20 cm thick with double reinforcement of # D16-150.

b. The principal beam of As 1-2 and As 13-14 is planned to be 50x70 cm in size with reinforcement of M^- 16 D22+ 8 D22 stirrup reinforcement of 2 Ø13-75 reinforcement of M^+ 8D22 + 16 D22 stirrup reinforcement of 2 Ø13-150. The principal beam of As 3 to As 12 is planned to be 40x70 cm in size with reinforcement of M^- 12 D22+ 6 D22 stirrup reinforcement of 2 Ø13-75 reinforcement of M^+ 6D22 + 12 D22 stirrup reinforcement of 2 Ø13-150. The Joist spans of 4 m is planned to be 25 x 40 cm in size with reinforcement of M^- 3 D22+ 3 D22 stirrup reinforcement of Ø13-100 reinforcement of M^+ 3D22 + 3 D22 stirrup reinforcement of Ø13-150. The Joist spans of 4 m is planned to be 25 x 40 cm in size with reinforcement of M^- 3 D22+ 3 D22 stirrup reinforcement of Ø13-100 reinforcement of M^+ 3D22 + 3 D22 stirrup reinforcement of Ø13-150. Lift hanger beam is planned to be 25 x 40 cm in size with reinforcement of M^- 3 D22+ 3 D22 stirrup reinforcement of 1.5Ø13-100 reinforcement of M^+ 3D22 + 3 D22 stirrup reinforcement of 1.5Ø13-150. The tie beam is planned to be 25 x 40 cm in size with reinforcement of M^- 3 D22+ 3 D22 stirrup reinforcement of Ø13-100 reinforcement of M^+ 3D22 + 3 D22 stirrup reinforcement of Ø13-150.

c. Column-K1 of Floor-1 is planned to be 70 x 70 cm in size with principal reinforment of 40 D22 and stirrup reinforcement of 1.5 Ø13-100. Column K1of Floor-2 is planned to be 70 x 70 cm in size with principal reinforcement of 28 D22 and stirrup reinforcement of 1.5Ø13-100. Column-K1 of Floor-3 is planned to be 70 x 70 cm in size with principal reinforcement of 24 D22 and stirrup reinforcement of 1.5Ø13-100. Column-K1of Floor-4, 5, and 6 is planned to be 60 x 60 cm in size with principal reinforcement of 20 D22 and stirrup reinforcement of 1.5Ø13-100. Column-K2 of Floor-1 to 5 is planned to be 45 x45 cm in size with principal reinforcement of 16 D22 and stirrup reinforcement of 1.5Ø13-100.

d. The foundation is planned to use PC square pile of 25x25 in the form of pile group from P2 (2 piles) to P16 (16 piles). The Pile group is bonded with 70-cm thick pile cap with reinforcement of M^- D16-200 and M^+ D16-150. PC square pile grade of K500

8. Suggestions

a. The analysis result shows that the deep force on the column of floor-1 is significantly different from that of the other floors that it is suggested that in the future plan, a calculation of moment redistribution should be carried out.

b. The structure modeling for tie beam and stairs analysis is not modeled in portal model, the sloof is calculated to bear 10% of the column vertical load of the ground floor added up with the force resulted from the foundation subsidence (Pamungkas & Harianti, 2013), the stairs is modeled as point load, it is suggested that in the future plan it should be modeled in portal so that structure model behavior which is closer to the real one can be obtained.

c. The Analysis result shows that the beam at the edge of Axis 1-2 and Axis 13-14 has big shear force so that it is compensated to the beam width of the axis as wide as 50 cm, therefore it is suggested that in the future plan, modeling using shear wall on that particular axis is carried out. By using the modeling using shear wall on the axis, it is expected that the other structural dimensions wil become more economical.

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