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Re-planning of Concrete Structures in the Ngoro Dormitory Project in Surabaya

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Abstract

The current trend of building construction is using steel frames and reinforced concrete frames. The construction design concept is based on strength limit analysis (ultimate strength) which has sufficient ductility to absorb earthquake energy according to applicable regulations. In addition to this, it is also necessary to consider the economic aspect. All of these structural considerations will influence planning alternatives, such as column layout, beam length, and span. Dormitori Ngoro Surabaya is one of the low-rise buildings that will be redesigned to comply with SNI 2847:2013. This research is expected to answer the need for earthquake-resistant building construction using the ETABS program and according to SNI 2847:2013 standards. In this study the method used with the concept of Flowcharts in order to facilitate the research stage. As for this study, it produced a maximum moment of 137.58 KNm with the use of beam dimensions of 40 x 75 cm, using 8D16 top main reinforcement and 3D16 bottom main reinforcement with stirrups D10-200 in the support area and D10-100 in the field area. Whereas the ratio of beam reinforcement based on manual calculations is 0.24% and the ETABS results are 0.22%.

Keywords: Construction; design; earthquake; etabs; reinforcement

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1. Introduction

Earthquake disaster is a natural disaster event that often occurs in Indonesia (Indriasari, 2018). For example, the earthquake in Bantul, Yogyakarta in 2006 (Hamid, 2020), although the magnitude of the earthquake was not very large (Somantri, 2021; Siswanto & Salim, 2018), it caused extensive damage and claimed many victims (Maulina, 2017; Kusno, 2020). The earthquake that occurred in Bantul Regency, Yogyakarta is a lesson for us that we need to be prepared in facing earthquake disasters in the future (Irawan et al., 2022).

Sometimes the earthquake itself does not cause disaster directly to humans (Hadibroto & Ronitua, 2018). But treasures and objects (Chazawi, 2021). Besides that, there was also a structural collapse of modern buildings (Baso et al., 2017; Aristyawati et al., 2021). However, along with the development of technology (Azis, 2022; Azis et al., 2021) and progress (Azis et al., 2020), people's thinking patterns view technology as one of the main elements in planned development (Krisnani et al., 2017; N. Azis, 2021). Then the concept of earthquake-resistant buildings emerged among the public and building construction practitioners (Rumbyarso & Pribadi, 2022; Husein, 2016; Zega et al., 2022).

Surabaya is the second largest city after Jakarta besides that it is a potential place for investors (Harnindra et al., 2017). The city of Surabaya is passed by two active faults that have the potential for an earthquake measuring 6.5M (Farichah & Kumala Sari, 2019). Seeing this phenomenon, the construction of structures and infrastructure should pay attention to seismic aspects to avoid losses due to earthquakes (Simanjuntak, 2020). Seeing the phenomenon above,

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this research is focused on one of the buildings in the Surabaya area, namely Dormitory Ngoro Surabaya, which is one of the low-rise buildings that will be redesigned to comply with SNI 2847:2013 (Febriana et al., 2022)

2. Research Method

In the research method used is to use flow charts or flow charts. The flow chart method is a method that represents a process, system, or computer algorithm and is commonly used to document, plan, refine, or describe a workflow with many steps. The flowchart in this study is presented in Figure 1.

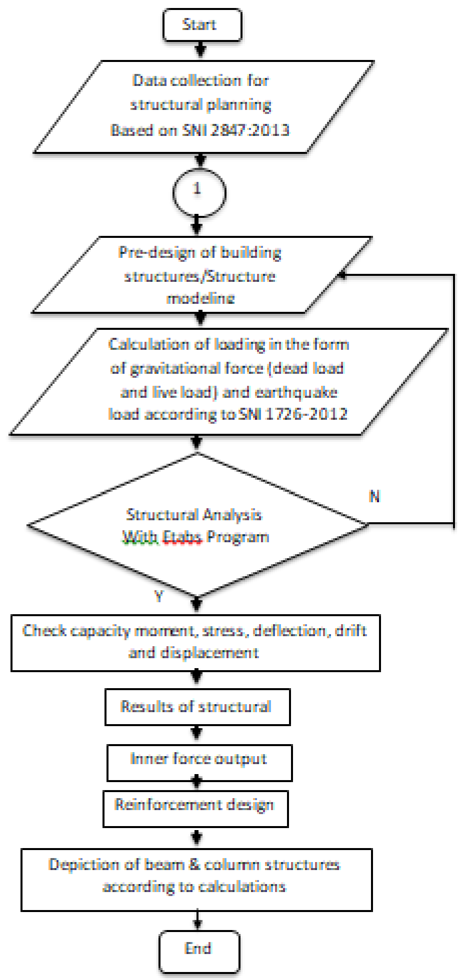

Fig 1. Research Flowchat

3. Result and Discussion

In the early stages, pre-planning calculations were carried out which were calculated manually to calculate the dimensions of structural plans such as slabs, beams and columns in order to obtain an optimal value. The calculation includes:

a. Pre Planned Plates
b. Pre Beam Plan
c. Pre Plan Column

Then proceed with loading. The structure that is carried out is:
a. Additional Dead and Dead Loads
b. Live Load
c. Earthquake Loading
d. Loading Combination

From the combined loading test, data is obtained as shown in table 1.

Table 1. Load Combination Calculation

Number	Combination	D	L	SIDL	SPECX	SPECY
1	U1	1.40	0	1.40	0	0
2	U2	1.20	1.6	1.20	0	0
3	U3-1	1.20	1	1.20	0	0
4	U5-1	1.29	1	1.29	0.3	1
5	U5-2	1.29	1	1.29	0.3	-1
6	U5-3	1.29	1	1.29	-0.3	1
7	U5-4	1.29	1	1.29	-0.3	-1
8	U5-5	1.29	1	1.29	1	0.3
9	U5-6	1.29	1	1.29	1	-0.3
10	U5-7	1.29	1	1.29	-1	0.3
11	U5-8	1.29	1	1.29	-1	-0.3
12	U7-1	0.81	0	0.81	0.3	1
13	U7-2	0.81	0	0.81	0.3	-1
14	U7-3	0.81	0	0.81	-0.3	1
15	U7-4	0.81	0	0.81	-0.3	-1
16	U7-5	0.81	0	0.81	1	0.3
17	U7-6	0.81	0	0.81	1	-0.3
18	U7-7	0.81	0	0.81	-1	0.3
19	U7-8	0.81	0	0.81	-1	-0.3

From the data obtained, the fundamental period is checked with the provisions imposed in SNI Earthquake 1727: 2013, then a comparison is made according to SNI requirements between the calculation fundamental period and the fundamental period from ETABS. The ETABS fundamental period can be obtained from displayà show table à modal information à modal participating mass ratios, here is the ETABS fundamental period data. As presented in Table 2.

Table 2. Mass Modal Period Data from ETABS

Mode	Period	SumUX	SumUY	SumUZ	RX	RY	RZ
1	1.989	84.729	0.002	0	0.002	99.106	0.277
2	1.801	84.737	84.456	0	98.461	0.009	0.633
3	1.662	84.998	86.086	0	0.813	0.293	85.072
4	0.654	95.495	86.086	0	0.000	0.339	0.037
5	0.607	95.498	96.065	0	0.501	0.000	0.097
6	0.556	95.535	96.164	0	0.004	0.001	9.933
7	0.377	99.146	96.165	0	0.000	0.250	0.018
8	0.362	99.149	99.306	0	0.213	0.000	0.040
9	0.330	99.167	99.346	0	0.002	0.001	3.186
10	0.280	99.984	99.352	0	0.000	0.000	0.006
11	0.277	99.984	99.989	0	0.003	0.000	0.007
12	0.251	100.000	100.000	0	0.000	0.000	0.695

Furthermore, the calculation of the deviation between floors in the X and Y directions was carried out and the data obtained as presented in Table 3.

Table 3. Calculation of Interfloor Deviation in X-Direction

STORY	Dynamic Earthquake	High (H) (mm)	Elevation Hx (mm)	DriftX (mm)	IZIN (0,02*H) (m)	DriftX*H(mm^2)	Δ (DriftX*H*Cd)(DriftX*H*4,5) (mm)	Status
LT.5	SPECX	4000	16000	0.00100	80	4.004	18.018	OK
LT.4	SPECX	4000	12000	0.00164	80	6.556	29.502	OK
LT.3	SPECX	4000	8000	0.00208	80	8.320	37.440	OK
LT.2	SPECX	4000	4000	0.00169	80	6.740	30.330	OK

Table 4. Calculation of Interfloor Deviation in Y-Direction

STORY	Dynamic Earthquake	High (H) (mm)	Elevation Hx (mm)	DriftY(mm)	IZIN (0,02*H) (m)	DriftY*H(mm^2)	Δ (DriftY*H*Cd)(DriftY*H*4,5) (mm)	Status
LT.5	SPECY	4000	16000	0.000745	80	2.980	13.410	OK
LT.4	SPECY	4000	12000	0.0001261	80	5.004	22.698	OK
LT.3	SPECY	4000	8000	0.00164	80	6.560	29.520	OK
LT.2	SPECY	4000	4000	0.00145	80	5.780	26.010	OK

Then continue the calculation of P-Delta in the X and Y directions As presented in Table 5 and Table 6.

Table 5. X-Direction P-Delta Calculation

STORY	Elevation Hx (mm)	INTERSTORY Drift Δ (mm)	floor load (total) (kN)	Cumulative load P (kN)	Seismic shear (VX)	Elevation, Hx (mm)	Ɵx (mm)	Ɵmax	Ɵlimit	Status
LT.5	16.0	18.018	3307.134	3307.134	1298215.730	16000	0.00	0.1111	0.25	OK
LT.4	12.0	29.502	3310. 478	6617. 612	2130552.820	12000	0.00	0.1111	0.25	OK
LT.3	8.0	37.440	2127. 312	8744.924	2775855.690	8000	0.00	0.1111	0.25	OK
LT.2	4.0	30.330	3769.840	12514.764	3222898.340	4000	0.00	0.1111	0.25	OK

Table 6. Y Direction P-Delta Calculation

STORY	Altitude (m)	INTERSTORY Drift Δ (mm)	Floor load (total) (kN)	Cumulative load P (kN)	Seismic shear (VY)	Elevation Hx (mm)	Ɵx (mm)	Ɵmax	Ɵlimit	Status
LT.5	16.0	13.410	3307.134	3307.134	1366378.530	16000	0.00	0.1111	0.25	OK
LT.4	12.0	22.698	3310. 478	6617. 612	2326924.810	12000	0.00	0.1111	0.25	OK
LT.3	8.0	29.520	2127. 312	8744.924	3072554.920	8000	0.00	0.1111	0.25	OK
LT.2	4.0	26.010	3769.840	12514.764	3559253.040	4000	0.00	0.1111	0.25	OK

Table 7. Earthquake-induced Shear Force in X-Direction

STORY	Column	Load Case Combo	Station m	P kN	V2 Kn	V3 Kn	T Kn-m	M2 Kn-m	M3 Kn-m
LT.5	C18	COMB2	0	-206.53	18.21	14.21	-0.15	26.89	32.47
LT.5	C18	COMB2	1.625	-195.06	18.21	14.21	-0.15	3.80	2.88
LT.5	C18	COMB2	3.25	-183.58	18.21	14.21	-0.15	-19.28	-26.71
LT.5	C18	COMB4	0	-210.15	17.39	14.07	-0.14	26.42	30.59
LT.5	C18	COMB5	0	-210.15	17.39	14.07	-0.14	26.42	30.59
LT.5	C18	COMB6	0	-210.15	17.39	14.07	-0.14	26.42	30.59
LT.5	C18	COMB7	0	-210.15	17.39	14.07	-0.14	26.42	30.59

Then to find out the maximum moment must know the beam forces. The data results from etabs the largest maximum moment is at the location of the 8 meter beam span with a moment of 137.58 kn.m in the field and 105.21 kn.m at the pedestal. For the maximum moment of the column column forces / column strength of the largest etabs on the 3rd floor with a column height of 4 meters the field moment is 883.37 kn.m and the pedestal moment is 32.21 kn.m. The obtained shear forces arising from the earthquake in the X direction and Y direction can be seen in Table 7 and Table 8.

From the results of the ETABS analysis, it can be seen that the shear force due to the X-direction earthquake is the column with code C18 located on the 5th floor or roof floor with a maximum load in the x-direction of the building of 18.21 Kn and the y-direction of 14.21 Kn.

Table 8. Earthquake-induced Shear Force in Y-Direction

STORY	Column	Load Case Combo	Station m	P kN	V2 Kn	V3 Kn	T Kn-m	M2 Kn-m	M3 Kn-m
LT.5	C18	COMB2	0	-150,95	4,46	**29,34**	-0,10	55,48	7,45
LT.5	C18	COMB2	1,625	-139,47	4,46	29,34	-0,10	7,79	0,21
LT.5	C18	COMB2	3,25	-128,00	4,46	29,34	-0,09	-39,89	-7,04
LT.5	C18	COMB4	0	-150,56	-5,15	29,08	-0,09	54,67	-10,18
LT.5	C18	COMB5	0	- 139,09	-5,15	29,08	-0,09	7,43	-1,81
LT.5	C18	COMB6	0	-127,61	-5,15	29,08	-0,09	-39,82	6,57
LT.5	C18	COMB7	0	-152,20	4,29	28,36	-0,09	53,28	7,05

From the results of the ETABS analysis, it can be seen that the shear force due to the Y-direction earthquake is the column with code C23 located on the 5th floor or roof floor with a maximum load in the y-direction of the building of 29.34 Kn and the x-direction of 4.46 Kn.

The last step is the Reinforcement of Beams and Columns

Beam Reinforcement Design 40x75

Details of the area of 40x75 beam reinforcement are presented in Figure 2.

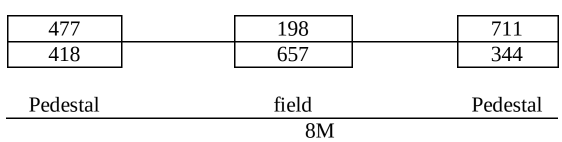

Fig 2. Detailed Area of 40x75 Beam Reinforcement

Deformed reinforcement (D16) is used, then:

Ace = ¼ xπx d2 = ¼ x 3.14 x 162 = 200.96 mm2

Deformed reinforcement (D13) is used, then:

Ace = ¼ xπx d2 = ¼ x 3.14 x 132 = 132.67 mm2

a. The main reinforcement of the support area:
   Upper reinforcement area = 477,amount of reinforcement = 477/200.96 = 2.4 ~ 3
   Bottom reinforcement area= 418,amount of reinforcement = 418/132.67 = 3.15 ~ 3
b. Field area main reinforcement:
   Upper reinforcement area = 198,amount of reinforcement = 198/132.67 = 1.49 ~ 2
   Bottom reinforcement area = 657,amount of reinforcement = 657/200.96 = 3.26~ 3

Using plain stirrup reinforcement diameter 8, then:

Ace = ¼ xπx d2 = ¼ x 3.14 x 102 = 78.5 mm2

a. Shear reinforcement area of support:

Assuming stirrups are used 2 D10-150
Area of reinforcement per 1 meter = 2 x ¼ d2 x 1000/150
Rebar area per 1 meter = 2 x ¼ . 3,14 . 102 x 1000/150 = 1046.66 mm2
So the area per meter of length = 1.046 mm2
Security control : Ldesign > Lplan = 1.046> 0.06 (OK)

b. Field area shear reinforcement:

Assuming 2D10-100 stirrups are used
Area of reinforcement per 1 meter = 2 x ¼ d2 x 1000/100
Rebar area per 1 meter = 2 x ¼ . 3,14 . 102 x 1000/100 = 1570 mm2
So the area per meter of length = 1.570 mm2
Security control : Ldesign > Lplan = 1.570> 0.07 (OK)

Based on SNI Concrete 03-2847-2013, SRPMM flexible structural components must meet the following requirements:

a. The factored axial force on the beam is limited to a maximum Ag x fc'/10
Check: (400x750) x 30 / 10 = 900 KN
From the ETABS calculation the axial force that occurs is 0.6470 KN, then:
0.6470 < 900 KN (OK)
b. A structural clear span of at least 4x the effective height
Effective height (d) = h – (SB + Dsengkang + ½ x Dutama)
= 750 – (50 + 10 + ½ x 16) = 682 mm x 4 = 2728 mm
Beam clear span = beam span – column dimensions
= 8000 – 500 = 7500 mm
Then 7500 > 2728 (OK)
c. The minimum width and height ratio is 0.3
b = 400 mm, h = 750 mm, b/h = 0.53
So, 0.53 > 0.3 (OK)
d. Element width must not be:
Less than 250mm
b = 400mm > 250mm (OK)
e. Longitudinal reinforcement requirements
The area of top and bottom reinforcement shall not be less than the minimum reinforcement requirements according to SNI 2847-2013:
As min = (0.25 x √fc')/(fy) xbxdt = (0.25 x √30)/(400) x400x682= 933.86mm
As min = (1.4 xbx s)/fy = (1.4 x 400 x 682)/400 = 954.8 mm
Based on the ETABS output:
Top reinforcement area = 711 mm
Area of bottom reinforcement = 344 mm
So: 198 + 657 = 1055 mm > 954.8 mm (OK)
f. Check reinforcement ratio
ρ = As/(bxd) = 1055/400x750 = 0.0035
ρb = β 0.85fc'/fy x 600/(600+fy) = 0.85 0.85x30/400 x 600/(600+400) = 0.03251
ρmax = 0.75 x ρb = 0.75 x 0.03251 = 0.0243 = 0.24%

The limit of reinforcement ratio used is 0.03251, then:
$\rho < \rho max$ and $\rho < 0.0243$ (OK)

From the calculation above, it is obtained:

a. Upper main reinforcement, (2D16) and lower main reinforcement (3D16)
b. The stirrup reinforcement used is 2D10-100 and 2D10-150

For the display of reinforcement can be seen in Figure 3.

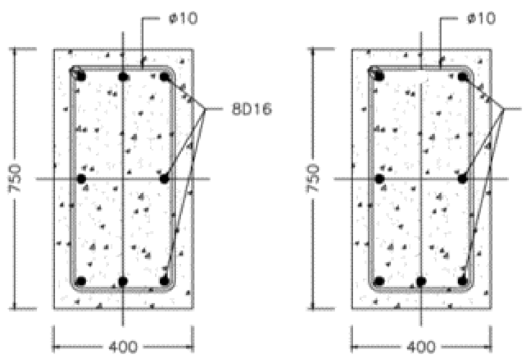

Fig 3. Details of Beam Reinforcement 40x75 cm

For the design of column reinforcement. The area of the main column reinforcement can be automatically determined by means of Design – Concrete Frame Design – Display Design Info – Longitudinal Reinforcing. The following is an image of a floor plan of one of the column types that will be analyzed and the reinforcement calculation will be included. Which can be seen in figure 4.

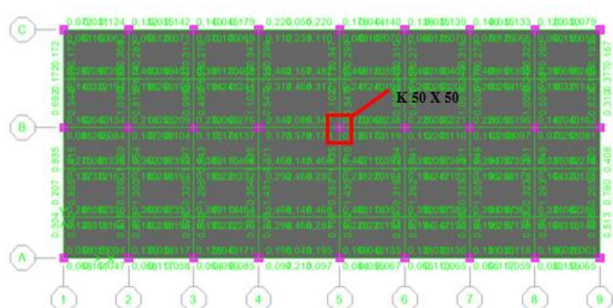

Fig 4. Field Support Reinforcement

Column dimensions : 500 x 500 mm
Concrete quality : 40 Mpa
Reinforcement quality : 400 Mpa
Diameter of longitudinal reinforcement : 20 mm
Diameter of transverse reinforcement : 10 mm
Concrete covers : 50 mm
Reinforcement Requirement As = % ETABS x Cross-sectional area = 1% x (500x500) = 2500 mm
Reinforcement Diameter Area = ¼ xπxd2 = ¼ x 3.14 x 222 = 379.94 mm2
Number of reinforcement = 2500/379.94 = 6.57 = 8 pieces

Total area of reinforcement = 8 x 379.94 = 2512 mm2 % Design = 2512/(250000) = 1.00048 %
As < Total Bar Area OK
2500 < 2512 OK
Reinforcement Needs = 8D22

From the calculation above, it is obtained:

a. Main reinforcement (8D22)
b. The stirrup reinforcement used is 2D10-150

Detailed Images of Column Reinforcement 50x50 cm can be seen in the figure 5.

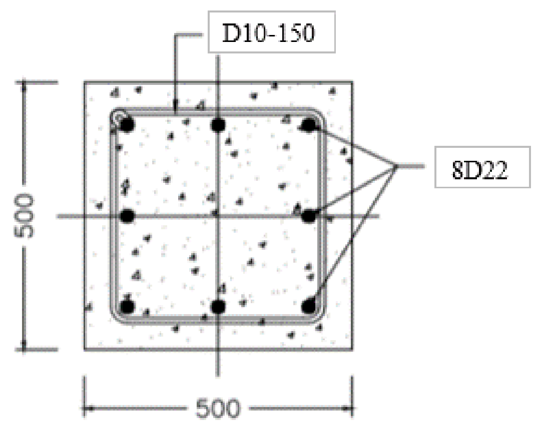

Fig 5. Column Reinforcement Details 50x50 cm

4. Conclusion

Based on the structural analysis results of ETABS V9.6.0 for columns, beams and slabs, it is obtained: 1). The structural design steps using ETABS and SNI standards start from the preliminary (Dimensional Planning), material specification planning, structural modeling, load calculations (dead loads, live loads and earthquake loads), and the final stage is conducting structural analysis. 2). Obtained loading analysis as follows: a). From the results of the ETABS analysis it can be seen that based on the dimensions of the beams and columns there is no over strength found, it can be said that they meet the SNI requirements. b). From the results of the ETABS analysis run in Shape 1 and Shape 2 modes, no torsion was found and a displacement of 0.29% was obtained. c). Based on the calculation, the maximum moment is 137, 58 KNm For beam dimensions 40x75 cm using 8D16 top main reinforcement and 3D16 bottom main reinforcement with stirrups D10-200 in the support area and D10-100 in the field area. d). The ratio of beam reinforcement based on manual calculations is 0.24% and 0.22% ETABS results. e). For columns with dimensions of 50x50 cm using main reinforcement 8D22 and D10-150 for stirrups. f). Column reinforcement ratio based on manual calculation is 1.00048% according to the results of 1% etabs. g). For slabs, use the same reinforcement dimensions as the existing one due to the use of bondek. h). Deflection of the beam at the longest span is obtained at 3.7 cm. So, 3.7 cm > 800/360 cm 3.7 cm > 2, 22 cm, in this case the height of the beam must be increased. 3). The column dimensions are 50x50 cm, the main beam dimensions at the longest stretch are 8 meters 40x75 cm, dimensions of the main beam on a stretch of 4 meters 30x50 and 25x40 cm joists. The thickness of the 2nd and 3rd floor plates is 150 mm while the 4th floor and roof floor are 120 mm.

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