CHAPTER II

Structural Design

II.1 Roof design

Designing the roof will require calculations of gording, truss and joints of the structure. The design process will use manual calculations and SAP 2000 program to analyze member forces and determine safety.

II.1.1 Purlin calculation

Purlin are the horizontal members of the roof. The horizontal members must meet certain buckling requirements so that it will satisfy the overall roof design. Designing the purlin will require data from the width of the roof and terrace. Angle of the roof will also be needed to determine the length of the angled roof (m) and terrace (n). Example of the calculation with 30º roof angle:

$$m=\frac{\frac{10}{2}+2}{\cos 30°}=8.0828 \quad n=\frac{8.0829}{5}=1.6164$$

After acquiring the length, I proceed with the load calculation. Using the data from the load calculation I can complete the load combination to find the total moment from all the loads. At this point, the purlin profile can be chosen depending on what the structure need.

There are three kinds of loading on the roof, dead load, live load and wind load. Purlin will receive dead loads from roof tiles, purlin itself, hanger, and ceiling, which will then be summarized as qd. The live load (ql) only consist of human load= 1 KN. Wind load (qw) will depend on the wind pressure and will require value of wind coefficient= 0.02α – 0.4 with α as the roof angle.

With the loads summed up, proceed to compute the moment. Moment will be important to the load combination formula to find the total moment on each axis (x and y). The moment itself must be calculated depending on the axis, so the α can be computed either with sin or cos. Here are the formula for dead load, live load and wind load: $Mdx = \frac{+1}{8} * qd * \cos\alpha * lx^2$, $Mdy = \frac{+1}{8} * qd * \sin\alpha * ly^2$ for dead and wind (wind load use qw instead of qd) and $Mlx = \frac{+1}{4} * P * \cos\alpha * lx$, $Mly = \frac{+1}{4} * P * \sin\alpha * ly$ for live, all use the same purlin span value (l). Wind load (qw) can be computed from: qw = *wind coefficient* * *wind pressure* * 1.6164.

Once the data has been acquired, it can be inserted into these load combinations:

Mux = 1.2 (Mdx) + 1.6(Mlx) + 0.8(Mwx)

Muy = 1.2 (Mdy) + 1.6(Mly) + 0.8(Mwy)

Continue to strength calculations. Depending on purlin profile, elements of the data can be different such as inertia, Fy, and the area to count a few. If the profile is already confirmed, then start center off mass calculation. After that, value of a_x or a_y (centroid of x and y) can be computed. Repeat these equations on both areas to obtain Z_x and Z_y value.

$y = \frac{2 * a}{2}$ $Z = \frac{A}{2} * a$ value a can be a_x or a_y

Then I must determine whether the chosen profile is compact or non compact by using the formulas below. The data for the formula will vary depending on what steel type was chosen.

$\lambda = \frac{bf}{2 * Tf}$ $\lambda_p = 0.38 * \sqrt{\frac{E}{Fy}}$

The value of λ will determine if the profile is compact or non compact. If it's $\lambda \leq \lambda_p$, the profile is compact. Compact profile needs to be checked for the lateral torsional buckling at x and y axis. The value of l_p, l_b and l_r are needed to satisfy this equation $l_p < l_b \leq l_r$. With it, it can be decided if it is inelastic LTB or elastic. These will be the formulas to find their values:

$l_p = 1.76 * ry * \sqrt{\frac{E}{F_y}}$ with l_p being the maximum unbraced strength.

l_b being the maximum unbraced length of compression flange.

$l_r = \frac{1.95 * rts * E}{0.7 F_y} * \sqrt{\frac{E}{F_y} + \sqrt{\left(\frac{J_c}{S_x * k_0}\right)^2 + (6.76) * \left(\frac{0.7 F_y}{E}\right)^2}}$ with l_r being limiting laterally unbraced length for the limit state.

$$rts^2 = \frac{\sqrt{I_y * C_w}}{S_x}$$

If it's canal, the C equation will be: $C = \frac{h_0}{2} * \sqrt{\frac{I_y}{C_w}}$

Then find the lateral torsional buckling at x and y direction. If $l_p < l_b \leq l_r$, it's inelastic lateral torsional buckling so that the value of M_n:

$$M_n = C_b Mp - \left((Mp - 0.75 * F_y * S) * \left(\frac{L_b - L_p}{L_r - L_p} \right) \right) \leq M_p \text{ with } Cb = 1.3$$

where $M_p = F_y * Z$. It will be safe if $M_n > M_p$

If it's elastic, then it has no lateral torsional buckling, therefore the $M_n = M_p$. If $\phi M_n > M_u$ ($\phi = 0.9$), then the torsional buckling will be safe.

II.1.2 Truss calculation

Truss are the vertical members of the roof structure. It must pass strength test to determine the overall safety of the design. To start the calculation, the loads must be determined. Computing the loads will use the exact same method as purlin design. The load types will be dead, live and wind load. But, the dead load will also include the additional purlin weight.

After calculating the loads, I draw the truss design in SAP 2000 so I can retrieve load data for each members. Here are the load combinations I use in SAP 2000:

COMB 1 = 1.4 DL

COMB 2 = 1.2 DL + 1.6LL

COMB 3 = 1.2DL + 1.6LL + 0.8WR

COMB 4 = 1.2DL + 1.6LL + 0.8LL

COMB 5 = 1.2DL + 1.6WL + 0.8LL

COMB 6 = 1.2DL + 1.6WR + 0.8LL

Data from SAP 2000 will let us know which member experiences tension or compression. Depending on the chosen profile for tension and compression member.

1. For tension member safety, check:

$$\frac{L}{r} < 300$$

Data from the member profile such as A_g, C_x, F_y, F_u, and e are needed verify tension members on their yield stress (ϕRn).

Find A_e

$$A_e = A_g * u * 2u = 1 - \left(\frac{C_x}{e}\right)$$ where e is welding length

Based on yield stress : $\phi R_n = 0.9 * A_g * 2 * F_y$, Where F_y value depends on steel type

Based on fracture : $\phi R_n = 0.75 * A_e * 2 * F_u$, Where F_u value depends on steel type

If $\theta R_n > P_u max$, then it's safe. $P_u max$ is the tension force at that member.

2. For compression member safety, check:

$$\frac{L}{r} < 200$$

Proceed with checking the thickness ratio, if the $\lambda_r > \lambda$, then the profile isn't slender.

$$\lambda = \frac{h}{t}, \lambda = \frac{b}{t}, \lambda_r = 0.45 * \sqrt{\frac{\delta}{Fy}}$$

Then continue with taking longest and highest member for the safety check. P_u will be the benchmark to satisfy the $\phi P_n > P_u$ equation. P_u is the compression force at that member. The formulas are:

$\frac{K * L}{r}$, where L is member length and K = 1

Find the value of $F_e = \frac{\pi^2 * E}{\left(\frac{K * L}{r}\right)^2}$

If $\frac{K * L}{r} > 4.71 * \sqrt{\frac{E}{F_y}}$, then use $F_{cr} = 0.877 F_e$

If $\frac{K*L}{r}<4.71*\sqrt{\frac{E}{F_y}}$, then use $Fcr=0.658^{\left(\frac{Fy}{Fe}\right)}*Fy$

$P_n=F_{cr}*A_g$, then find θP_n with θ value of 0.9. If the ϕP_n > P_u, then member is safe.

II.1.3 Bolt connection calculation

To design a bolted connection, I must decide on how thick the gusset plate is and the bolt diameter. From there, I can calculate the shear strength, spacing, bearing strength, and how many bolts I need. Force data from SAP 2000 will also be used for the design. The formula needed to calculate these are:

1. Shear strength of a bolt

$A_b=\frac{1}{4}*\pi d^2$ and then find $R_n=F_{nv}*A_b$

2. Spacing of the bolt

Minimum spacing in any direction: $S=2\frac{2}{3}d$, hole diameter: $h=d+\frac{1}{16}$

3. Bearing strength of the bolt

$l_c=l_e-½h$, where l_e is the minimum distance from the edge to the nearest bolt

$R_n=1.2*S*t*F_u$ (in between holes area)

$R_n=1.2*l_e*t*F_u$ (tear out area)

4. Gross tension

$P_n=F_y*A_g$, with F_y and A_g data depend on the bolt type

5. Bearing (all holes)

R_n = 2.4 * dt * F_u, with dt and F_u data depend on the bolt type

6. Net tension

$P_n=F_u*A_e$

7. Block shear

$R_n=0.6*F_u*A_{nv}+u*F_u*A_{nt}$ and $R_n=0.6*F_y*A_{gv}+u*F_u*A_{nt}$

Then, use ϕR_n > P_u to prove that it's safe.

II.2 Stairs and bordes calculation

Stairs calculation will include the design for the reinforcement and bordes calculation. SAP 2000 will be used to model and inspect the moment and safety of the structure. Then, the finished design will be drawn in Autocad (Appendix A.1).

II.2.1 Stair calculation

1. Stair design

O_p (step height) and A_n (step length) value must be determined. The values can be adjusted between these two so it will satisfy α (slope of the stairs).

$n=\left(\frac{H}{O_p}\right)-1$ where n is the number of steps

$O_p=\frac{H}{n+1}$ where H is interstorey height.

Control = 600 < (2 * Op + An) < 650

$\tan\alpha=\frac{Op}{An}$, the value of α must be between 25° < α < 45°.

After calculating the dimensions of the steps, I pick the bordes and stairs' plate thickness = 140 mm. I tried with other thinner and thicker thickness, but this gives me the best result per its thickness.

$t^1=\frac{0.5*Op*An}{\sqrt{Op^2+An^2}}\; h^1=\frac{tt+t^1}{\cos\alpha}$

2. Stair loading

Loading per 1 meter of stair width will consist of dead load (DL) and (LL). Dead load will be the load summation of plate, stairs, tile, spesi and railing. Live load according to SNI 1727:2013 for office building, 4.79 KN/m ≈ 5 KN/m.

3. Determining minimum plate thickness

$h_{min}=\frac{L}{20}*\left(0.4+\frac{F_y}{700}\right)$, both bordes and stairs use exact same formula to find the plate thickness.

4. Determining the effective plate height

To determine effective plate height use: $d = h - p - \emptyset - ½\emptyset$. Both bordes and stairs use exact same formula to find the plate height.

5. Determining the stair reinforcement

$$Rn_{need} = \frac{Mu}{0.9 * bw * d^2}$$

$$P_{needed} = \frac{0.85 * f_c^1}{f_y} * \left(1 - \sqrt{1 - \frac{2R_n}{0.85 * f_c^1}}\right)$$

$As_{needed} = P_{needed} * bw * d$

$As_{min} = 0.0018 * b_w * h$,

If $As_{need} > As_{min}$, use As_{need} value.

$$S_{needed} = \frac{A_s * b_w}{As_{used}}$$

$S_{shrinkage} = 200$ mm

$$As_{shrinkage} = \frac{b_w}{S_{shrinkage}} * A_s > As_{min}$$

II.2.2 Bordes calculation

Bordes and stairs calculation has same formulas regarding load calculation, reinforcements and plate design. It only needs to add concrete connection calculation:

1. Strength of the connection

$$\phi V_c = 0.75\left(\frac{1}{6} * \sqrt{f_c^1} * bw * d\right) * 10^{-3}$$

2. V_s value limit:

$$\frac{1}{3} * \sqrt{f_c^1} * bw * d \text{and} \frac{2}{3} * \sqrt{f_c^1} * bw * d$$

3. Finding crossbar distance:

First, find the Vs_{max},

$Vs_{max} = \frac{Vw_{max} - Vc}{\phi}$, then find the distance with these 4 equations:

$$S_1 \leq \frac{A_v * f_y * d}{Vs_{max}},\ S_2 \leq \frac{d}{2},\ S_3 \leq 300\,mm, S_4 \leq \frac{16 * A_v * f_y}{b_w * \sqrt{f_c^1}}, S_5 \leq \frac{3 * A_v * f_y}{b_w}$$

II.3 Slab calculation

Slab design must include calculation of the loading, plate thickness, plate types, slab thickness and reinforcements. Reinforcement calculations are the same as previous calculations. Refer to previous formulas to design the reinforcements for the slab.

Loading of the slab (W_u) will include dead load and live load. Dead load (DL) consists of slab, tiles, spesi and ceiling weight. Live load according to SNI 1727:2013 for office building, 4.79 KN/m ≈ 5 KN/m. Then, insert both loads to combination: $W_u = 1.2D + 1.6L$.

The plate thickness formula is:

$$h_{min} = \frac{\ln\left(0.8 + \frac{f_y}{1500}\right)}{36 + 5\rho\left(\alpha_m - 0.2\right)} \text{ for } \alpha f_m < 2.0,\quad h_{min} = \frac{\ln\left(0.8 + \frac{f_y}{1500}\right)}{36 + 9\beta} \text{ for } \alpha f_m \geq 2.0$$

where $\alpha = \left(\frac{E_{cb} * I_{cb}}{E_{cp} * I_{cp}}\right) * \alpha_{fm}$ is the average of α from every side of the plate.

To determine if the plate is one direction or two direction, use following equations: $One\ direction\ plate: \frac{l_y}{l_x} > 2, Two\ direction\ plate: \frac{l_y}{l_x} < 2$. This is important for reinforcement design.

Calculate the moment ultimate using these formulas: 1. $M_{ux} = 0.001 * q * l_x^2 * x$

2. $M_{uy} = 0.001 * q * l_y^2 * y$

II.4 Earthquake calculations

Refer to *Hitungan Beban Gempa* SNI 1726:2012 (Appendix A.15) for the detailed step by step process. Building will be located at Yogyakarta, spectrum data at that location can be acquired from http://puskim.pu.go.id/Aplikasi/desain_spektra_indonesia_2011/.

II.5 Beam reinforcement planning

From ETABS, data for M_u will be found for both types of beams (support and main). As for the reinforcement design, here are the formulas:

$$R_n = \frac{M_u}{\phi * b * d^2}$$

$$\rho_{min} = \frac{1.4}{F_y}, \rho_{need} = \frac{0.85 * f_c^1}{f_y} * \left(1 - \left(\sqrt{1 - \frac{2 * R_u}{0.85 * f_c^1}}\right)\right), \rho_{max} = \frac{0.429 * 0.85 * f_c^1}{f_y}$$

If $\rho_{need} > \rho_{min}$, use, ρ_{need}. If it isn't, use ρ_{min} value. After that, find number of reinforcement (n) needed using: $As_{need} = bw * d * \rho_{need}$ then use $n_{reinf} = \frac{As_{need}}{As_{reinf}}$. You can choose the reinforcement profile, then verify its distance (x) with: $x = \frac{b - ((n * cover) + (n * d_{stirrup}) * (n * d_{reinf}))}{n - 1} > d_{reinf}$.

Start checking the $\phi M_n = 0.9 * T_s * z * 10^{-6} > M_u$. Then calculate space:

$V_c = \frac{1}{6} * \sqrt{f_c} * b * d$, then $V_s = \frac{V_u}{\phi}$, then check $V_{c2} = \frac{1}{3} * \sqrt{f_c^1} * b * d > V_s$

Then, proceed to determine $S = \frac{A_v * f_y * d}{V_s}$ where A_v is stirrup area.

II.6 Column reinforcement plan

Decide on the column dimension first to find A_g. Then design the column and retrieve these column data from ETABS P_u, M_{ux}, M_{uy}, V_u, f_c^1, f_y and E_s. These data may vary depending on reinforcement steel type and concrete type. Use IKOLAT ($\phi = 0.65$) to find $\rho = 2.722\%$. Begin calculating the $Mu_{equivalent}$, N_{od}, M_{od}:

$$Mu_{equivalent} = M_{uy} + M_{ux} \frac{b}{h} * \left(\frac{1 - \beta}{\beta}\right), N_{od} = \frac{P_u}{f_c^1 * b * h} \text{ and } M_{od} = \frac{M_u}{f_c^1 * b * h^2}$$

Calculate $As_{total} = \rho * A_g$ and As_{reinf} to find number of reinforcement (n) with reinforcement diameter will be taken from the ETABS column design.

Then proceed to check the distance (x) and find $d = h - d^1$.

Estimate the column against shear = $V_c = \left(1 + \frac{M_u}{14 * A_g}\right) * \left(\frac{\sqrt{f_c}}{6}\right) * b * d$

Check the stirrup shear strength = $V_s = \frac{V_u}{\phi} - V_c$, then check $\phi V_c > V_u$

Finally, check S_{max} = ½ d. Using these data determine the needed stirrup profile and how many are needed.

Proceed to check requirement for "strong column weak beam" using beam and slab data: bw beam, h beam, beam and slab reinforcements, slab thickness and primary beam A_s and A_s^1.

Use smallest B_e value: $b_{e1} = \frac{1}{4} * l_b, b_{e2} = bw + (8 * t), b_{e3} = 0.5(l_b - bw)$

Then find y with d from beam field reinforcement:

$$y = \frac{As_{beam} * \left(40 + 10 + d + \frac{d}{2}\right) + \left(5\frac{\pi}{4} * 10^2\right) * \left(13 + \frac{10}{2}\right) + \left(2\frac{\pi}{4} * 10^2\right) * \left(140 - 13 - \frac{10}{2}\right)}{As_{beam1}}$$

$d_{above} = h - y,\ d_{bottom} = h - cover - 10 - \frac{d}{2}$

Find M_g value for field and support reinforcement each then summarize:

$$q = A_s\left(\frac{1.25 * f_y}{0.85 * f_c * b}\right), M_g = \phi * A_s^1 * f_y * \left(\frac{d - a}{2}\right), \Sigma M_g = M_{gprimary} + M_{gsecondary}$$

Then check $\frac{\Sigma M_e}{\phi} > \frac{1.2 * \Sigma M_g}{\phi}$

II.7 Foundation plan

Foundation plan require data from the support reaction: $P_u + Live\ Load$, column, soil and concrete specific weight. Then assume the values of: foundation thickness and soil depth to find summation of soil and foundation weight, allowing this formula to compute: $\sigma_{nett} = allowable\ stress - total\ weight$ with allowable stress = 220 KN/m^2. This allow calculation of: $A_{need} = \frac{P_u}{\sigma_{nett}}$, then $b = h = \sqrt{A}$, then check $A = b * h > A_{need}$.

Start checking the soil bearing capacity: $q_u = \sigma = \frac{P_u}{A} < \sigma_{soil}$, σ_{soil} = 220 KN/m^2

Check one way shear: $V_u = q_u * (B * x)$. Then proceed with one way shear strength (V_c) calculation, refer to section **II.5** for the formula and requirement.

Two way critical cross section: $b_0 = 2 * ((c_1 + d) + (c_2 + d))$

Two way factored shear force: $V_u = P_u * (b^2 - (c_1 + d) * (2 + d))$

Two way concrete shear strength:

$$V_{c1} = \left(1 + \frac{2}{\beta_c}\right) * \left(\frac{\sqrt{f_c}}{6}\right) * b_0 * d, V_{c2} = \left(\left(\frac{\alpha_s * d}{b_0}\right) + 2\right) * \left(\frac{\sqrt{f_c^1}}{12}\right) * b_0 * d,$$

$V_{c3} = \frac{1}{3} * \sqrt{f_c^1} * b * d$ where α_s = 30.

Then choose the smallest V_c value that meet the requirement $\phi V_c > V_u$.

Reinforcement calculations (ϕ = 0.9)

$$M_u = \frac{1}{2} * q_u * x^2, K = \frac{M_u}{\phi * b * d^2}, \text{then} \rho_{need} = \frac{0.85 * f_c^1}{f_y} * \left(1 - \left(\sqrt{1 - \frac{2 * K}{0.85 * f_c^1}}\right)\right)$$

Refer to section II.**5** to calculate number of reinforcements (n) and spacing (S). Use biggest value of A_s for S calculation. For shrinkage, $As_{shrinkage}$ = ½ A_s.

II.8 Conclusion

Summary of the overall building design:

1. Roof : Purlin = C125 * 50 * 20 * 2.3
   Truss = 2L 70 * 70 * 6
2. Bolt Connection : Gusset plate = 10 mm
   Bolt diameter = 5/8 inch
3. Stairs and bordes :

<u>First/bottom stair</u>

Stair "field" reinforcement = D13 – 50; P10 – 200

Stair "support" reinforcement = D13 – 200; P10 – 200

<u>Second/top Stair</u>

Stair "field" reinforcement = D13 – 200; P10 – 200

Stair "support" reinforcement = D13 – 500; P10 – 200

<u>Bordes</u>

Beam "field" and "support" reinforcement = D13 – 500; P10 – 200

4. Slab : 5 types (3 two direction and 2 one direction plate)
P10 – 250

5. Secondary beam (250*400) : 3 types (A,B,C) top and bottom 2D16; 2D10–150
1 type (D) top 2D16, bottom 4D16; 2D10–150
1 type (E) top and bottom 4D16; 2D10–150

6. Primary beam (300*500)

(2nd floor) : Beam 1 end: Top 5D16, bottom 3D16; 2D10–80
Beam 1 mid: Top and bottom 3D16; 2D10-200
Beam 2 end: Top 6D16, bottom 3D16; 2D10-200
Beam 2 mid: Top and bottom 3D16; 2D10-200

(3rd floor) : Beam 1 end: Top 4D16, bottom 3D16; 2D10–80
Beam 1 mid: Top and bottom 3D16; 2D10-200
Beam 2 end: Top and bottom 4D16; 2D10-200
Beam 2 mid: Top and bottom 4D16; 2D10-200

(4th floor) : Beam 1 end: Top 5D16, bottom 3D16; 2D10–80
Beam 1 mid: Top and bottom 3D16; 2D10-200
Beam 2 end: Top 5D16, bottom 4D16; 2D10-200
Beam 2 mid: Top and bottom 4D16; 2D10-200

7. Column (500*500) : Main 24D19; Top, middle, bottom 2P10-200

8.Foundation : P1 = 12D13-250; 12D16-250
P2 = 20D13-200; 27D16-150

Drawn results will be attached in the appendix (Appendix A).