Partial Prestress Concrete Beams Reinforced Concrete Column Joint Earthquake Resistant On Frame Structure Building

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Abstract. Floor Building that requires a large space such as for the meeting room, so it must remove the column in the middle of the room, then the span beam above the room will be long. If the beam of structural element with a span length reaches 15.00 m, then it is less effective and efficient using a regular Reinforced Concrete Beam because it requires a large section dimension, and will reduce the beauty of the view in terms of aesthetics of Architecture. In order to meet these criteria, in this design will use partial prestressing method with 400/600 mm section dimension, assuming the partial Prestressed Beam structure is still able to resist the lateral force of the earthquake. The design of the reinforcement has taken into account to resist the moment due to the gravitational load and lateral forces. The earthquake occurring on the frame structure of the building. In accordance with the provisions, the flexural moment capacity of the tendon is permitted only by 25% of the total bending moment on support of the beam, while the 75% will be charged to the reinforcing steel. Based on the analysis result, bring in 1 (one) tendon contains 6 strand with diameter 15.2 mm. On the beam pedestal, requires 5D25 tensile reinforcement and 3D25 for the compression reinforcement, for shear reinforcement on the pedestal using Ø10-100 mm. Dimensional column section are 600/600 mm with longitudinal main reinforcement of 12D25, and transverse reinforcement Ø10-150. At the core of the beam-column joint, use the transversal reinforcement Ø10-100 mm. The moment of Column versus Beam Moment ΣMe > 1.2 Mg, with a value of 906.99 kNm > 832.25 kNm, qualify for ductility and Strong Columns-weak beam. Capacity of contribution bending moment of Strand Tendon’s is 23.95% from the total bending moment capacity of the beam, meaning in accordance with the provisions. Thus, the stability and ductility structure of Beam-Column joint is satisfy the requirements of SNI 2847: 2013 and ACI 318-11.

1. Introduction
The service load on the Building Frame Structure, consisting of live and dead load, temporary load consists of lateral load are the earthquake or wind. The calculated temporary load is one of the two because usually the earthquake and wind load rarely work together at the same time. In buildings requiring large spaces with beam lengths exceeding 10.00 m, when using the usual Concrete Beam Structure, will require large dimensional section beams that are less efficient, and when viewed in terms of Architecture becomes less aesthetic. Therefore for the Structure Beam design it is better to use Partial Prestressed Concrete Beam Structure. Some previous researchers who have used Partial Prestressed Beam on BuildingFrame Structure, among others: Naaman (1985), Partial Prestressed Concrete (Review
and Recommendation), Raka IGP, et al (2014), State of the art Prestressed Partial Structure-Column Concrete Building Structure, in high zone Earthquake, Astawa MD, et al (2016), Shear Behavior and ductility Partial Prestressed Beam -Column Concrete Reinforcement connections on Building structure with Cyclic lateral load, and several other authors are not mentioned here.

2. Material and Method
The structural analysis methodology is described according to the process as follows:

2.1. Material of Concrete
Compression Strength of concrete using $f'_c = 37$ MPa, Stress-Strain relationship of concrete Compression of $f'_c$ 37 MPa caused by press load. Based on the explanation of R8.5 ACI 318-11, the inelastic stress occurs after $0.45 f'_c$ from the elastic state as shown in the following table:

Table 1. Inelastic Stress of Concrete $f'_c$ 37 MPa.

| STRUCTURE ELEMENT | $f'_c$ (MPa) | 0.45 $f'_c$ |
|-------------------|-------------|-------------|
| Beam-Column Joint | 37          | 16.65       |

Once calculated, the stress-strain curve relationship of concrete $f'_c$ 37 MPa, as the Figure 3.1. following.

![Image of stress-strain curve relationship](image)

Figure 1. The Stress-Strain curve relationship of concrete $f'_c$ 37 MPa.

2.2. Dimension of Prestressed Beam Section.

\[
c = \frac{h}{2} + t; \quad y_t = \frac{(h_{\text{plate}} + \frac{t}{2}) (h_{\text{beam}} - w)}{A_{\text{total}}}; \quad d_l = y_t - \frac{t}{2}; \quad d_p = y_b - \frac{h - t}{2};
\]

\[
I = \frac{1}{12} b h^3 + (A_{\text{balok}} x d_p^2) + \frac{1}{12} \frac{b x}{n} t^3 + A_{\text{pelat}} x d_t^2
\]

\[
W_t = \frac{I_{\text{komposit}}}{y_t}; \quad W_b = \frac{I_{\text{komposit}}}{y_b}; \quad K_t = \frac{W_t}{A_{\text{total}}}; \quad K_b = \frac{W_b}{A_{\text{total}}}
\]

2.3. The Partial Prestressed Concrete Requirement Analysis.
Requirements of Partial Prestressed Concrete is the ratio of the sum of Strand Tendon usage with non Prestressed reinforcing, with the equation: $(\omega + \omega_p - \omega_p') \leq 0.30$.

2.4. Preliminary Design of Prestressed and Regional Limit Cable.

Initial Prestressed force $F_i = \frac{M_T}{0.65h}$. The limit region of the cable other than limited by kern on the beam is also limited by the $a_{\text{min}}$ and $a_{\text{max}}$. The values are: $a_{\text{max}} = \frac{M_T}{F_g}$ and $a_{\text{min}} = \frac{M_G}{F_o}$

2.5. Deflection Limit.
Deflection permits on prestressed concrete components shall be eligible for Table 9.5 (b) SNI 2847: 2013 part 9.5.3.1, that is: $\Delta_{\text{lin}} = \frac{L}{480}$

2.6. Analysis Cracks moment of Beams.
The ultimate strength calculation of the prestressing beam shall meet the requirements of SNI 2847: 2013 concerning the total amount of non-prestressed reinforcing and the prestress shall be sufficient to
produce a rated load of at least 1.2 cracking loads based on the crack modulus value of $0.7 \sqrt{f'_c}$, Obtained $\phi M_n \geq 1.2 M_C$, with value $\phi = 0.85$.

2.7 Beam-Column joint Analysis.
Calculates possible shear forces at x-x distance, $V_{xx} = T1 - V_h$; Calculating shear forces in concrete: $\phi V'_c = 0.75 \frac{1}{1000} \left(1.25, \sqrt{f'_c}, b, h\right)$; So: $V_{xx} < \phi V_c$.

3. Results and Discussion
3.1. The Beam Design Data.
Dimension cross section beams = 400/600 mm, plate thickness = 120 mm, beam length of span = 15.00 m, Concrete Pressure Strength ($f'_c$) = 37 Mpa, Steel yield stress ($f_y$) = 350 MPa, column height = 3.40 m. The beam design is a T beam so that the floor plate is monolith with beams.

3.2. Dimensional Sectional Beams.
Calculate the section area of the beam as: $A_{plate} = b_a \times t = \frac{136 \times 12}{1} = 163200 \text{ mm}^2$
$A_{beam} = b_a \times (h-t) = 40 \times (60-12) = 192000 \text{ mm}^2$; $A_{total} = A_{plate} + A_{beam} = 355200 \text{ mm}^2$
The static value of the central grade of section the beam is: $c = \frac{h}{2} + t = \frac{600}{2} + 120 = 420 \text{ mm}$
$y_i = 204.6 \text{ mm}; y_b = 600-204.6=395.4 \text{ mm}; d_i = \frac{t}{2} = 204.6 - \frac{120}{2} = 144.6 \text{ mm};$
$W_{b} = \frac{h \times t^3}{12} = \frac{600 \times 120^3}{12} = 14085493632 \text{ mm}^4$
The resistance moment on upper region: $W_t = 68844055 \text{ mm}^3$; on bottom region: $W_b = 35623403 \text{ mm}^3$
Upper Kern limit: $K_t = \frac{W_t}{A_{total}} = \frac{35623403}{355200} = 100 \text{ mm}$; Bottom Kern limit: $K_b = \frac{W_b}{A_{total}} = \frac{68844055}{355200} = 194 \text{ mm}$
Figures of beam section at midspan and at the end support, see Figure 2 and 3 below.

3.3. Calculating Moments.
3.3.1. Before Composite
Once calculated with help of the soft ware SAP 2000, yields $MG = 239.76 \text{ kNm}$
3.3.2. After Composites
The combination of loads using the dead and live load factor = 1.0 (1.0 DL + 1.0 LL), results the support moment = 268.04 kNm, and field moment = 393.27 kNm.

3.4. Prestressed Concrete Partial compute requirements.
The tensile and pressure reinforced ($As$) and ($A's$) are tried using diameter 25; 1 D25 with $As = 490.9 \text{ mm}^2$, $f_y = 350 \text{ MPa}$ and wide area 1 Strand tendon ($A_{ps}$) = 143.3 mm2
Strand Ratio to reinforcement: $(\omega + \omega_p - \omega') \leq 0.30$
$\omega = \omega' = \frac{\rho f_y}{f'_c} = \frac{(A_{ps} / b_d) f_y}{f'_c} = 0.021; \omega_p = \frac{\rho_p f_{ps}}{f'_c}; \rho_p = \frac{(\rho_{ps} / b_d)}{f'_c} = 0.002; \omega_p = \frac{\rho_p f_{ps}}{f'_c} = 0.09$
$(\omega + \omega_p - \omega') = (0.021 + 0.09 - 0.021) = 0.09 \leq 0.30$……… (OK)

3.5. Calculating Prestressed Force.
Design Prestressed Force : $F = \frac{M_{midspan}}{0.65h} = \frac{393.27}{0.65 \times 0.6} = 1008.38 \text{ kN}$
3.6. Design of Tendon limit region.
Tendon limit regions other than bounded by kern on the beam are limited by \( a_{\text{max}} \) and \( a_{\text{min}} \) values are:
\[
a_{\text{max}} = \frac{M_T}{F} = \frac{393.27}{(1008.38).10^3} = 390 \text{mm}; \quad a_{\text{min}} = \frac{M_G}{F_o} = \frac{(239.76)}{(150000)} = 0.1598 \text{ m} = 159.8 \text{ mm}.
\]

![Figure 4. Tendon Limit region on Beam.](image)

3.7. Calculate the Number of Strand
The Strand Tendon data is taken from the VSL table as follows:

- Using data from the VSL strand properties table to AS-1311 for post tensioning.
- Includes the type of uncoated low relaxation strand.
- The nominal diameter is used at 15.2 mm with a nominal wire area of 143.3 mm².
- Minimal breaking load 250 KN.

\[
f_{\text{ps}} = \frac{250000}{143.3} = 1744.592 \text{ Mpa.}
\]

The licence stress for prestressing where the tensile force of the tendon force is taken is the smallest value between 0.94fpy, 0.80fpu and 0.7fpu, so that the fpu and fpy values can be calculated as follows: 0.7fpu = 0.7 x 1744.592 =1221,214 Mpa. Fetched = 1221 Mpa (rounded)

Calculated the amount of strand area required to produce the prestress force \( F = 1008.36 \text{ kN} \).

Number of Strand Requirements: \( A_{ps} = \frac{1008.36.10^3}{1221} = 825.85 \text{ mm}^2 \); \( n = \frac{825.85}{143.3} = 5.7 \approx 6 \) pieces (rounded).

3.8 Loss of Prestressed.
Calculating loss categorization based on short-term (instantaneous) loss occurs shortly after the execution of Stressing, long term according to time. After calculating each of these losses are included in the Table as follows.

| Table 2. Loss of Prestressed |
|-----------------------------|
| No. | LOSS OF PRESTRESSED | PERCENTAGE LOSS OF PRESTRESSED (%) |
|-----|----------------------|----------------------------------|
| I   | Short-Term Loss (Instantaneous) |                                      |
| 1   | Due to the Elastic Shortening of Concrete Material | 0                                   |
| 2   | Due to Slip Anchor when Insert Strand | 5.49                                |
| 3   | Due to column confinement | 0.31                                |
| II  | Long-term loss by time |                                      |
| 1   | Due to Tendon Friction (Woble Efect) | 5.37                                |
| 2   | Due to Relaxation (Fatigu) Strand Tendon | 0.008                               |
| 3   | Due to concrete creep under pressure | 2.23                                |
| 4   | Due to the shrinkage of concrete material | 2.98                                |
| Total Los of Prestressed | 16,388                          |

The result of Prestressed loss calculation:16,388 % < 20,0 %....... (OK)

3.9 Deflection
a. Allowable deflection.
   Deflection permitted on the block is: \( \Delta_{\text{allow}} = \frac{L}{480} = \frac{15000}{480} = 31.25 \text{ mm} \).

b. The deflection is happening.
   - Early deflection when jacking: \( \Delta_l = \Delta_{l_P} + \Delta_{l_m} + \Delta_{l_Q} = -33.78 + 7.99 + 16.46 = -9.33 \text{ mm} \), to bottom deflection has not yet occurred.
   - Total deflection that occurs.

   When the external load is working, the prestress force has been fully working effectively \( =1053.84 \text{ kN} \). So after the total calculated deflection:
   \( \Delta_l = \Delta_{l_P} + \Delta_{l_m} + \Delta_{l_Q} = -26.04+6.68+7.621=11,739 \text{ mm} < 31,25 \text{ mm} \).

3.10 Calculating Moment of cracks.
The ultimate strength calculation of the prestressing beam according to the total amount of non-prestressed reinforcing and Strand Tendon shall be capable of producing a load of at least 1.2 crack loads.
occurring based on the crack modulus value of 0.7 $\sqrt{fc}$, thus obtaining $\phi M_n \geq 1.2 M_C$, with Value $\phi = 0.85$.

Because in this paper focus on Block-Beam Column Structure then the moment is calculated only at the moment of the beam pedestal. Cracked moment calculation result $M_C = 257593373,2$ Nmm, so: $\phi M_n = 1.2 \times M_C = 1.2 \times 257593373,2 = 309112047,8$ Nmm.

3.11 Reinforcing Calculate and Strand Moment.

Maximum moment on the beam due to a combined of Gravity and Lateral Load of earthquake calculated with help software of SAP 2000, amounted to 446.42 kNm. Delivery of loads in accordance with the provisions of SNI 2847: 2013 and ACI 318-11 section 21.5.2.5 (c), minimum be imposed to reinforcement is $\frac{1}{4}$ (75%) and maximum by Strand $\frac{1}{4}$ (25%). Then the load be imposed to reinforcement = 75% X 446.42 = 334,815 kNm.

Dimension cross section 400/600 mm, d’ taken 60 mm so that effective height $d = 600 - 60 = 540$ mm.

$$M_n\text{require} = \frac{M_u}{\phi} = \frac{334,815}{0,8} = 418,52 \text{ kNm}.$$ $$R_n = \frac{M_{Mn\text{perlu}}}{b.d^2} = 3,59; m = \frac{f_y}{0,85.f_c} = \frac{350}{0,85.37} = 11,13.$$ $$\rho_b = \frac{0,85.f_c.b}{f_y} = 0,037; \rho_{max} = 0,75 \times 0,037 = 0,028; \rho_{min} = \frac{1,4}{350} = 0,004;$$

$$\rho_p = \frac{1}{m} \left( 1 - \frac{1}{m} \frac{2.m.Rn}{f_y} \right) = \frac{1}{11,13} \left( 1 - \frac{2.11,13 \times 3,59}{350} \right) = 0,011$$

As = $\rho_b$. b. d = 0,011. 400. 540 = 2376 mm², used 5D25 with As = 2454,8 mm² ... (OK)

Using a pressure Reinforcement As’ = 60%. As = 60% (2376) = 1425.6 mm².

Used 3 D25 with As there = 1472,60 mm² ... (OK)

Moment be imposed to Strand Tendon = 25% X 446.42 = 111.60 kNm, Sectional Area 6 Strand Aps = 6 X 143.3 = 859.8 mm². Transverse reinforcement in the support beam using $\odot$8-100mm.

3.12 Bending Capacity of Reinforcement and Strand.

3.12.1 Reinforcement bending Capacity.

High of Concrete Block Stress: $a = \frac{T}{0,85.f_c} = \frac{2454,8 \times 350}{0,85 - 37} = 30$ mm.

$\phi M_n = \phi[T \ (d-a/2)] = 0,8[2454,8 \times 350 (540-30/2) 10^{-6}] = 360,86 \text{ kNm}$

3.12.2 Strand Tendon bending Capacity.

High of Concrete Block Stress to Tendon used: $a_p = \frac{T_p}{0,85.f_c} = \frac{859,8 \times 1221}{0,85 - 37} = 34$ mm.

In order for the moment's contribution to the Tendon not to exceed $\frac{1}{4}$ the total moment capacity of the Beam, the Tendon moment arm (Zp) shall be arranged in such a way: $Z_p = \frac{M_p}{\phi T_p} = 135$ mm.

Effective high beam: $d_p = Z_p + a_p/2 = 135 + 34/2 = 152$ mm. $dp' = h - d_p = 600 - 152 = 448$ mm. So, $\phi Mpn = \phi[T_p (Z_p)] = 0,8[859,8 \times 1221 (135) \times 10^{-6}] = 113,66$ kNm

Total Bending Moment Capacity = $\phi M_n + \phi Mpn = 360,86 + 113,66 = 474,52$ kNm > 446,42 kNm... (OK)

Percentage capacity of Tendon Moments to total Moments: $\frac{113,66}{474,52} \times 100\% = 23,95 \% < 25 \% ...$ (OK).

3.13. Column Design.

Property data as follows:

Pressure strength of Concrete: $fc' = 37$ MPa; Steel yield Stress $f_y = 350$ MPa

Dimension section of Column 600/600 mm

Column Height = 3.40 m

The main reinforcement uses D25

Transversal hoops = $\odot$10

Analysis assisted PCA COL program, so obtained 12D25 results as follows: and shear reinforcement on column mounted shear reinforcement $\varphi$ 10-150 (as Figure 5)
3.14. Beam-Column Joint Design

One of the ductility requirements of the Beam-Column joint Structure is the Strong Column-weak Beam, which means the bending capacity of the Column must be greater than the Beam that composes the Column. Results of analysis of SAP 2000 software, the number of moments in the column $Me = 906.99$ kNm. To find out if the Column Moment is greater than the Moment on support Beam, the following will calculate the magnitude of the Moments on the Beam pedestal based on the mounts of the mounts attached.

a. Strong Column-Weak Beam Requirements Control.

Calculating the magnitude of the beam bearing moment ($M_g$):

On the upper side (tensile reinforcement): Donations of Reinforcement, $M^+_g = T(\frac{d - a}{2})0.8$; $a = 30$ mm, then: $M^+_g = 2454.8 \times 350 \times 540 - \frac{30}{2} \times 0.8 \times 10^{-6} = 360.86$ kNm; Donations of Strand Tendon:

$$M^+_g = T(\frac{d - a}{2})0.8 = 859.8 \times 1221 \times 152 - \frac{34}{2} \times 0.8 \times 10^{-6} = 113.38$ kNm.

On the bottom side (Pressure reinforcement): There are 3 D25 with section area $A_s' = 1472.60$ mm$^2$

$$M^-_g = T(\frac{d - a}{2})0.8; a = \frac{T}{0.85f_{c'}} = 16.40$ mm. $M^-_g = 1472.6 \times 350 \times 540 - \frac{1.64}{2} \times 0.8 \times 10^{-6} = 219.3$ kNm

Total $M_g = M^+_g + M^+_g + M^-_g = 360.86 + 113.38 + 219.30 = 693.54$ kNm. Strong Column-Weak Beam Requirement: $\Sigma Me \geq 1.2 \Sigma M_g$; $906.99 \geq 1.2 \times 693.54$, produce: $906.99 > 832.25$ … (qualify)

b. Transverse reinforcement design at the core of Beam-Column joint.

The Beam-Column joint analyzed is the Exterior Beam-Column Connection, because this Structure in the middle of span without columns, so that the meeting of the Beam-Column connection is directly on the Exterior Column. In this connection towards weakening to beam, the plastic hinge are extruded out of the beam-Column connection joint by increasing the transverse reinforcement at the junction joint parallel to the beam axis. The transverse strand is calculated based on the shear forces acting on the connection core. The shear forces that occur on x-x core connection pieces are as follows:

$$V_{xx} = T_1 - V_h = 831.69 - 121.78 = 709.91$ kN. The magnitude of $V_{x-x}$ shall be compared with the nominal shear strength at the side of the joint as set forth in article 23.5.3 of SNI 2847: 2013. $\alpha V_c = 0.75/1000 (1.25 \sqrt{f_{c'.}} b h) = 0.75/1000 (1.25 \sqrt{37}) 400.600 = 1368.62$ kN > 709.91 kN, means that the structure at the core of the joint strong is enough, so that the shear reinforcement in the core area does not need to be calculated theoretically, but the installation of transverse reinforcement is still done with $\varnothing 10-100$ mm.
4. Conclusion

The conclusion of the Analysis of Beam-Column joint Structure as follows:

a. Non-prestressed reinforcement on beam, are 5 D25 tensile reinforcement and 3 D25 pressure reinforcement, Strand Tendon uses 6 pieces. This result is already qualified as Partial Prestressed Beam because the reinforcement ratio with Strand: \((\omega + \omega_p - \omega') = 0.09\) according to the provision \((\omega + \omega_p - \omega') = 0.09 \leq 0.30\).

b. Bending Load Capacity \(\phi M_n=360,86kNm\) and Strand \(\phi M_pn=113,66 kNm\), total bending capacity \(M_n= 474,52 kNm > Mu = 446,42 kNm\).

c. Flexure capacity contribution Strand's of 23.95% < 25% of the total bending capacity of the beams meets the requirements of SNI 2847: 2013 and ACI 318-11 section 21.5.2.5 (c).

d. The total deflection occurring 11.379mm < allowable deflection of \(L / 480 = 31.25 \text{ mm}\), and the cracking moments occurring are also smaller than the allowable, the Structure is eligible.

e. The number of main reinforcing bars of the Column is 12 D25, with the number of moments \(\sum M_e = 906.99 kNm > 1.2 \text{ total bearing support moment, in accordance with } \sum M_e > 1.2 Mg (906.99 kNm > 832.25 kNm)\).

f. Shear Strength at the core of the beam-Column joint is very good, so for the installation of horizontal transversal reinforcement at the connection cores made practically \(\varnothing 10-100 \text{ mm}\), means the structure is very strong.

5. References

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