DAFTAR PUSTAKA

Abdullah D., Takiguchi, K., 2003, *An Investigation into the Behavior and Strength of Reinforced Concrete Columns Strengthened with Ferrocement Jackets*, Cement Concrete Composite 25, pp.233-242.

ASCE Standard 41-17 Seismic Evaluation and Retrofit of Existing Buildings (Virginia: American Society of Civil Engineers)

Badan Standarisasi Nasional Indonesia, *SNI 2052-2015 Baja Tulangan Beton,* BSN, Jakarta.

Badan Standarisasi Nasional Indonesia, *SNI 1726-2012 Tata Cara Perencanaan Ketahanan Gempa Untuk Gedung dan Non Gedung,* BSN, Jakarta.

Badan Standarisasi Nasional Indonesia, *SNI 07-0663-1995 Jaringan Kawat Baja Las Untuk Tulangan Beton*, BSN, Jakarta.

Badan Standarisasi Nasional Indonesia, *SNI 7834:2012 Metode Uji Dan Kriteria Penerimaan Sistem Struktur Rangka Pemikul Momen Beton Bertulang Pracetak Untuk Bangunan Gedung*, BSN, Jakarta.

Bayrak, Sheik S A, 2002, *Response Prediction by Plastic Hinge Analysis Technique*, Dep.of onCivil Engineering, University of Toronto, Ontario, Canada.

Bayrak O., Sheikh S.A., 1997, *High Strength Concrete Columns under Simulated Earthquake Loading*, ACI Journal V.94 No.6, pp. 708 – 722.

Bett, B.J, Klingner, R.E, Jirsa, J.O, 1988, *Lateral Load Response of Strengthened and Repaired Reinforced Concrete Columns*, ACI Structural Journal, Vol. 85 (5), pp. 499–508.

Biskinis D, Fardis M., 2009, *Deformations of Concrete Members at Yielding And Ultimate UnderMonotonic or Cyclic Loading (Including Repaired and Retrofitted Members)*, Advanced Centre of Excellence in Structural And Earthquake Engineering, European Commision.

Eduardo N.B.S, Fernando A.B, Vitor D.S, 2005, *Reinforced Concrete Jacketing Interface Influence on Monotonic Loading Response*, ACI Structural Journal, Vol.102 (2), pp. 252–257.

EFNARC (2005). *The European Guidelines for Self-Compacting Concrete*: Specification,Production and Use, UK (www.efnarc.org), May, 2005.

Ersoy U, Tankut A.T, Suleiman R, 1993, *Behavior of Jacketed Columns*, ACI Structural JournalV.90, pp. 288–293.

Ghobarah A., Attar M.E., Aly M.M., 2000, *Evaluation of Retrofit Strategis for Reinforced Concrete Columns: a Case Study*, Engineering Structures 22 (2000), pp. 490-501.

Goodnight J., Kowalsky M.J., Nau J.M., 2016, *Modified Plastic-Hinge Method for Circular RC Bridge Columns*, Journal Structure Engineering ASCE, pp. 1 – 12.

Green M.F, Bisby L.A, Fam A.Z, Kodur V.K, 2006, *FRP Confined Concrete Columns: Behaviour Under Extreme Conditions*, Cem. Concr. Compos. 28 (10), pp. 928 – 937.

Irsyam M., Sangara I.W., Aldiamar F., Widiyantoro S., Triyoso W., Natawidjaja D.H., Kertapati E., Meilano I., Suharjono, Asrurifak, Ridwan M., 2000, *Ringkasan Hasil Studi Tim Revisi Peta Gempa Indonesia,* Bandung.

Jiang C., Wu Y.F., Wu G., 2014, *Plastic Hinge Length of FRP-Confined Square RC Columns, Journal Composite Construction*, Journal Composite Structure ASCE, pp. 1 – 12.

Kadir A, Satyarno I, Suhendro B, Triwiyono A, 2016, *Perilaku Kolom Yang Diperkuat KombinasiWire Rope dan Wire mesh*, Prosiding Seminar Nasional Teknik Sipil 2016, Fakultas Teknik Universitas Muhammadiyah Surakarta.

Karimah R., Wahyudi Y., 2010*, Daktalitas Kolom Beton Bertulang Dengan Pengekangan Di Daerah Sendi Plastis*, Jurnal Jurusan Teknik Sipil, Universitas Muhammadiyah Malang.

Ma C.K., Apandi N.M., Yung S.C.S., Hau N.J., Haur L.W., Awang A.Z., Omar W., 2016, *Repair and Rehabilitation of Concrete Structures Using Confinement A Review*, Elsevier Journal of Construction and Building Materials, 133 (2017), pp. 502 – 515.

Nawy E.G, 2008, *Reinforced Concrete : A Fundamental Approach***,** Prentice Hall, Pearson Education, New York, USA.

Okamura H., Ouchi M., 2003, *Self Compacting Concrete*, Journal of Advanced Concrete Technology Vol. 1 No.1, pp. 5 – 15.

Park H, Eom T, 2004, Energy Dissipation Capacity of Reinforced Concrete Members, CTBUH Research Paper, October : pp. 378 – 384.

Park R, Paulay T, 1975, Reinforced Concrete Structures, Jhon Wiley, New York.

Park H.G., Yu E.J., Choi K.K., 2012, Shear Strength Degradation Model for RC Columns Subjected to Cyclic Loading, Engineering Structures (34), pp. 187 – 197.

Panagiotakos T.B., Telemachos B, Fardis M.N., 2001, *Deformations of Reinforced Concrete Members at Yielding and Ultimate*, ACI Structural Journal, V.98(2), pp. 135 – 148.

Parung H., 2012., *Seismic Design of Building*, Badan Penerbit UNM, Makassar.

Paulay T., Priestley M.J.N., 1992, *Seismic Design of Reinforced Concrete and Masonry Structures*, John Wiley & Sons, Inc.

Priestley M.J.N., Park R., 1987, *Strength and Ductility of Concrete Bridge Columns Under Seismic Loading*, ACI Structural Journal, V.84 (1), pp. 61 – 76.

Priestley M.J.N., Verma R., Xiao Y., 1994, *Seismic Shear Strength of Reinforced Concrete Columns*, Journal of Structural Engineering ASCE, 120(8), pp. 2310 – 2329.

Priestley M.J.N., Calvi G.M., Kowalsky M.J., 2007, *Displacement Based Seismic Design of Sructures*, IUSS Press, Pavia, Italy.

Priestley M.J.N., Seible F., Calvi G.M., 1996, *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, Inc, Canada.

Sarno L.D, Elnashai A.S, Nethercot D.A, 2006, *Seismic Retrofitting of Framed Structures withStainless Steel*, J. Constr. Steel Res. 62, pp. 93–104.

Saadatmanesh H., Ehsani M.R., Li M.W., 1994, *Strength and Ductility of Concrete Columns Externally Reinforced with Fiber Composite Straps*, ACI Structural Journal, July – August,pp. 434 – 447.

Saatcioglu M., 2003, *Seismic Retrofit of Reinforced Concrete Structures*, Journal of Seismic Assessment and Rahabilitation of Existing Building, pp. 457 – 486.

Seible F., Priestley M.J.N., Hegemier G.A., Innamorato D., 1997, *Seismic Retrofit Of RC Columns with Continuous Carbon Fiber Jackets*, Journal of Composites for Construction,May, pp. 52 – 62.

Williams M.S., Villemure I., Sixsmith R.G., (1997), Evaluation Of Seismic Damage Indices for Concrete Elements Loaded in Combined Shear and Flexure, ACI Structural Journal, May – June, pp. 315 – 322.

Wu R.Y., Pantelides C.P., 2017, Rapid Repair and Replacement of Earthquake Damaged Concrete Columns Using Plastic Hinge Relocation, Elsevier Journal of Composite Structures.

Wu Y.F., Griffith M.C.,Oehlers D.J., 2003, Improving the Strength and Ductility of Rectangular Reinforced Concrete Columns through

Composite Partial Interaction: Tests, Journal of Structural Engineering ASCE, September, pp. 1183 – 1190.

Wu G, Lu Z.T, Wu Z.S, 2006, Strength and Ductility of Concrete Cylinders Confined with FRP Composites, Constr. Build. Mater. 20 (3), pp.134–148.

Wu Y.F., Liu T., Wang L., 2008, Experimental Investigation on Seismic Retrofitting of Square RC Columns by Carbon FRP Sheet Confinement Combined with Transverse Short Glass FRP Bars in Bored Holes, Journal of Structural Engineering ASCE, January – Febuary, pp. 53 – 60.

Xiao Y., Wu H., 2003, Retrofit of Reinforced Concrete Columns Using Partially Stiffened Steel Jackets, Journal of Structural Engineering ASCE, June, pp. 725 – 732.

Yuan F., Wu Y.F., 2017, Effect of Load Cycling on Plastic Hinge Length in RC Columns, Elsevier Journal of Engineering Structures 147 (2017), pp 90 – 102.

Yuce S.Z., Ercan Y., Yilmaz B., Kivanc T., Karadogan F.H., 2007, Local Thin Jacketing For The Retrofitting of Reinforced Concrete Columns, Struct. Eng. Mech. 27 (5) (2007) pp. 589 – 607.

LAMPIRAN

Perakitan Tulangan dan Pembuatan Bekisting

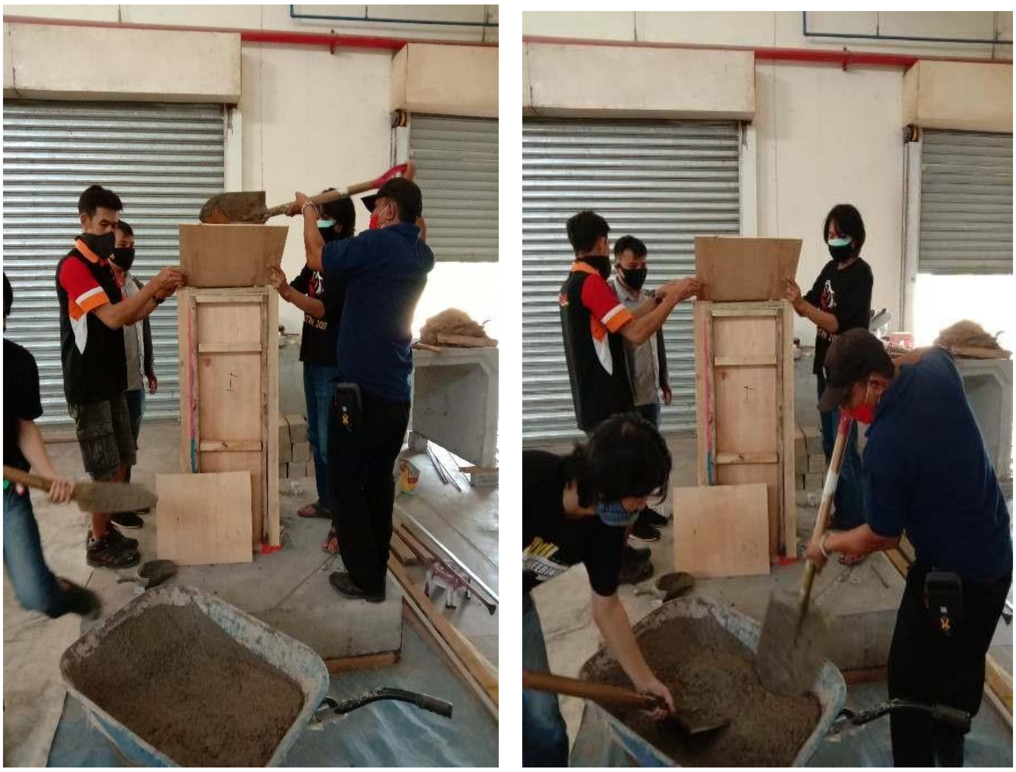
Pemasangan Strain Gauge Baja

Persiapan Pengecoran

Proses Pengecoran Benda Uji

Perawatan Benda Uji

Pemasangan *Wire mesh* dan Bekisting Retrofit



Pengecoran Retrofit

Setting Up Benda Uji

Pengujian dan Pola Retak Benda Uji

Tim Kolom Retrofit

Preliminary Design

1. Desain Penampang Kolom

Penampang kolom :

B = 300 mm

H = 300 mm

d = 275 mm

d_s = 25 mm

f'_c = 25 MPa

f_y = 474 MPa

E_s = 200000 MPa

Regangan tarik baja tulangan pada saat leleh (ε_y) :

$$\varepsilon_y = \frac{f_y}{E_s}$$

$$= \frac{473{,}744}{200000}$$

$$= 0{,}00237$$

Luas penampang bruto kolom (A_g) :

A_g = B.H

= 300.300

= 90000 mm^2

Luas tulangan terpasang (A_{st}) :

$A_{st} = n.0{,}25.\pi.D^2$

$= 8.0{,}25.\pi.13^2$

= 1061,32 mm^2

Persyaratan :

Berdasarkan SNI 2847-2019 Pasal 10.6.1 tulangan longitudinal terpasang harus lebih besar dari 0,01.A_g dan lebih kecil dari 0,08.A_g :

$0{,}01.A_g \leq A_{st} \leq 0{,}08.A_g$

$900 \leq 1061{,}32 \leq 7200 \rightarrow$ Syarat terpenuhi

Inersia penampang bruto :

$$I_g = \frac{1}{12} b.h^3$$

$$= \frac{1}{12} 300.300^3$$

$$= 675000000 \text{ mm}^4$$

Nilai momen inersia penampang kolom harus dihitung berdasarkan ketentuan dalam SNI 2847-2019 Pasal 6.6.3.1.1 dengan persamaan :

$$I = 0{,}70.I_g$$

$$= 0{,}70.675000000$$

$$= 472500000 \text{ mm}^4$$

Radius girasi (r) :

$$r = \sqrt{\frac{I}{A_g}}$$

$$= \sqrt{\frac{472500000}{90000}}$$

$$= 72{,}46$$

Persyaratan kelangsingan :

Berdasarkan SNI 2847-2019 Pasal 6.2.5 pengaruh kelangsingan boleh diabaikan apabila memenuhi persyaratan :

$\frac{k.l_u}{r} \leq 40$ dimana faktor panjang efektif (k) = 0,5

$\frac{0{,}5.1465}{72{,}46} = 10{,}075 \leq 40 \rightarrow$ Syarat terpenuhi sebagai kolom pendek

2. Tinjauan Beban Sentris

$P_0 = 0{,}85.f'_c.(A_g - A_{st}) + A_{st}.f_y$

$= [0{,}85.25.(90000 - 1061{,}858) + 1061{,}858.\ 473{,}744].10^{-3}$

$= 2392{,}985$ kN

$\phi.P_0 = 0{,}65.2392{,}985$

$= 1555{,}440$ kN

Kapasitas aksial maksimum :

$P_{n\,max} = 0{,}80.P_0$

$= 0{,}80.2392{,}985$

$= 1914{,}388$ kN

3. Tinjauan Kondisi Tekan Menentukan ($c > c_b$)

Lebar efektif (d) :

$d = h - d_s$

$= 300 - 25$

$= 275$ mm

Jarak antara garis netral dan tepi serat beton tekan pada kondisi regangan seimbang (c_b) :

$$c_b = \frac{600.d}{600 + f_y} = \frac{600.275}{600 + 473{,}744} = 153{,}668 \text{ mm}$$

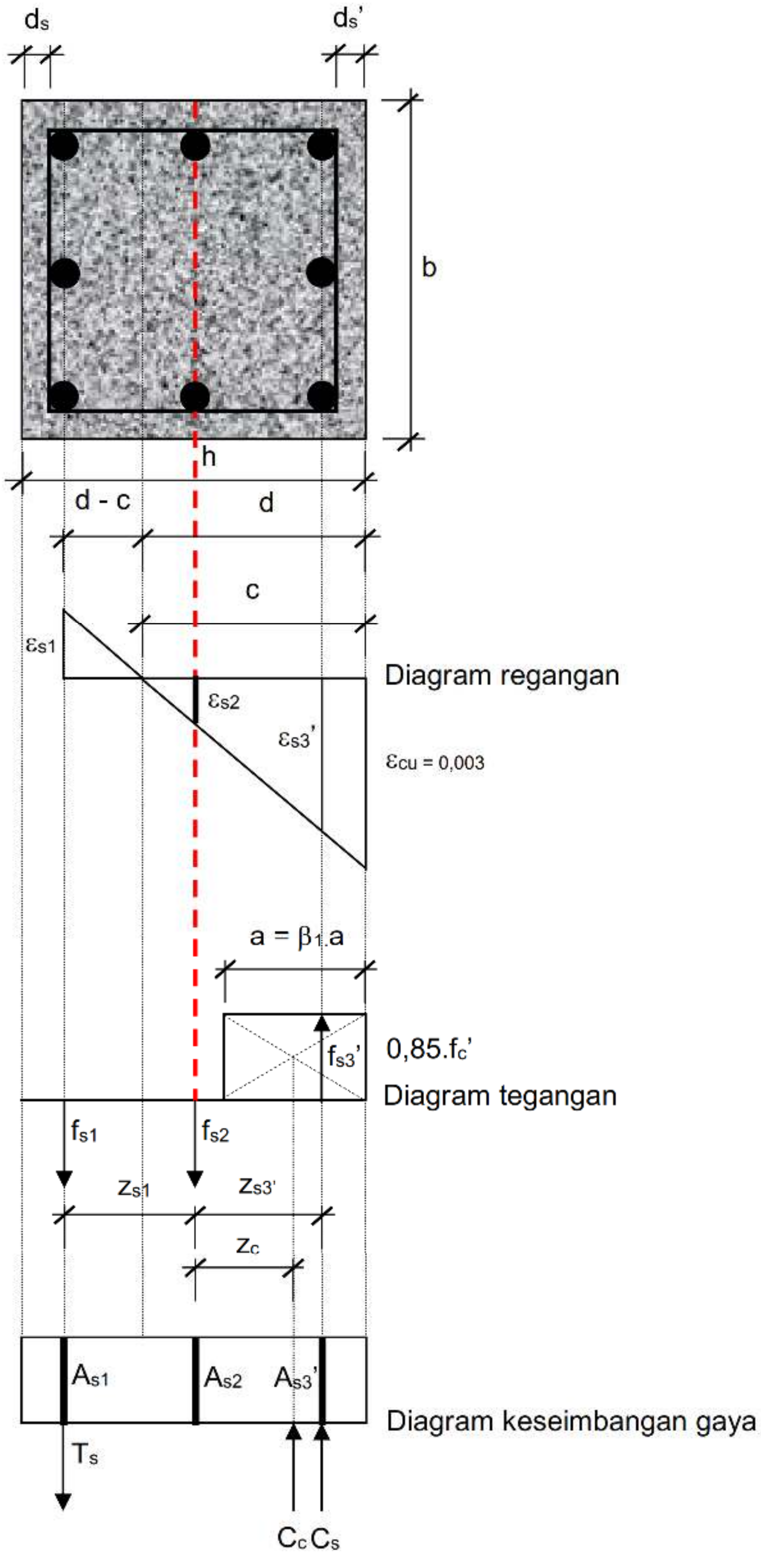
$c = 200$ mm

Tinggi blok tegangan beton tekan ekivalen (a) :

$a = \beta_1.c$

$= 0{,}85.200$

$= 170$ mm



Gambar 57. Diagram regangan, tegangan dan keseimbangan gaya kondisi tekan menentukan

Regangan dan tegangan pada setiap baris baja tulangan :

$\varepsilon_{s1} = \frac{d-c}{c}.\varepsilon_c' = \frac{275-200}{200}.0{,}003 = 0{,}00113 < \varepsilon_y$

$\varepsilon_{s2} = \frac{d-(d-c)}{c}.\varepsilon_c' = \frac{275-(275-200)}{200}.0{,}003 = 0{,}00300 > \varepsilon_y$

$\varepsilon_{s3}' = \frac{c-d_s'}{c}.\varepsilon_c' = \frac{200-25}{200}.0{,}003 = 0{,}00263 > \varepsilon_y$

$f_{s1} = \varepsilon_{s1}.E_s = 0{,}225$ kN/mm^2

$f_{s2}' = f_y.10^{-3} = 0{,}474$ kN/mm^2

$f_{s3}' = f_y.10^{-3} = 0{,}474$ kN/mm^2

Gaya-gaya internal yang bekerja :

$-T_s$	=	$A_{s1}.f_{s1}$	=	398,197.0,225	=	-89,594 kN
C_{s2}	=	$A_{s2}'.f_{s2}'$	=	265,465.0,474	=	125,762 kN
C_c	=	$0{,}85.f'_c.a.b$	=	0,85.25.170.300	=	1083,750 kN
C_{s3}	=	$A_{s3}'.f_{s3}'$	=	398,197.0,474	=	188,643 kN
P_n	=	$-T_s + C_{s2} + C_c + C_{s3}$			=	1308,561 kN

Lengan momen akibat gaya-gaya internal :

$-z_{s1}$	=	(h/2) - d_s	=	(0,300/2) – 0,025	=	-0,125 m
z_{s2}'	=	(h/2) - (h/2)	=	(0,300/2) – (0,30/2)	=	0,000 m
z_c	=	(h – a)/2	=	(0,300 – 0,170)/2	=	0,065 m
z_{s3}'	=	(h/2) - d_s'	=	(0,300/2) – 0,025	=	0,125 m

Momen akibat gaya-gaya internal :

M_{Ts}	=	$-T_s.z_{s1}$	=	-89,594.-0,125	=	11,199 kN.m
M_{Cs2}	=	$C_{s2}.z_{s2}'$	=	125,762.0	=	0,000 kN.m
M_{Cc}	=	$C_c.z_c$	=	1083,750.0,065	=	70,440 kN.m
M_{Cs3}	=	$C_s.z_{s3}'$	=	188,643.0,125	=	23,580 kN.m
M_n	=	$M_{Ts} + M_{Cs2} + M_{Cc} + M_{Cs3}$			=	105,223 kN.m

Eksentrisitas dalam kondisi tekan menentukan (e) :

$$e = \frac{M_n}{P_n} = \frac{105,223}{1308,561} = 0,0804 \text{ m} = 80,412 \text{ mm}$$

Gaya aksial dan momen nominal untuk kondisi kuat nominal (P_n dan M_n) :

P_n = 1308,561 kN

M_n = 105,223 kN.m

Gaya aksial dan momen nominal untuk kondisi kuat rencana ($\phi.P_n$ dan $\phi.M_n$) :

$\phi.P_n$ = 0,65.1308,561 = 850,565 kN

$\phi.M_n$ = 0,65.105,223 = 68,395 kN.m

4. Tinjauan Kondisi Seimbang ($c = c_b$)

d = 275 mm

$$c_b = \frac{600.d}{600 + f_y} = \frac{600.275}{600 + 473,744} = 154 \text{ mm}$$

$c = c_b$

$a_b = \beta_1.c_b$

= 131 mm

Regangan dan tegangan pada setiap baris baja tulangan :

$$\varepsilon_{s1} = \frac{d - c}{c}.\varepsilon_c' = \frac{275 - 154}{154}.0,003 = 0,00237 = \varepsilon_y$$

$$\varepsilon_{s2} = \frac{d - (d - c)}{c}.\varepsilon_c' = \frac{275 - (275 - 154)}{154}.0,003 = 0,00300 > \varepsilon_y$$

$$\varepsilon_{s3}' = \frac{c - d_s'}{c}.\varepsilon_c' = \frac{154 - 25}{154}.0,003 = 0,00251 > \varepsilon_y$$

$f_{s1} = \varepsilon_{s1}.E_s$ = 0,474 kN/mm^2

$f_{s2}' = f_y.10^{-3}$ = 0,474 kN/mm^2

$f_{s3}' = f_y.10^{-3}$ = 0,474 kN/mm^2

Gaya-gaya internal yang bekerja :

$-T_s = A_{s1}.f_{s1} = 398,197.0,474 = -188,643$ kN

$C_{s2} = A_{s2}'.f_{s2}' = 265,465.0,474 = 125,762$ kN

$C_c = 0,85.f'_c.a.b = 0,85.25.131.300 = 832,688$ kN

$C_{s3} = A_{s3}'.f_{s3}' = 398,197.0,474 = 188,643$ kN

$P_{nb} = -T_s + C_{s2} + C_c + C_{s3} = 958,450$ kN

Lengan momen akibat gaya-gaya internal :

$-z_{s1} = (h/2) - d_s = (0,300/2) – 0,025 = -0,125$ m

$z_{s2}' = (h/2) - (h/2) = (0,300/2) – (0,3/2) = 0,000$ m

$z_c = (h – a)/2 = (0,300 – 0,170)/2 = 0,085$ m

$z_{s3}' = (h/2) - d_s' = (0,300/2) – 0,025 = 0,125$ m

Momen akibat gaya-gaya internal :

$M_{Ts} = -T_s.z_{s1} = -188,643.-0,125 = 23,580$ kN.m

$M_{Cs2} = C_{s2}.z_{s2}' = 125,762.0 = 0,000$ kN.m

$M_{Cc} = C_c.z_c = 832,688.0,085 = 70,521$ kN.m

$M_{Cs3} = C_s.z_{s3}' = 188,643.0,125 = 23,580$ kN.m

$M_{nb} = M_{Ts} + M_{Cs2} + M_{Cc} + M_{Cs3} = 117,682$ kN.m

Eksentrisitas dalam kondisi seimbang menentukan (e_b) :

$$e_b = \frac{M_{nb}}{P_{nb}} = \frac{117,682}{958,450} = 0,123 \text{ m} = 122,784 \text{ mm}$$

Gaya aksial dan momen nominal untuk kondisi kuat nominal (P_n dan M_n) :

$P_{nb} = 958,450$ kN

$M_{nb} = 117,682$ kN.m

Gaya aksial dan momen nominal untuk kondisi kuat rencana ($\phi.P_n$ dan $\phi.M_n$) :

$\phi.P_n = 0,65.958,450 = 622,993$ kN

$\phi.M_n = 0,65.117,682 = 76,493$ kN.m

5. Tinjauan Kondisi Tarik Menentukan ($c < c_b$)

Lebar efektif (d) :

$d = 275$ mm

$c_b = \frac{600.d}{600 + f_y} = \frac{600.275}{600 + 473{,}744} = 154$ mm

$c = 100$ mm

$a = \beta_1.c$

$= 85$ mm

Regangan dan tegangan pada setiap baris baja tulangan :

$\varepsilon_{s1} = \frac{d - c}{c}.\varepsilon_c' = \frac{275 - 100}{100}.0{,}003 = 0{,}00525 > \varepsilon_y$

$\varepsilon_{s2} = \frac{d - (d - c)}{c}.\varepsilon_c' = \frac{275 - (275 - 100)}{100}.0{,}003 = 0{,}00300 > \varepsilon_y$

$\varepsilon_{s3}' = \frac{c - d_s'}{c}.\varepsilon_c' = \frac{100 - 25}{100}.0{,}003 = 0{,}00300 > \varepsilon_y$

$f_{s1} = \varepsilon_{s1}.E_s = 1{,}050$ kN/mm^2

$f_{s2}' = f_y.10^{-3} = 0{,}474$ kN/mm^2

$f_{s3}' = f_y.10^{-3} = 0{,}474$ kN/mm^2

Gaya-gaya internal yang bekerja :

$-T_s$	=	$A_{s1}.f_{s1}$	=	398,197.1,050	=	-418,107 kN
C_{s2}	=	$A_{s2}'.f_{s2}'$	=	265,465.0,474	=	125,762 kN
C_c	=	$0{,}85.f'_c.a.b$	=	0,85.25.100.300	=	541,875 kN
C_{s3}	=	$A_{s3}'.f_{s3}'$	=	398,197.0,474	=	188,643 kN
P_n	=	$-T_s + C_{s2} + C_c + C_{s3}$			=	438,174 kN

Lengan momen akibat gaya-gaya internal :

$-z_{s1}$	=	$(h/2) - d_s$	=	(0,300/2) – 0,025	=	-0,125 m
z_{s2}'	=	$(h/2) - (h/2)$	=	(0,300/2) – (0,30/2)	=	0,000 m
z_c	=	$(h - a)/2$	=	(0,300 – 0,100)/2	=	0,108 m

z_{s3}' = (h/2) - d_s' = (0,300/2) – 0,025 = 0,125 m

Momen akibat gaya-gaya internal :

M_{Ts} = $-T_s.z_{s1}$ = -418,107.-0,125 = 52,263 kN.m

M_{Cs2} = $C_{s2}.z_{s2}$' = 125,762.0 = 0,000 kN.m

M_{Cc} = $C_c.z_c$ = 541,875.0,108 = 58,265 kN.m

M_{Cs3} = $C_s.z_{s3}$' = 188,643.0,125 = 23,580 kN.m

M_n = $M_{Ts} + M_{Cs2} + M_{Cc} + M_{Cs3}$ = 134,095 kN.m

Eksentrisitas dalam kondisi seimbang menentukan (e) :

$$e = \frac{M_n}{P_n} = \frac{134,095}{438,174} = 0,306 \text{ m} = 306,032 \text{ mm}$$

Gaya aksial dan momen nominal untuk kondisi kuat nominal (P_n dan M_n) :

P_n = 438,174 kN

M_n = 134,095 kN.m

Gaya aksial dan momen nominal untuk kondisi kuat rencana ($\phi.P_n$ dan $\phi.M_n$) :

$\phi.P_n$ = 0,65.438,174 = 284,813 kN

$\phi.M_n$ = 0,65.134,095 = 87,162 kN.m

Batas struktur boleh dianggap hanya menahan momen lentur pada kondisi :

$P_{u\phi}$ = 0,10.f'$_c$.b.h

= $(0,10.25.300.300).10^{-3}$

= 225,000 kN

$P_{u\phi}$ = $\phi.P_{nb}$

= 0,65.958,450

= 622,993 kN

Digunakan nilai terkecil.

6. Tinjauan keadaan P = 0

Dalam kondisi ini perhitungan dilakukan seperti pada balok, dimana hal ini disebabkan oleh luas tulangan tekan (A'_2) yang terpasang sama dengan luas tulangan tarik (A_1), sehingga tulangan tekan pasti belum leleh.

Menentukan nilai a, a_{min} dan a_{maks} :

$$a = \frac{(A_s - A').f_y}{0,85.f'_c.b} = \frac{(1061,858 - 398,197).474}{0,85.25.300} = 49,319 \text{ mm}$$

$$a_{min} = \frac{600.\beta_1.d_s'}{600 - f_y} = \frac{600.0,85.25}{600 - 474} = 100,985 \text{ mm}$$

$$a_{max} = \frac{600.\beta_1.d_d}{600 + f_y} = \frac{600.0,85.275}{600 + 474} = 130,618 \text{ mm}$$

Syarat :

$a_{min} < a < a_{maks}$

$100,985 > 49,319 < 130,618$

Syarat di atas tidak terpenuhi, nilai $a < a_{min}$, sehingga tulangan tekan belum leleh dan nilai a baru harus dihitung ulang, sebagai berikut :

$$p = \frac{600.A_s' - A_s.f_y}{1,7.f'_c.b} = \frac{600.398,197 - 398,197.474}{1,7.25.300} = 3,943$$

$$q = \frac{600.\beta_1.d_s'.A_s'}{0,85.f'_c.b} = \frac{600.0,85.25.398,197}{0,85.25.300} = 796,394$$

Nilai a baru :

$$a = \sqrt{p^2 + q} - p = \sqrt{3,943^2 + 796,394} - 3,943 = 24,551 \text{ mm}$$

$$f'_s = \frac{a - \beta_1.d_s'}{a}.600 = \frac{24,551 - 0,85.25}{24,551}.600 = 80,683 \text{ MPa}$$

M_{nc} = $0,85.f'_c.a.b.(d - 0,5.a)$

= $[0,85.25.24,551.300 - (275 - 0,5.24,551)].10^{-6}$

= 41,120 kN.m

M_{ns} = $A_s'.f_s'.(d - d_s')$

= $[398,197.80,683.(275 - 25)].10^{-6}$

= 8,032 kN.m

M_n = $M_{nc} + M_{ns}$

= 41,120 + 8,032

= 49,152 kN.m

M_r = $\phi.M_n$

ϕ = 0,65 $\rightarrow$ M_r = 0,65.49,152

= 31,949 kN.m

ϕ = 0,80 $\rightarrow$ M_r = 0,80.49,152

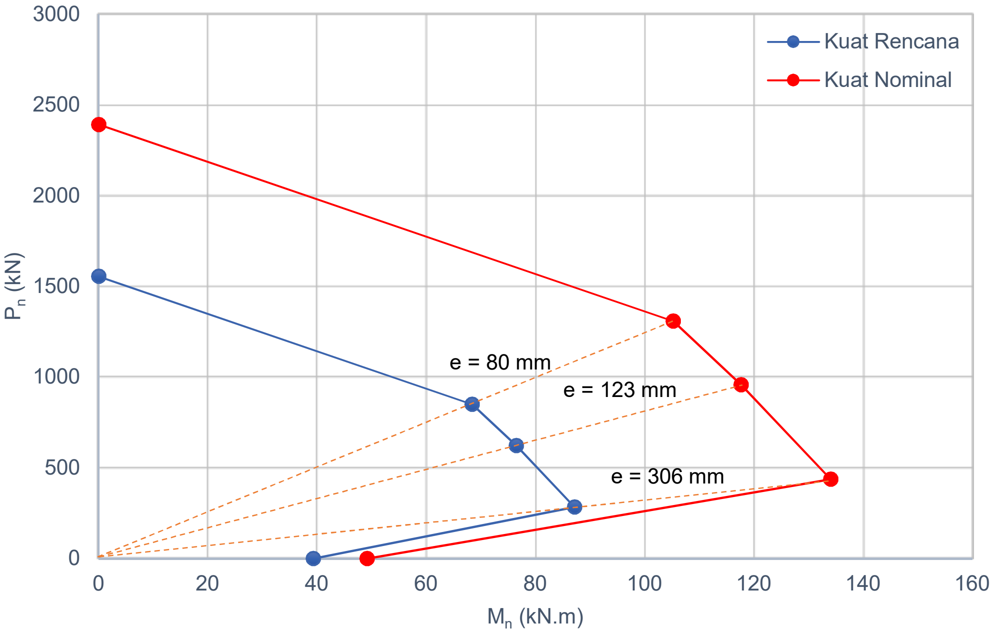
= 39,322 kN.m

Selanjutnya nilai kuat rencana dan nilai kuat nominal dari hasil perhitungan di atas disajikan dalam Tabel 29 di bawah ini.

Tabel L1. Kuat rencana dan kuat nominal kolom

Kondisi	**Kuat Rencana**		**Kuat Nominal**	
	$\phi.M_n$ (kN.m)	**$\phi.P_n$ (kN)**	**M_n (kN.m)**	**P_n (kN)**
Aksial sentris	0	1555,440	0	2392,985
Tekan	68,395	850,565	105,223	1308,561
Seimbang	76,493	622,993	117,682	958,460
Tarik	87,162	284,813	134,095	438,174
Lentur murni	39,322	0	49,152	0

Selain itu, nilai-nilai dalam tabel di atas, dapat dibuat dalam bentuk diagram interaksi seperti disajikan pada Gambar 57 di bawah ini.



Gambar L1. Diagram interaksi penampang kolom