COLUMN DESIGN FOR FLEXURAL (NON DUCTILE-BRACED)

DESIGN CODE ACI 318-2014

General Data

$ColumnNo$: C1

$Level$: 16 ft To 32 ft

$FrameType := 1$ ----------- 1- for Non Ductile, 2 for Special

$B := 15\ in$ ----------- Width of the Column

$D := 24\ in$ ----------- Depth of the Column

$f'c := 3.5\ ksi$ ----------- Grade of Concrete (Cylindrical Strength)

$fy := 60\ ksi$ ----------- Grade of Main Reinforcement

$fyt := 60\ ksi$ ----------- Grade of Secondary Reinforcement

$Cc := 2\ in$ ----------- Nominal Cover to Beam Tension Reinforcement

$Es := 29007\ ksi$ ----------- Modulus of elasticity of reinforcement

$\phi BM := 0.65$ ----------- Capacity reduction factor for Bending Moment

$\phi shear := 0.75$ ----------- Capacity reduction factor for Shear and torsion

$ptmax := 4$ ----------- Maximum % reinforcement

$ptmin := 1$ ----------- Minimum % reinforcement

$\varepsilon cu := 0.003$ ----------- Permissible Strain in Concrete

$lux := 162\ in$ ----------- Clear Floor Height along Major Direction@lux

$luy := 162\ in$ ----------- Clear Floor Height along Minor Direction@luy

$NoOfFloors := 1$ ----------- Number of Floors considered after merged of levels

$NoOfColumnsInGroup := 1$ ----------- Number of column in Group

$ColumnType := 1$ ----------- 1- for Braced, 2 for Un-Braced

$MinimumEccentricityCheck$ ----------- Simultaneously (Both Axis)

$CodeDefinedDBRatio := 4$

$$DBRatio := \left\| \begin{array}{l} \text{if } \dfrac{D}{B} < 4 \\ \quad \left\| \text{“DesignAsColumn”} \right. \\ \text{else} \\ \quad \left\| \text{“DesignAsWall”} \right. \end{array} \right. = \text{“DesignAsColumn”}$$

Reinforcement Provided

$\phi 1 := 19.1\ mm$

$N1 := 10$

$\phi 2 := 15.9\ mm$

$N2 := 2$

$$Ast := \frac{\pi \cdot \phi 1^2}{4} \cdot N1 + \frac{\pi \cdot \phi 2^2}{4} \cdot N2 = 5.057\ in^2$$

----------- Area of Reinforcement Provided

Flexural Design (Analysis Forces)

$AnalysisReferenceNo$ 501

$LoadCombination$ [9] : 1.5 (LOAD 1: LOAD CASE 1) -1.5 (LOAD 4: LOAD CASE 4 EQ-Y)

$CriticalLoacation$ Top Joint

$Put := 109.91\ kip$ ----------- Ultimate moment from analysis at Top Node

$Muxt := 56.62\ kip \cdot ft$ ----------- Ultimate moment from analysis along D at Top Node

$Muyt := 157.25\ kip \cdot ft$ ----------- Ultimate moment from analysis along B at Top Node
$Vuxt := -18.29\ kip$ ----------- Ultimate Shear from analysis along D at Top Node
$Vuyt := 6.55\ kip$ ----------- Ultimate Shear from analysis along B at Top Node
$Pub := 117.11\ kip$ ----------- Ultimate moment from analysis at Bottom Node
$Muxb := -48.1\ kip \cdot ft$ ----------- Ultimate moment from analysis along D at Bottom Node
$Muyb := -135.35\ kip \cdot ft$ ----------- Ultimate moment from analysis along B at Bottom Node
$Vuxb := 18.29\ kip$ ----------- Ultimate Shear from analysis along D at Bottom Node
$Vuyb := 6.55\ kip$ ----------- Ultimate Shear from analysis along B at Bottom Node

Sway Calculation (Stability Index)

$For\_Global\_X\_Direction$

$LoadCombination$ [10] : 1.2(LOAD 1: LOAD CASE 1) +0.5 (LOAD 2: LOAD CASE 2)-1.0 (LOAD 3: LOAD CASE 3 EQ-X)

$StoreyHeight(A) := 192\ in$ ----------- Height of Story Considered for design
$GravityLoad(B) := 1499.22\ kip$ ----------- Summation of Axial Load of Story selected
$RelativeDisplacement(C) := 0.47\ in$ ----------- Relative Displacement for Story selected
$StoreyShear(D) := 120.05\ kip$ ----------- Story Shear for Story selected

$$StabilityIndex\_Q := \frac{GravityLoad(B)}{StoreyHeight(A)} \cdot \frac{RelativeDisplacement(C)}{StoreyShear(D)} = 0.031$$

----------- Clause 6.6.4.4.1

$$Check := \begin{array}{l} \text{if } StabilityIndex\_Q \le 0.05 \\ \quad \text{"Non Sway Frame(Braced)"} \\ \text{else} \\ \quad \text{"Sway Frame(UnBraced"} \end{array} = \text{"Non Sway Frame(Braced)"}$$

----------- Clause 6.6.4.3

$For\_Global\_Y\_Direction$

$LoadCombination$ [13] : 1.2(LOAD 1: LOAD CASE 1) +0.5 (LOAD 2: LOAD CASE 2)+1.0 (LOAD 3: LOAD CASE 3 EQ-Y)

$StoreyHeight(A) := 192\ in$ ----------- Height of Story Considered for design
$GravityLoad(B) := 1499.22\ kip$ ----------- Summation of Axial Load of Story selected
$RelativeDisplacement(C) := 0.27\ in$ ----------- Relative Displacement for Story selected
$StoreyShear(D) := 120.05\ kip$ ----------- Story Shear for Story selected

$$StabilityIndex\_Q := \frac{GravityLoad(B)}{StoreyHeight(A)} \cdot \frac{RelativeDisplacement(C)}{StoreyShear(D)} = 0.018$$

$$Check := \begin{array}{l} \text{if } StabilityIndex\_Q \le 0.05 \\ \quad \text{"Non Sway Frame(Braced)"} \\ \text{else} \\ \quad \text{"Sway Frame(UnBraced"} \end{array} = \text{"Non Sway Frame(Braced)"}$$

----------- Clause 6.6.4.3

Effective Length Factor

$EffectiveLengthFactorAlongD := 1.2$
$EffectiveLengthFactorAlongB := 1.2$

Calculation of Slenderness Check

Along D:

$EffectiveLengthD := EffectiveLengthFactorAlongD \cdot luy = 194.4\ in$

$A := D \cdot B = 360\ in^2$ ----------- Area of Cross Section

$IxxD := B \cdot \frac{D^3}{12} = 17280\ in^4$ ----------- Moment of Inertia

$rD := \sqrt{\left(\frac{IxxD}{A}\right)} = 6.928\ in$ ----------- Radius of Gyration

$k\_D := EffectiveLengthFactorAlongD = 1.2$

$Actual\_Slender\_D := \frac{EffectiveLengthD}{rD} = 28.059$

$$M1 := \begin{array}{l} \text{if abs}(Muxt) > \text{abs}(Muxb) \\ \quad \| Muxb \\ \text{else} \\ \quad \| Muxt \end{array} = -48.1\ kip \cdot ft$$

----------- Smaller Design Bending Moment at the ends of the the column. Clause 6.2.5.1(b)

$$M2 := \begin{array}{l} \text{if abs}(Muxb) > \text{abs}(Muxt) \\ \quad \| Muxb \\ \text{else} \\ \quad \| Muxt \end{array} = 56.62\ kip \cdot ft$$

----------- Higher Design Bending Moment at the ends of the the column. Clause 6.2.5.1(b)

$LerPerD := min\left(40, 34 - 12 \cdot \frac{M1}{M2}\right) = 40$ ----------- Clause 6.2.5.1(b)

$$Check1 := \begin{array}{l} \text{if } LerPerD > Actual\_Slender\_D \\ \quad \| \text{“Column Not Slender Along B”} \\ \text{else if } Actual\_Slender\_D < 100 \\ \quad \| \text{“Slender\_Use Approximate method”} \\ \text{else} \\ \quad \| \text{“Slender\_P–delta effect (revise)”} \end{array} = \text{“Column Not Slender Along B”}$$

Along B:

$EffectiveLengthB := EffectiveLengthFactorAlongB \cdot lux = 194.4\ in$

$A := D \cdot B = 360\ in^2$ ----------- Area of Cross Section

$IxxB := D \cdot \frac{B^3}{12} = 6750\ in^4$ ----------- Moment of Inertia

$rB := \sqrt{\left(\frac{IxxB}{A}\right)} = 4.33\ in$ ----------- Radius of Gyration

$$M1 := \begin{array}{l} \text{if } \mathrm{abs}(Muyt) > \mathrm{abs}(Muyb) \\ \quad \| Muyb \\ \text{else} \\ \quad \| Muyt \end{array} = -135.35\ kip \cdot ft$$

----------- Smaller Design Bending Moment at the ends of the the column. Clause 6.2.5

$$M2 := \begin{array}{l} \text{if } \mathrm{abs}(Muyb) > \mathrm{abs}(Muyt) \\ \quad \| Muyb \\ \text{else} \\ \quad \| Muyt \end{array} = 157.25\ kip \cdot ft$$

----------- Higher Design Bending Moment at the ends of the the column. Clause 6.2.5

$$Actual\_Slender\_B := \frac{EffectiveLengthB}{rB} = 44.895$$

$$LerPerB := min\left(40, 34 - 12 \cdot \frac{M1}{M2}\right) = 40$$ ----------- Clause 6.2.5.1(b)

$$Check1 := \begin{array}{l} \text{if } LerPerB > Actual\_Slender\_B \\ \quad \| \text{“Column Not Slender Along B”} \\ \text{else if } Actual\_Slender\_B < 100 \\ \quad \| \text{“Slender\_Use Approximate method”} \\ \text{else} \\ \quad \| \text{“Slender\_P–delta effect (revise)”} \end{array} = \text{“Slender\_Use Approximate method”}$$

Calculation of Slenderness Moment - Braced (Non-Sway) Frame

Along B:

$$\beta dns := 0.6$$ ----------- Clause 6.6.4.4.4

$$Ec := \frac{57000}{1000} \cdot \sqrt{\left(\frac{f'c \cdot 1000}{ksi}\right)} = 3372.165$$ ----------- Clause 19.2.2.1

$$Ig := D \cdot \frac{B^3}{12} = \left(6.75 \cdot 10^3\right)\ in^4$$ ----------- Moment of Inertia of Section

$$EI := 0.288675 \cdot Ec \cdot Ig = \left(6.571 \cdot 10^6\right)\ in^4$$

$$M1min := Pub \cdot (0.6 \cdot in + 0.03 \cdot B) = 10.247\ kip \cdot ft$$ ----------- Clause 6.6.4.5.4

$$M2min := Pub \cdot (0.6 \cdot in + 0.03 \cdot B) = 10.247\ kip \cdot ft$$ ----------- Clause 6.6.4.5.4

$$Cm := \begin{array}{l} \text{if } M2min > M2 \\ \quad \| 1 \\ \text{else} \\ \quad \| min\left(\left(0.6 + 0.4 \cdot \left(\frac{M1}{M2}\right)\right), 0.4\right) \end{array} = 0.256$$

----------- Clause 6.6.4.5.3

$$Pc := \pi \cdot \pi \cdot \frac{EI \cdot kip}{\left(EffectiveLengthB^2\right) \cdot in \cdot in} = 1716.046\ kip$$

----------- Clause 6.6.4.4.2

$$\delta := \max\left(1, \frac{Cm}{\left(1 - \frac{Put}{0.75 \cdot Pc}\right)}\right) = 1$$ ----------- Clause 6.6.4.5.2

$Mc1 := \text{sign}(Muyt) \cdot \delta \cdot \max(\text{abs}(Muyt), M1min) = 157.25\ kip \cdot ft$

----------- Clause 6.6.4.5.3

$Mc2 := \text{sign}(Muyb) \cdot \delta \cdot \max(\text{abs}(Muyb), M2min) = -135.35\ kip \cdot ft$

----------- Clause 6.6.4.5.3

$1.4\ Muyt = 220.15\ 1 \cdot kip \cdot ft$

$1.4\ Muyb = -189.49\ 1 \cdot kip \cdot ft$

$Mc1\_final := \text{sign}(Mc1) \cdot min(\text{abs}(1.4\ Muyt), \text{abs}(Mc1)) = 157.25\ kip \cdot ft$

----------- Clause 6.2.5.3

$Mc2\_final := \text{sign}(Mc2) \cdot min(\text{abs}(1.4\ Muyb), \text{abs}(Mc2)) = -135.35\ kip \cdot ft$

----------- Clause 6.2.5.3

Final Design Forces for Flexural Design

At Top Node:

$Put = 109.91\ kip$

$Mux := Muxt = 56.62\ kip \cdot ft$

$Muy := Mc1\_final = 157.25\ kip \cdot ft$

At Bottom Node:

$Pub = 117.11\ kip$

$Mux := Muxb = -48.1\ kip \cdot ft$

$Muy := Mc2\_final = -135.35\ kip \cdot ft$

Final Critical Design Forces

At Top Node:

$Put = 109.91\ kip$

$Mux\_Final := Muxt = 56.62\ kip \cdot ft$

$Muy\_Final := Mc1\_final = 157.25\ kip \cdot ft$

ϕPn, Max Check

$fy\_1 := fy = 60\ ksi$ ----------- Clause 22.4.2.1

$LoadCombination$ [2] : 1.2 (LOAD 1: LOAD CASE 1) +1.6 (LOAD 2: LOAD CASE 2)

$CriticalLocation$ Bottom Joint

$Pu := 863.41\ kN$ ----------- Factored Axial force

$Mux1 := -188.66\ kip \cdot ft$ ----------- Factored Bending Moment along Major Direction

$Muy1 := -37.88\ kip \cdot ft$ ----------- Factored Bending Moment along Minor Direction

$Ag := B \cdot D = 360\ in^2$ ----------- Cross Section Area

$Ast = 5.057\ in^2$ ----------- Area of Reinforcement Provided

$pt := \frac{Ast \cdot 100}{Ag} = 1.405$ ----------- % Reinforcement Provided

$\phi Pnmax := 0.8 \cdot \phi BM \cdot (0.85 \cdot f'c \cdot (Ag - Ast) + (Ast \cdot fy\_1)) = 706.864\ kip$

----------- Clause 22.4.2.2

$$Check := \left\| \begin{array}{l} \text{if } Pu \le \phi Pnmax \\ \quad \| \text{“Ok”} \\ \text{else} \\ \quad \| \text{“Revise”} \end{array} \right. = \text{“Ok”}$$

Minimum Ast Calculation

$fy\_2 := fy = 60\ ksi$ ----------- Clause 22.4.2.1

$LoadCombination$ [2] : 1.2 (LOAD 1: LOAD CASE 1) +1.6 (LOAD 2: LOAD CASE 2)

$Pu\_max := 156.37\ kip$

$$AstReq := \frac{0.442 \cdot f'c \cdot Ag - Pu\_max}{(0.442 \cdot f'c) - (0.52 \cdot fy\_2)} = -13.508\ in^2$$

----------- Clause 22.4.2.2

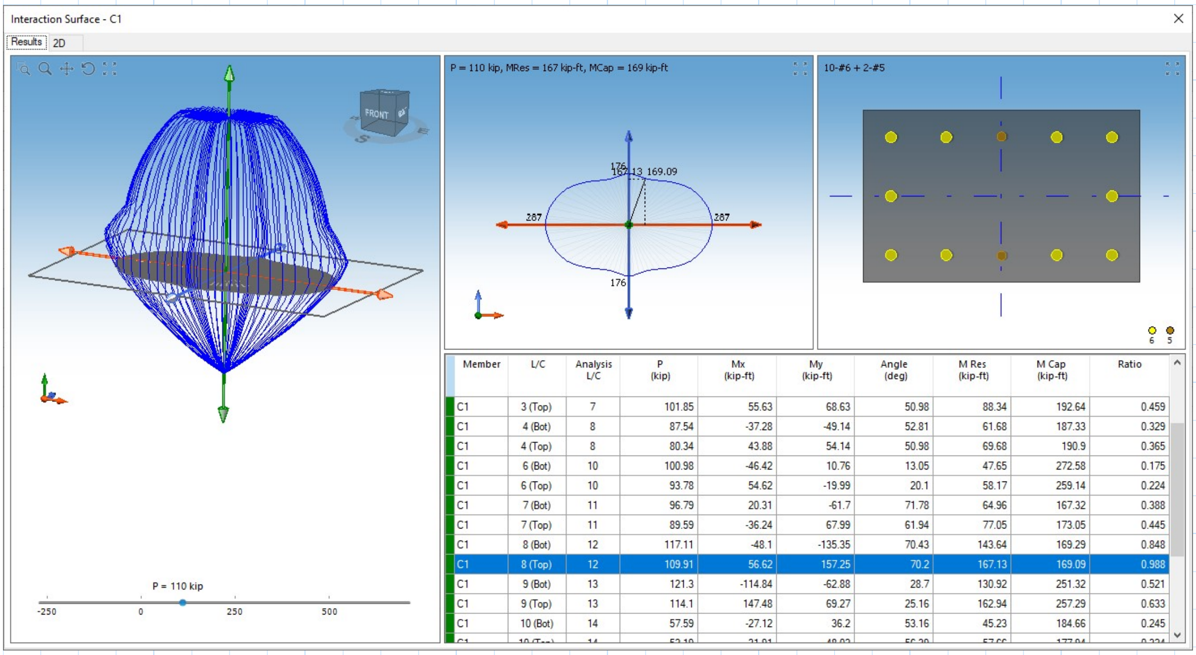
$$pt := \left\| \begin{array}{l} \text{if } AstReq \le 0 \\ \quad \| \ 0 \\ \text{else} \\ \quad \left\| \ \dfrac{AstReq}{Ag} \right. \end{array} \right. = 0$$

$ptmin := \max(0.5, pt) = 0.5$ ----------- Clause 10.3.1.2

$UserDefinedpt := 1$

$Finalpt := \max(ptmin, UserDefinedpt) = 1$

PM Curve



Member	L/C	Analysis L/C	P (kip)	Mx (kip-ft)	My (kip-ft)	Angle (deg)	M Res (kip-ft)	M Cap (kip-ft)	Ratio
C1	3 (Top)	7	101.85	55.63	68.63	50.98	88.34	192.64	0.459
C1	4 (Bot)	8	87.54	-37.28	-49.14	52.81	61.68	187.33	0.329
C1	4 (Top)	8	80.34	43.88	54.14	50.98	69.68	190.9	0.365
C1	6 (Bot)	10	100.98	-46.42	10.76	13.05	47.65	272.58	0.175
C1	6 (Top)	10	93.78	54.62	-19.99	20.1	58.17	259.14	0.224
C1	7 (Bot)	11	96.79	20.31	-61.7	71.78	64.96	167.32	0.388
C1	7 (Top)	11	89.59	-36.24	67.99	61.94	77.05	173.05	0.445
C1	8 (Bot)	12	117.11	-48.1	-135.35	70.43	143.64	169.29	0.848
C1	8 (Top)	12	109.91	56.62	157.25	70.2	167.13	169.09	0.988
C1	9 (Bot)	13	121.3	-114.84	-62.88	28.7	130.92	251.32	0.521
C1	9 (Top)	13	114.1	147.48	69.27	25.16	162.94	257.29	0.633
C1	10 (Bot)	14	57.59	-27.12	36.2	53.16	45.23	184.66	0.245

Resultant Moment (Combined Action)

Moment Capacity Check

$$Ptcalculated := \frac{Ast \cdot 100}{Ag} = 1.4$$

----------- % Reinforcement Provided

$LoadAngle := 39.8$ ----------- tan-1 (Muy_Final / Mux_Final)

$$Mres := \sqrt{Mux\_Final^2 + Muy\_Final^2} = 167.133\ kip \cdot ft$$

----------- Resultant Bending Moment

$\phi Mcap := 169.08\ kip \cdot ft$ ----------- Moment Capacity from PM Curve

$CapacityRatio := \dfrac{Mres}{\phi Mcap} = 0.988$ ----------- Clause 10.4.2

$$Check := \left\| \begin{array}{l} \text{if } CapacityRatio < 1 \\ \quad \| \text{"Ok"} \\ \text{else} \\ \quad \| \text{"Revise"} \end{array} \right\| = \text{"Ok"}$$

----------- Clause 10.4.2

Design Data for All Load Combination

C1

Level	:	16 ft To 32 ft
Frame Type	=	Non-Ductile
Design Code	=	ACI 318 - 14
Grade Of Concrete (f'c)	=	3.5 ksi
Grade Of Steel (Main)	=	60 ksi
Grade Of Steel (Shear)	=	60 ksi
Column B	=	15 in
Column D	=	24 in
Clear Cover	=	2 in
Pt	=	1.4 %

Design Table :

Member	LOC	L/C	Analysis L/C No	Pu	Analysis		Msldr or MC		Design			Mcap	Capacity Ratio
					Mx	My	Mx	My	Mux	Muy	MuRes		
				(kip)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	
41	BOT	1	5	102.13	-43.49	-57.33	-	-57.33	-43.49	-57.33	71.96	188.48	0.382
41	TOP	1	5	93.72	51.19	63.16	-	63.16	51.19	63.16	81.3	191.93	0.424
41	BOT	2	6	156.37	-69.22	-91.24	-	-91.24	-69.22	-91.24	114.52	191.41	0.598
41	TOP	2	6	149.16	81.47	100.52	-	100.52	81.47	100.52	129.39	195.2	0.663
41	BOT	3	7	109.05	-47.26	-62.29	-	-62.29	-47.26	-62.29	78.19	189.02	0.414
41	TOP	3	7	101.84	55.62	68.63	-	68.63	55.62	68.63	88.34	192.64	0.459
41	BOT	4	8	87.54	-37.28	-49.14	-	-49.14	-37.28	-49.14	61.68	187.33	0.329
41	TOP	4	8	80.34	43.88	54.14	-	54.14	43.88	54.14	69.68	190.9	0.365
41	BOT	6	10	100.98	-46.42	10.76	-	10.76	-46.42	10.76	47.65	272.58	0.175
41	TOP	6	10	93.78	54.63	-19.99	-	-19.99	54.63	-19.99	58.17	259.14	0.224
41	BOT	7	11	96.79	20.31	-61.71	-	-61.71	20.31	-61.71	64.96	167.32	0.388
41	TOP	7	11	89.59	-36.24	67.99	-	67.99	-36.24	67.99	77.04	173.05	0.445
41	BOT	8	12	117.11	-48.1	-135.35	-	-135.35	-48.1	-135.35	143.64	169.29	0.848
41	TOP	8	12	109.91	56.62	157.25	-	157.25	56.62	157.25	167.13	169.08	0.988
41	BOT	9	13	121.3	-114.84	-62.88	-	-62.88	-114.84	-62.88	130.92	251.32	0.521
41	TOP	9	13	114.1	147.48	69.27	-	69.27	147.48	69.27	162.94	257.29	0.633
41	BOT	10	14	57.59	-27.12	36.2	-	36.2	-27.12	36.2	45.23	184.66	0.245
41	TOP	10	14	52.19	31.91	-48.02	-	-48.02	31.91	-48.02	57.65	177.94	0.324
41	BOT	11	15	53.4	39.61	-36.27	-	-36.27	39.61	-36.27	53.71	202.84	0.265
41	TOP	11	15	48	-58.95	39.96	-	39.96	-58.95	39.96	71.22	216.64	0.329
41	BOT	12	16	73.72	-28.8	-109.91	-	-109.91	-28.8	-109.91	113.62	162.42	0.7
41	TOP	12	16	68.31	33.9	129.22	-	129.22	33.9	129.22	133.59	161.05	0.83
41	BOT	13	17	77.91	-95.53	-37.44	-	-37.44	-95.53	-37.44	102.61	250.36	0.41
41	TOP	13	17	72.51	124.77	41.24	-	41.24	124.77	41.24	131.41	252.53	0.52