COLUMN DESIGN FOR SHEAR AND LINKS (DUCTILE SPECIAL FRAME)

DESIGN CODE ACI 318-2014

Input / Defaults

$ColumnNo$: C21

$Location$: Bottom Joint

$TypeOfColumn := 1$ ----------- 1- for Non-Ductile, 2 for Intermediate, 3 for Special

$B := 30\ in$ ----------- Width of the Column

$D := 36\ in$ ----------- Depth of the Column

$Pu\_D := 1240.19\ kip$ ----------- factored axial force occurring simultaneously with Vu along D

$Pu\_B := 788.73\ kip$ ----------- factored axial force occurring simultaneously with Vu along B

$Mshear\_D := 47.9\ kip \cdot ft$ ----------- factored Bending Moment occurring simultaneously with Vu along D

$Mshear\_B := 12.67\ kip \cdot ft$ ----------- factored Bending Moment occurring simultaneously with Vu along D

$Vuy3 := 277.38\ kip$ ----------- Ultimate Shear force at section considered along D

$Vux3 := 9.4\ kip$ ----------- Ultimate Shear force at section considered along B

$f'c := 3.0\ ksi$ ----------- Grade of Concrete (Cylindrical Strength)

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

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

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

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

$LuD := 69\ in$ ----------- Clear Floor Height @ lux

$LuB := 69\ in$ ----------- Clear Floor Height @ luy

$\lambda := 1$ ----------- Modification factor for compressive strength

Reinforcement Provided

$\phi 1 := 1\ in$ ----------- Diameter of Reinforcement

$N1 := 4$ ----------- No of Rebar

$\phi 2 := 0.75\ in$ ----------- Diameter of Reinforcement

$N2 := 18$ ----------- No of Rebar

$$Ast := \frac{\pi \cdot \phi 1^2}{4} \cdot N1 + \frac{\pi \cdot \phi 2^2}{4} \cdot N2 = 11.094\ in^2$$

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

Shear reinforcement Provided

$\phi 3 := 0.375\ in$ ----------- Diameter of Link

$Bundled\_1 := 1$

$Legs1 := 6$ ----------- Number of shear Legs along D

$Legs2 := 7$ ----------- Number of shear Legs along B

$Spc := 12\ in$ ----------- Spacing of Non-ductile links provided

$$Asvprv\_D := \left(\frac{\pi \cdot \phi 3^2}{4} \cdot Legs1\right) \cdot \frac{1}{Spc} \cdot Bundled\_1 = 0.663\ \frac{in^2}{ft}$$

----------- Area of Links Provided along D

$$Asvprv\_B := \left(\frac{\pi \cdot \phi 3^2}{4} \cdot Legs2\right) \cdot \frac{1}{Spc} \cdot Bundled\_1 = 0.773\ \frac{in^2}{ft}$$

----------- Area of Links Provided along B

$$fy\_s := \begin{cases} \text{if } fy > 60\ ksi \\ \quad 60\ ksi \\ \text{else} \\ \quad fy \end{cases} = 60\ ksi$$

----------- Permissible yield strength of transverse reinforcement Table 20.2.2.4a

Shear Force Calculation

Shear as per Column Flexural Capacity (Fig. R18.3.3)

18.3.3 Columns having unsupported length $\ell_u \leq 5c_1$ shall have ϕV_n at least the lesser of (a) and (b):

(a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.

(b) The maximum shear obtained from design load combinations that include E, with $\Omega_o E$ substituted for E.

Along D

$LuD = 69\ in$

$D = 36\ in$

$$Check := \begin{cases} \text{if } LuD < 5 \cdot D \\ \quad \text{"ShortColumn"} \\ \text{else} \\ \quad \text{"LongColumn"} \end{cases} = \text{"ShortColumn"}$$

$Pu\_top := 788.73\ kip$

$Mnt\_top := 1153.25\ kip \cdot ft$

$Pu\_bot := 797.09\ kip$

$Mnt\_bot := 1143.83\ kip \cdot ft$

$$Vuy1 := \frac{(Mnt\_top + Mnt\_bot)}{LuD} = 399.492\ kip$$

Along B

$LuB = 69\ in$

$B = 30\ in$

$$Check := \begin{cases} \text{if } LuB < 5 \cdot B \\ \quad \text{"ShortColumn"} \\ \text{else} \\ \quad \text{"LongColumn"} \end{cases} = \text{"ShortColumn"}$$

$Pu\_top := 788.73\ kip$

$Mnt\_top := 957.56\ kip \cdot ft$

$Pu\_bot := 797.09\ kip$

$Mnt\_bot := 948.41\ kip \cdot ft$

$$Vux1 := \frac{(Mnt\_top + Mnt\_bot)}{LuB} = 331.473\ kip$$

Note:
The column moment capacity (Mnt_top and Mnt_bot) is calculated for axial force (Pu) which gives the maximum moment capacity. The PM curve is and the capacity is calculated.

Shear from Load combinations with Enhanced Eq factor

Shear Design along D

Critical Load Combination 1.2 (LOAD 1: LOAD CASE 1) +0.5 (LOAD 2: LOAD CASE 2) +3 (LOAD 3: LOAD CASE 3 EQ-X)

$$Vuy2 := 268.69\ kip$$

Shear Design along B

Critical Load Combination 1.2 (LOAD 1: DEAD LOAD) +0.5 (LOAD 2: LIVE LOAD) +3 (LOAD 4: EQ-Y)

$$Vux2 := 28.58\ kip$$

$$Vu'y := min(Vuy1, Vuy2) = 268.69\ kip$$
$$Vu'x := min(Vux1, Vux2) = 28.58\ kip$$

Shear Design along D

Critical Load Combination [1] : 1.4 (LOAD 1: LOAD CASE 1)

$$Pu\_D := 788.73\ kip$$
$$Mshear\_D := 0.3\ kip \cdot ft$$
$$Vuy3 := -1.91\ kip$$

$$Vuy := \max(Vu'y, Vuy3) = 268.69\ kip$$

$$\phi := \begin{cases} \text{if } TypeOfColumn > 2 \\ \quad 0.6 \\ \text{else} \\ \quad 0.75 \end{cases} = 0.75$$

----------- Strength Reduction Factor

$$deff := D - Cc - \frac{\phi 1}{2} = 33.5\ in$$

$$pt := \frac{Ast}{2 \cdot B \cdot deff} = 0.00552$$

----------- 50% of total reinforcement assumed as Tension Reinforcement

$$Ag := D \cdot B = 1080\ in^2$$
$$Aeff\_y := deff \cdot B = 1005\ in^2$$

$$Mm\_y := Mshear\_D - Pu\_D \cdot \frac{(4 \cdot D - deff)}{8} = -907.561\ kip \cdot ft$$

$$Vcy1 := 2 \cdot \left(1 \cdot psi + \left(\frac{Pu\_D}{500 \cdot Ag}\right)\right) \cdot \lambda \cdot \sqrt{f'c \cdot psi} \cdot B \cdot \frac{deff}{psi} = 270.894\ kip$$

----------- Clause 22.5.7.1

$$Vcy2 := \left(1.9 \cdot \sqrt{f'c \cdot psi} + 2500 \cdot pt \cdot psi \cdot \left(\frac{Vuy \cdot deff}{Mm\_y}\right)\right) \cdot B \cdot deff = 93.126\ kip$$

----------- Table 22.5.6.1 (a)

$$Vcy3 := \left(3.5 \cdot \sqrt{f'c} \cdot \sqrt{1 \cdot psi + \left(\frac{Pu\_D}{500 \cdot Ag}\right)}\right) \cdot B \cdot deff = 302.215\ kip$$

----------- Table 22.5.6.1 (b)

$$\phi Vcy := \begin{array}{l} \text{if } Pu\_D < 0 \\ \quad Vcy1 \cdot \phi \\ \text{else if } Mm\_y < 0 \\ \quad Vcy3 \cdot \phi \\ \text{else} \\ \quad \phi \cdot min(Vcy2, Vcy3) \end{array} = 226.661\ kip$$

$$Check1 := \begin{array}{l} \text{if } \phi Vcy \geq Vuy \\ \quad \text{"ShearReinfNotRequired"} \\ \text{else} \\ \quad \text{"ShearReinfRequired"} \end{array} = \text{"ShearReinfRequired"}$$

----------- Clause 7.1.1

$$fy\_s = 60\ ksi$$

$$Vsy := \begin{array}{l} \text{if } \phi Vcy \geq Vuy \\ \quad 0\ kip \\ \text{else} \\ \quad \frac{(Vuy - \phi Vcy)}{\phi} \end{array} = 56.038\ kip$$

----------- Clause 7.1.1

$$Vsy\_perm := 8 \cdot \sqrt{f'c \cdot psi} \cdot B \cdot deff = 440.369\ kip$$

$$Check2 := \begin{array}{l} \text{if } Vsy\_perm \geq Vsy \\ \quad \text{"Ok"} \\ \text{else} \\ \quad \text{"Revise"} \end{array} = \text{"Ok"}$$

$$Check3 := \begin{array}{l} \text{if } Vuy > 0.5 \cdot \phi Vcy \\ \quad \text{"Check for Min. Shear Reinf"} \\ \text{else} \\ \quad \text{"Check for Min. Shear Reinf Not Req"} \end{array} = \text{"Check for Min. Shear Reinf"}$$

$$Asv\_min1 := \max\left(0.75 \cdot \sqrt{f'c \cdot psi} \cdot \frac{B}{fy\_s} \cdot 12 \cdot in, 50 \cdot \frac{B}{fy\_s} \cdot 12 \cdot psi \cdot in\right) = 0.3\ in^2$$

$$Asv\_min := \begin{array}{l} \text{if } Vuy > 0.5 \cdot \phi Vcy \\ \quad Asv\_min1 \\ \text{else} \\ \quad 0\ in^2 \end{array} = 0.3\ in^2$$

$$Asv\_shear\_y := \begin{Vmatrix} \text{if } \phi Vcy \ge Vuy \\ \quad \| 0\ in^2 \\ \text{else} \\ \quad \left\| \dfrac{Vsy \cdot ft}{fy\_s \cdot deff} \right. \end{Vmatrix} = 0.335\ in^2$$

$$Asv\_req\_y := \frac{\max(Asv\_min, Asv\_shear\_y)}{ft} = 0.335\ \frac{in^2}{ft}$$

$$Check := \begin{Vmatrix} \text{if } Asvprv\_D > Asv\_req\_y \\ \quad \| \text{“Ok”} \\ \text{else} \\ \quad \| \text{“Increase Reinforcement”} \end{Vmatrix} = \text{“Ok”}$$

Shear Design along B

Critical Load Combination [13] : 0.9 (LOAD 1: LOAD CASE 1) -(LOAD 4: LOAD CASE 4 EQ-Y)

$Pu\_B = 788.73\ kip$

$Mshear\_B = 12.67\ kip \cdot ft$

$Vux3 = 9.4\ kip$

$$Vux := \max(Vu'x, Vux3) = 28.58\ kip$$

$$\phi := \begin{Vmatrix} \text{if } TypeOfColumn > 2 \\ \quad \| 0.6 \\ \text{else} \\ \quad \| 0.75 \end{Vmatrix} = 0.75$$

----------- Strength Reduction Factor

$$beff := B - Cc - \frac{\phi 1}{2} = 27.5\ in$$

$$pt := \frac{Ast}{2 \cdot D \cdot beff} = 0.0056$$

----------- 50% of total reinforcement assumed as Tension Reinforcement

$Ag := D \cdot B = 1080\ in^2$

$Aeff\_x := beff \cdot D = 990\ in^2$

$$Mm\_x := Mshear\_B - Pu\_B \cdot \frac{(4 \cdot B - beff)}{8} = -747.304\ kip \cdot ft$$

$$Vcx1 := 2 \cdot \left(1 \cdot psi + \left(\frac{Pu\_B}{500 \cdot Ag}\right)\right) \cdot \lambda \cdot \sqrt{f'c \cdot psi} \cdot D \cdot \frac{beff}{psi} = 266.851\ kip$$

----------- Clause 22.5.7.1

$$Vcx2 := \left(1.9 \cdot \sqrt{f'c \cdot psi} + 2500 \cdot pt \cdot psi \cdot \left(\frac{Vux \cdot beff}{Mm\_x}\right)\right) \cdot D \cdot beff = 101.811\ kip$$

----------- Table 22.5.6.1 (a)

$$Vcx3 := \left(3.5 \cdot \sqrt{f'c} \cdot \sqrt{1 \cdot psi + \left(\frac{Pu\_B}{500 \cdot Ag}\right)}\right) \cdot D \cdot beff = 297.704\ kip$$

----------- Table 22.5.6.1 (b)

$\phi Vcx := \begin{cases} \text{if } Pu\_B < 0 \\ \quad Vcx1 \cdot \phi \\ \text{else if } Mm\_x < 0 \\ \quad Vcx3 \cdot \phi \\ \text{else} \\ \quad \phi \cdot min(Vcx2, Vcx3) \end{cases} = 223.28\ kip$

$Check1 := \begin{cases} \text{if } \phi Vcx \geq Vux \\ \quad \text{"ShearReinfNotRequired"} \\ \text{else} \\ \quad \text{"ShearReinfRequired"} \end{cases} = \text{"ShearReinfNotRequired"}$

$fy\_s = 60\ ksi$

$Vsx := \begin{cases} \text{if } \phi Vcx \geq Vux \\ \quad 0\ kip \\ \text{else} \\ \quad \dfrac{(Vux - \phi Vcx)}{\phi} \end{cases} = 0\ kip$

----------- Clause 7.1.1

$Vsx\_perm := 8 \cdot \sqrt{f'c \cdot psi} \cdot D \cdot beff = 433.796\ kip$

$Check2 := \begin{cases} \text{if } Vsx\_perm \geq Vsx \\ \quad \text{"Ok"} \\ \text{else} \\ \quad \text{"Revise"} \end{cases} = \text{"Ok"}$

$Check3 := \begin{cases} \text{if } Vux > 0.5 \cdot \phi Vcx \\ \quad \text{"Check for Min. Shear Reinf"} \\ \text{else} \\ \quad \text{"Check for Min. Shear Reinf Not Req"} \end{cases} = \text{"Check for Min. Shear Reinf Not Req"}$

$Asv\_min1 := \max\left(0.75 \cdot \sqrt{f'c \cdot psi} \cdot \dfrac{D}{fy\_s} \cdot 12 \cdot in, 50 \cdot \dfrac{D}{fy\_s} \cdot 12 \cdot psi \cdot in\right) = 0.36\ in^2$

$Asv\_min := \begin{cases} \text{if } Vux > 0.5 \cdot \phi Vcx \\ \quad Asv\_min1 \\ \text{else} \\ \quad 0\ in^2 \end{cases} = 0\ in^2$

$Asv\_shear\_x := \begin{cases} \text{if } \phi Vcx \geq Vux \\ \quad 0\ in^2 \\ \text{else} \\ \quad \dfrac{Vsx \cdot ft}{fy\_s \cdot beff} \end{cases} = 0\ in^2$

$$Asv\_req\_x := \frac{\max(Asv\_min, Asv\_shear\_x)}{ft} = 0\ \frac{in^2}{ft}$$

$$Check := \begin{cases} \text{if } Asvprv\_D > Asv\_req\_x \\ \quad \text{"Ok"} \\ \text{else} \\ \quad \text{"Increase Reinforcement"} \end{cases} = \text{"Ok"}$$

Detailing of Links

Check for Minimum Diameter (Clause 25.7.2.2)

$$maxDia := \max(\phi1, \phi2) = 1\ in$$

$$Bundled := 1$$

$$Effective\_Area := \frac{\pi}{4} \cdot maxDia^2 \cdot Bundled = 0.785\ in^2$$

$$Effective\_Dia := \sqrt{\frac{Effective\_Area \cdot 4}{\pi}} = 1\ in$$

$$Min\_Dia := \begin{cases} \text{if } Effective\_Dia > 1.41\ in \\ \quad 0.5\ in \\ \text{else if } Bundled > 1 \\ \quad 0.5\ in \\ \text{else} \\ \quad 0.375\ in \end{cases} = 0.375\ in$$

$$Check := \begin{cases} \text{if } \phi3 \geq Min\_Dia \\ \quad \text{"Ok"} \\ \text{else} \\ \quad \text{"Increase Diameter"} \end{cases} = \text{"Ok"}$$

Check for Minimum Spacing (Clause 25.7.2.1)

$$Spc1 := 16 \cdot min(\phi1, \phi2) = 12\ in$$

$$Spc2 := 48 \cdot \phi3 = 18\ in$$

$$Spc3 := B = 30\ in$$

Criterion for spacing for shear reinforcement (Clause 10.7.6.2)

Along D

$$Vsy := \begin{cases} \text{if } \phi Vcy \geq Vuy \\ \quad 0\ kip \\ \text{else} \\ \quad \dfrac{(Vuy - \phi Vcy)}{\phi} \end{cases} = 56.038\ kip$$

$$Vsy\_1 := 4 \cdot \sqrt{f'c \cdot psi} \cdot Aeff\_y = 220.184\ kip$$

$$Spc4 := \begin{cases} \text{if } Vsy \leq Vsy\_1 \\ \quad \dfrac{deff}{2} \\ \text{else} \\ \quad \dfrac{deff}{4} \end{cases} = 16.75\ in$$

$$Spc5 := \begin{cases} \text{if } Vsy \le Vsy\_1 \\ \quad 24\ in \\ \text{else} \\ \quad 12\ in \end{cases} = 24\ in$$

Along B

$$Vsx := \begin{cases} \text{if } \phi Vcx \ge Vux \\ \quad 0\ kip \\ \text{else} \\ \quad \dfrac{(Vuy - \phi Vcy)}{\phi} \end{cases} = 0\ kip$$

$$Vsx\_1 := 4 \cdot \sqrt{f'c \cdot psi} \cdot Aeff\_y = 220.184\ kip$$

$$Spc6 := \begin{cases} \text{if } Vsx \le Vsx\_1 \\ \quad \dfrac{beff}{2} \\ \text{else} \\ \quad \dfrac{beff}{4} \end{cases} = 13.75\ in$$

$$Spc7 := \begin{cases} \text{if } Vsx \le Vsx\_1 \\ \quad 24\ in \\ \text{else} \\ \quad 12\ in \end{cases} = 24\ in$$

$$SpcReq := min(Spc1, Spc2, Spc3, Spc4, Spc5, Spc6, Spc7) = 12\ in$$

$$Check := \begin{cases} \text{if } Spc \le SpcReq \\ \quad \text{“Ok”} \\ \text{else} \\ \quad \text{“Reduce spacing”} \end{cases} = \text{“Ok”}$$

Check for Minimum Area of Shear Reinforcement (Clause 7.10.5.1)

Along D

$$Asv\_minD := \max\left(0.75 \cdot \sqrt{f'c \cdot psi} \cdot \frac{B}{fy\_s} \cdot 12 \cdot in, 50 \cdot \frac{B}{fy\_s} \cdot 12 \cdot psi \cdot in\right) = 0.3\ in^2$$

$$Check := \begin{cases} \text{if } Asvprv\_D \ge \dfrac{Asv\_minD}{ft} \\ \quad \text{“Ok”} \\ \text{else} \\ \quad \text{“Increase Shear Reinf”} \end{cases} = \text{“Ok”}$$

Along B

$$Asv\_minB := \max\left(0.75 \cdot \sqrt{f'c \cdot psi} \cdot \frac{D}{fy\_s} \cdot 12 \cdot in, 50 \cdot \frac{D}{fy\_s} \cdot 12 \cdot psi \cdot in\right) = 0.36\ in^2$$

$$Check := \begin{cases} \text{"Ok"} & \text{if } Asvprv\_B \geq \dfrac{Asv\_minB}{ft} \\ \text{"Increase Shear Reinf"} & \text{else} \end{cases} = \text{"Ok"}$$

Table For Links

Note: Ductile Design of Links is Applicable Only For Boundary Elements

	Required			Provided	
	Normal Design	**Shear Design**	**Ductile Design**	**Normal Zone**	**Ductile Zone**
Link Rebar Number	3	---	---	3	---
Spacing	12	---	---	12	---

RCDC Output - Design Calculation Report

General Data

Column No.	:	C21	
Level	:	-8.25 ft To 0 ft	
Frame Type	=	Non-Ductile	
Response Modification Coefficient	=	3	
Design Code	=	ACI 318 - 14	
Grade Of Concrete (f'c)	=	C3	ksi
Grade Of Steel (Main)	=	Fy60	ksi
Grade Of Steel (Shear)	=	Fy60	ksi
Grade Of Steel - Flexural Design	=	Fy60	ksi
Grade Of Steel - Shear Design	=	Fy60	ksi
Consider Ductile	=	No	
Column B	=	30	in
Column D	=	36	in
Clear Cover, Cc	=	2	in
Clear Floor Height @ lux	=	69	in
Clear Floor Height @ luy	=	69	in
No Of Floors	=	1	
No Of Columns In Group	=	1	

Shear Calculation (Analysis Forces)	**Along D**	**Along B**
lu (in)	69	69
Column Dimension (D , B) (in)	36	30
Check	lu <= 5 x D	lu <= 5 x B
Shear from Moment Capacity		
Lu (in)	69	69
Pu Top (kip)	788.73	788.73
Mnt (kip-ft)	1153.25	957.56
Pu Bottom (kip)	797.09	797.09
Mnb (kip-ft)	1143.83	948.41
Vu1 (kip)	399.49	331.47
Shear from Load combinations with Enhanced Eq factor		
Load Combination	1.2 (LOAD 1: DEAD LOAD) +0.5 (LOAD 2: LIVE LOAD) +3 (LOAD 3: EQ-X)	1.2 (LOAD 1: DEAD LOAD) +0.5 (LOAD 2: LIVE LOAD) +3 (LOAD 4: EQ-Y)
Vu2 (kip)	268.69	28.58
Critical Analysis Load Combination	108	108
Critical Load Combination	[9] : 0.9 (LOAD 1: DEAD LOAD) +(LOAD 4: EQ-Y)	[9] : 0.9 (LOAD 1: DEAD LOAD) +(LOAD 4: EQ-Y)
Nu (kip)	788.73	788.73
Mu (kip-ft)	0.3	12.67
Vu3 (kip)	-1.91	9.4
Vu' (kip)	Minimum(Vu1, Vu2)	
	268.69	28.58
Design Shear, Vu (kip)	Maximum(Vu', Vu3)	
	268.69	28.58
λ	1	1
ϕ	0.75	0.75
Deff (in)	33.5	27.5
ρw (50% of As provided)	0.006	0.006
Mm (kip-ft)	-907.56	-747.31
ϕVc (kip)	226.64	223.26
Check	Vu > ϕVc	Vu < ϕVc
Link For Shear Design	Required	Not Required
Shear Links Design		
Vs (kip)	(Vu - ϕVc) / ϕVc	
	56.07	-
Vs Permissible (kip)	8 x sqrt(f'c) x b x deff	
	440.32	-
Vs Permissible Check	Vs < Vs permissible; Hence, OK	-
Check for Minimum Shear		-
0.5 x ϕVc (kip)	113.32	-
Minimum Shear Reinforcement Check	Vu > 0.5 x ϕVc; Hence, Minimum Shear reinforcement required	-
Av/s minimum (in^2/ft)	0.3	-
Av/s shear (in^2/ft)	0.33	-
Av/s required (in^2/ft)	max (Av/s minimum , Av/s shear)	
	0.33	-
Link Rebar Number	3	-
Diameter of link (in)	0.37	-
Numbers of legs provided	6	-
Spacing of Link Provided (in)	12	-
Av/s provided (in^2/ft)	0.66	-
Av/s provided check	Av/s required < Av/s provided; Hence, OK	